Performance Evaluations on Ditch Check Practices and Products for Channelized Stormwater Runoff Control using Large-Scale Testing

by

Wesley Nelson Donald

A dissertation submitted to the Graduate Faculty of
Auburn University
in partial fulfillment of the
requirements for the Degree of
Doctor of Philosophy

Auburn, Alabama
May 3, 2014

Keywords: Auburn University Erosion and Sediment Control Testing Facility, construction best management practices, erosion control, sediment control, stormwater control

Approved by

Wesley C. Zech, Chair, Associate Professor of Civil Engineering
Xing Fang, Professor of Civil Engineering
Prabhakar Clement, Professor of Civil Engineering
Mark Dougherty, Associate Professor of Biosystems Engineering

ABSTRACT

The primary purpose and focus of this research is to evaluate and improve erosion and sediment control practices deployed in channelized flow conditions on construction sites in an effort to better understand these practices and products. The main focus of this research was to improve the effectiveness of ditch check practices for reducing channel erosion and providing conditions that will also allow sedimentation to occur. The effects of various ditch check practices, installation configurations, and performance evaluations on wattle, sand bag, silt fence, and riprap ditch checks are examined. A large-scale research facility designated as the Auburn University Erosion and Sediment Control Testing Facility (AU-ESCTF) was designed and constructed to meet these requirements. A methodology was developed to test, evaluate, and improve different ditch check installations and applications. From this research, the wattle and rock ditch check installations were modified increasing impoundment performance by 100% for each. Wattle impoundments increased from 10.3 to 20.5 ft as a result of a modified installation. Enhancing the riprap ditch check installation improved performance from 14.5 ft to 29.1 ft. A modified sand bag ditch check installation increased the structural integrity of the practice and allowed it to withstand hydrostatic forces at a high flow rate of 1.68 cfs. A performance criteria was developed using the Froude number (Fr) and a dimensionless relationship that evaluates the ratio of water depth to specific energy (y/E). A recommendation of a minimum y/E = 0.75 was developed through this research.

ACKNOWLEDGEMENTS

The author would like to thank Dr. Wesley Zech, Dr. Xing Fang, Dr. Prabhakar Clement, Dr. Mark Dougherty, Dr. Jeffery LaMondia, and Dr. John Fulton for their time and guidance throughout this research effort. The author would also like to thank the Alabama Department of Transportation (ALDOT) for their financial support, Mr. Barry Fagan of ALDOT for his guidance and support, the National Center for Asphalt Technology (NCAT) Testing Facility for their assistance, and all of the research assistants that came and went throughout this entire project. The author would like to thank his family and most notably his parents, Mr. Randy and Mrs. Joanna Donald, who through love, hard work, and sacrifice, provided the author this opportunity. Finally, the author would like to thank his wife Mrs. Kayla King Donald for the love, support, and sacrifice that she has so graciously given which provided the means and showed the strength required to complete this endeavor.

TABLE OF CONTENTS

ABSTRACT ii

ACKNOWLEDGEMENTS iii

LIST OF TABLES xii

LIST OF FIGURES xiv

CHAPTER ONE INTRODUCTION 1

1.1 Introduction 1

1.2 Background 3

1.3 Stormwater, Erosion, and Sediment Control 7

1.4 Research Objectives 10

1.5 Organization of Dissertation 12

CHAPTER TWO LITERATURE REVIEW 15

2.1 Introduction 15

2.2 Ditch Check Testing Requirements per ASTM D 7208-06 16

2.3 Large Scale Research Facilities 17

2.3.1 Texas Transportation Institute: The Hydraulics, Sedimentation and Erosion Control Lab 17

2.3.2 North Carolina Cooperative Extension: SECREF 20

2.3.3 American Excelsior Company (AEC): The ErosionLab 22

2.3.4 Texas Research International, Inc.: Denver Downs Research Facility 24

2.3.5 Field Observations of Ditch Check Performance 28

2.4 ALDOT Traditional ALDOT Sediment Control Ditch Check Practices 39

2.4.1 Sand Bag Ditch Checks 39

2.4.2 Hay Bale Ditch Checks 39

2.4.3 Silt Fence Ditch Check 40

2.4.4 Wattle Ditch Checks 41

2.4.5 Silt Dike Ditch Check 42

2.4.6 Riprap (Rock) Ditch Check 43

2.5 Ditch Check Application Guidance and Recommendations 44

2.5.1 Alabama Handbook 44

2.5.2 California Department of Transportation 45

2.5.3 Georgia Erosion and Sediment Control Manual 45

2.5.4 North Carolina Department of Transportation 46

2.6 Review Summary 46

CHAPTER THREE AU-ESCTF DESIGN AND CONSTRUCTION 48

3.1 Introduction 48

3.2 Test Facility Location 48

3.3 Facility Design 50

3.3.1 Topographical Survey 50

3.3.2 Facility Design 53

3.3.3 Hydrologic Design 58

3.3.4 Test Channels 65

3.4 Summary 71

CHAPTER FOUR CONSTRUCTION OF THE AU-ESCTF 72
4.1 Introduction 72
4.2 Clearing and Grubbing 72
4.3 Grading and Excavation 74
4.3.1 Retention Pond 75
4.3.2 Sediment Basin 84
4.3.3 Upper Storage Pond and Riprap Channels 86
4.3.4 Test Channels Excavation and Construction 88
4.4 Summary 97
CHAPTER FIVE METHODOLOGY 99
5.1 Introduction 99
5.2 Test Flow System 99
5.2.1 Design Storm and Drainage Area 100
5.2.2 Pump System 105
5.3 Test Channel Preparation 109
5.4 Data Collection 112
5.5 Ditch Check Installation Improvement Testing Methodology 115
5.5.1 Bare Soil Control 116
5.5.2 Installation Improvement Testing Regime 116
5.6 Installation Methodologies 118
5.6.1 Wattle Installations 118
5.6.2 Riprap Installation 122
5.6.3 Silt Fence Installation 125
5.6.4 Sand Bag Ditch Check 129

5.7 Statistical Analysis ... 131
5.8 Summary ... 133
CHAPTER SIX INSTALLATION IMPROVEMENTS ... 134
6.1 Introduction ... 134
6.2 Bare Soil Control Test ... 135
6.3 Wattle Installation Improvements ... 136
6.3.1 Wattle Installation Descriptions ... 137
6.3.2 Wattle Installation Improvement Results and Discussion ... 139
6.3.3 Statistical Analysis Results ... 145
6.3.4 Wattle Installation Improvements Conclusions ... 147
6.4 Riprap Installation Improvements ... 148
6.4.1 Riprap Ditch Check Practice and Modifications ... 150
6.4.2 Riprap Installation Improvement Results and Discussion ... 151
6.4.3 Riprap Installation Improvements Conclusions ... 154
6.5 Silt Fence Installation Improvement ... 155
6.5.1 Silt Fence Installation Improvements Results and Discussion ... 156
6.5.2 Downstream Dissipation Installation Evaluation ... 159
6.5.3 Silt Fence Installation Improvements Summary ... 167
6.6 Sand Bag Installation Improvements ... 168
6.6.1 Sand Bag Installation Improvements Results and Discussion ... 168
6.6.2 Sand Bag Installation Improvement Conclusion ... 173
6.7 Summary ... 174
CHAPTER SEVEN PRODUCT EVALUATION TESTING ... 178
7.1 Introduction ... 178

7.2 Current ALDOT Approved Materials ... 179
7.3 Product Approval Results and Discussion ... 183
7.3.1 Wattle Material Comparisons ... 183
7.3.2 Wattle Material Properties Evaluation Conclusions ... 192
7.4 Wattle Product Performance Criteria ... 193
7.4.1 Failure Modes ... 193
7.4.2 Performance Criteria Data Analysis ... 194
7.5 Summary ... 211
CHAPTER EIGHT COST ANALYSIS ... 214
8.1 Introduction ... 214
8.2 Cost of Ditch Checks ... 215
8.2.1 Unit Cost ... 215
8.2.2 Further Ditch Check Cost Comparisons ... 222
8.3 Cost Comparisons versus Performance ... 223
8.4 Summary ... 226
CHAPTER NINE CONCLUSIONS AND RECOMMENDATIONS ... 228
9.1 Summary and Recommendations ... 228
9.2 Installation Evaluation and Improvements Summary ... 229
9.3 Product Performance Criteria ... 231
9.4 Cost Summary ... 232
9.5 Conclusions ... 233
9.6 Limitations and Recommendations for Future Work ... 235
9.7 Acknowledgements ... 237

REFERENCES 238

APPENDIX A: ALDOT ESC – 300: DITCH CHECK STRUCTURES, TYPICAL

APPLICATIONS AND DETAILS.. 245

APPENDIX B: WATTLE PRODUCT SPECIFICATION SHEETS... 255

APPENDIX C: WATTLE TIER FLOW AVERAGE TEST DATA.. 279

LIST OF TABLES

Table 1.1: Natural and Artificial Erosion, Sediment, and Stormwater BMPs 8

Table 2.1: Erosion and Sediment Control Testing Facility Capabilities 27

Table 3.1: ESC Facility Preconstruction Runoff Characteristics.. 61

Table 3.2: ESC Facility Post Construction Runoff Characteristics 62

Table 3.3: Retention Pond Routing Information... 65

Table 3.4: Comparison of AU-ESCTF Channels vs. ASTM D 7208-06 Standard 66

Table 5.1: Volume and Runoff for Montgomery County, AL 2-year, 24-hour Event 104

Table 5.2: Manufacturer Specified Flow Rates and Vol. for Northstar® 3 in. Trash Pump 105

Table 5.3: Flow Rate Determination for Each Pump (Trough Volume = 20.4 ft^3) 107

Table 5.4: Sieve Analysis of Earthen Section Soil 110

Table 5.5: Properties of Earthen Section Soil 111

Table 5.6: Proctor Test Data for Earthen Section Soil 112

Table 6.1: Western Excelsior EXCEL Straw Log Manufacturer Material Specifications 136

Table 6.2: Comparative Results of Each Wattle Installation Configuration and the Control..... 140

Table 6.3: Statistical Relationships of Installation Components 146

Table 6.4: Comparative Results of Each Riprap Installation Configuration 152

Table 6.5: Comparative Results of Each Sand Bag Installation Configuration......................... 168

Table 7.1: Manufacturer Specified Dimensions of Wattles Tested 181

Table 7.2: Manufactured Wattles with Product Density and Resultant Flow Depths 184

Table 7.3: Determination of Significance between Wattle Material Groups............................. 191

Table 7.4: Low Flow Rate Wattle Ditch Check Results... 197

Table 7.5: Medium Flow Rate Wattle Ditch Check Results.. 198

Table 7.6: Results of Small Scale Channel Test of Froude Number vs y/E for 0.2 cfs.............. 204

Table 8.1: Cost Analysis of New Wattle Installation per ALDOT Division............................. 219

Table 8.2 Wattle Ditch Check Performance versus Spacing Requirements............................. 224

Table 8.3: Cost of Channel Erosion Control (per 150 ft)[1] 225

Table 9.1: Required Ditch Check Product Performance Criteria 232

Table 9.2: Cost of Channel Erosion Control (per 150 ft)[1] 233

LIST OF FIGURES

Figure 1.1: Time Line of Construction and Development Category Effluent Limitation Guidelines and New Source Performance Standards.. 6

Figure 1.2: Typical Ditch Check Structures used by ALDOT (ALDOT 2012). 10

Figure 2.1: TTI Sediment Retention Device Exp. Setup (McFalls et al 2010)............................ 19

Figure 2.2: TTI Sediment Collection and Discharge System (McFalls et al. 2010).................... 19

Figure 2.3: NCSU SECREF Facility (McLaughlin 2001)... 21

Figure 2.4: SECREF Ditch Check Testing (McLaughlin 2010).. 21

Figure 2.5: Plan View of The ErosionLab.. 23

Figure 2.6: Profile View of AEC's Open Channel Flume Test Plots... 23

Figure 2.7: TRI's Denver Downs Research Facility (TRI 2010).. 24

Figure 2.8: TRI's ASTM D 6459 Setup (TRI 2008). 25

Figure 2.9: TRI's ASTM D 7208 Setup and Test (Courtesy of Earl Norton: 2008). 25

Figure 2.10: TRI's ASTM D 7351 (TRI 2008)... 25

Figure 2.11: TRI's Drop Inlet Testing Area. (Courtesy of Earl Norton: 2008)........................... 26

Figure 2.12: Installation of a Wattle Check Dams using PAM Dosing. (King 2011)................. 32

Figure 2.13: Small Scale Test Results of Check Dams Tested at 3 Slopes. (Dalton 2011).......... 34

Figure 2.14: Field Scale Check Dam Results for Four Storm Events (Dalton 2011).................. 35

Figure 2.15: Sand Bag Ditch Check on ALDOT Project in Franklin County, AL...................... 39

Figure 2.16: Typical Hay bale Ditch Check Installations... 40

Figure 2.17: ALDOT Specified Silt Fence Ditch Check.. 41

Figure 2.18: Wattle Ditch Check Applications... 42

Figure 2.19: Silt Dike Ditch Check.. 42

Figure 2.20: Rock Ditch Checks. 44

Figure 3.1: Preconstruction Aerial View of the Facility Location (Google Earth). 49

Figure 3.2: Aerial View of Proposed Site Conditions (Google Earth). 51

Figure 3.3: Topographical Rendering of the Proposed Site's Watershed. 52

Figure 3.4: Topographical and Profile View of the Proposed ESC Facility Location. 53

Figure 3.5: Topographical Plan View Rendering of the AU-ESCTF Design. 55

Figure 3.6: Three-Dimensional Rendering of AU-ESCTF Layout. 56

Figure 3.7: Win TR-55 Storm Data for Lee County, Alabama. 60

Figure 3.8: Win TR-55 Input of the ESC Facility Preconstruction Hydrologic Details. 61

Figure 3.9: Original Retention Pond and ESC Facility Design. 63

Figure 3.10: Final ESC Facility Topographical Design. 64

Figure 3.11: MicroStation Drawing of the Erosion and Sediment Control Test Channels. 67

Figure 3.12: MicroStation Drawing of the Inlet Protection Device Test Channel. 68

Figure 3.13: 3-Dimensional Rendering of the ESC Facility Test Channels. 69

Figure 3.14: Isometric View of the AU-ESCTF Experimental Channels. 70

Figure 4.1: Installation of the ESC Plan Temporary Riser Pipe. 73

Figure 4.2: Inlet and Outlet of Control Structures Used for the ESCP. 73

Figure 4.3: Clearing and Grubbing Stages of the AU-ESCTF. 74

Figure 4.4: Installation of the Retention Ponds Riser and Outlet Pipe. 76

Figure 4.5: Relocation of the Temporary Riser Pipe for Sediment Control. 76

Figure 4.6: Permanent Riser Pipe for the AU-ESCTF Lower Retention Pond. 77

Figure 4.7: Pond Liner Installation Set Up. 78

Figure 4.8: Progression of the Akwaseal GCL Installation. 80

Figure 4.9: Initial Erosion Control and Slope Stabilization Measures on Retention Pond........... 81

Figure 4.10: Initial Retention Pond Stabilization and Erosion Problems. 82

Figure 4.11: New Temporary Stabilization of the ESC Facility's Retention Pond. 83

Figure 4.12: Retention Pond After Rain Event. 84

Figure 4.13: Construction Progression of the Sediment Basin. 85

Figure 4.14: Construction Progression of the Storage Pond. 87

Figure 4.15: Location of Riprap Lined Drainage Channels. 88

Figure 4.16: Rough Excavation of the AU-ESCTF Experimental Test Channels. 89

Figure 4.17: Excavation of the Conveyance and Sampling Area of the Right Channel. 90

Figure 4.18: Fine Grading of the Prototype Test Channel. 91

Figure 4.19: Installation Progression of the Prototype Sheet Metal Channel Liner. 92

Figure 4.20: Initial Testing to Evaluate Ditch Check Channel Testing Feasibility. 93

Figure 4.21: Construction of Permanent Ditch Check Conveyance Channels. 94

Figure 4.22: Construction of Permanent Inlet Protection Discharge Area. 95

Figure 4.23: Construction of Second Ditch Check Channel. 96

Figure 4.24: Construction of Drop Inlet Channel. 97

Figure 4.25: Final View of Completed Construction Effort. 97

Figure 5.1: Southeast U.S. SCS Rainfall Distributions. 101

Figure 5.2: I-85 Field Survey Single Drainage Area Aerial. 102

Figure 5.3: Runoff Hydrograph for Montgomery County, AL 2-year, 24-hour Event. 104

Figure 5.4: Trough Openings and Flow Levels at Various Flow Conditions. 106

Figure 5.5: 90° Sharp Crested V-notch Weir. 108

Figure 5.6: Test Data vs. Weir Equation.. 109

Figure 5.7: Grain Size Distribution for Earthen Section. 110

Figure 5.8: Proctor Curve for Earthen Section Soil. 111

Figure 5.9: Ditch Check Test Channel Dimensions and Configuration. 113

Figure 5.10: Ditch Check Data Collection Process. 114

Figure 5.11: Bare Soil Control Average EGL, WSE, and Velocity. 116

Figure 5.12: Ditch Check Testing Regime for Determining the MFE-I. 117

Figure 5.13: Comparison of ALDOT (ALDOT 2012) and NCDOT Wattle Installation Practices (NCDOT 2012). 120

Figure 5.14: American Excelsior Premier Straw Wattle^{TM} Install Guide (AEC 2013). 121

Figure 5.15: ALDOT Riprap Ditch Check Installation Detail (ALDOT 2012). 123

Figure 5.16: Standard ALDOT Silt Fence Ditch Check Detail (ALDOT 2012). 125

Figure 5.17: Tennessee Department of Transportation Enhance Silt Fence Installation (TDOT 2012). 126

Figure 5.18: ALDOT Sand Bag Ditch Check Installation (ALDOT 2012). 129

Figure 6.1: Spacing Guide Based upon Channel Slope and Ditch Check Height. 135

Figure 6.2: Ditch Check Test Channel Dimensions and Configuration. 136

Figure 6.3: Control and Seven Wattle Installations Tested. 139

Figure 6.4: Comparison of Installations Energy Grade Line (EGL) and Water Surface Elevation (WSE) to the Control Test with No Wattle Installed. 142

Figure 6.5: Test Comparison of Trenched Wattle Configurations. 143

Figure 6.6: Installation Comparison with and without Staples. 144

Figure 6.7: Adopted ALDOT Wattle Installation Detail (ALDOT 2012). 148

Figure 6.8: Alabama Handbook Recommended Ditch Check Spacing. 149

Figure 6.9: Riprap Ditch Check Installation Configurations. 151

Figure 6.10: Average Performance of Each Riprap Ditch Check Installation. 153

Figure 6.11: Riprap Ditch Check Choker Comparisons. 154

Figure 6.12: Silt Fence Structural Failures. 156

Figure 6.13: Standard ALDOT Silt Fence Ditch Check Detail (ALDOT 2012). 158

Figure 6.14: Standard V-Installation Silt Fence Ditch Check Tier Flow Test. 159

Figure 6.15: Standard ALDOT Silt Fence Installation. 160

Figure 6.16: Downstream Dissipation Modifications for V-Installation Silt Fence. 161

Figure 6.17: TDOT Enhanced Silt Fence Installation (TDOT 2012). 162

Figure 6.18: TnDOT Enhanced Silt Fence Ditch Check Test. 163

Figure 6.19: Pinned Silt Fence Installation Test. 164

Figure 6.20: Sediment-Laden Silt Fence Longevity Test. 166

Figure 6.21: Erosion and Deposition Patterns of Silt Fence Longevity Test. 166

Figure 6.22: Standard ALDOT Sand bag Installation. 169

Figure 6.23: Sand Bag Burrito Installation on ALDOT Project in Franklin County, AL. 169

Figure 6.24: Sand Bag Burrito Ditch Check Installation. 170

Figure 6.25: Sand Bag Burrito Ditch Check Test. 170

Figure 6.26: Second Iteration of the Modified ALDOT Sand Bag Installation. 172

Figure 6.27: Modified ALDOT Sandbag Installation. 173

Figure 7.1: ALDOT Materials List II-24. 180

Figure 7.2: Excelsior Fiber Wattles Tested. 181

Figure 7.3: Wheat Straw Wattles Tested. 182

Figure 7.4: GeoHay: GeoWattle. 183

Figure 7.5: Ditch Check Test Channel Dimensions and Configuration. 184

Figure 7.6: Excelsior Wattles Results for Low Flow Conditions. 186

Figure 7.7: Wheat Straw Wattle Results for Low Flow Conditions. 188

Figure 7.8: GeoHay GeoWattle Results for Low Flow Conditions. 188

Figure 7.9: Impoundment Depth vs. Density Relationships. 190

Figure 7.10: Low Flow Relationship of HGL:EGL vs. Depth. 199

Figure 7.11: Low Flow Relationship of HGL:EGL vs. Froude Number. 201

Figure 7.12: Low Flow Relationship of y/E and Froude Number. 202

Figure 7.13: Reformatted Low Flow Relationship of y/E and Froude Number. 203

Figure 7.14: Medium Flow Fr vs. y/E Relationship. 203

Figure 7.15: All Lab Scenarios Plotted Froude Number vs. y/E. 206

Figure 7.16: Froude Number vs. y/E for Theoretical and Measured Data. 207

Figure 7.17: Performance Criteria Based Upon Avg. Performance and Theoretical Data 210

Figure 8.1: Unit Cost of Wattles per Linear Foot for ALDOT Construction Projects. 218

Figure 8.2: Unit Cost of Sand Bags per Bag for ALDOT Construction Projects. 221

Figure 8.3: Spacing Guide Based upon Channel Slope and Ditch Check Height. 224

CHAPTER ONE

INTRODUCTION

1.1 Introduction

Construction and development (C&D) activities can generate new sources of water pollution by altering the physical characteristics of construction sites (Benik et al. 2003). Such activities typically involve clearing existing vegetation, excavating, and grading which may lead to increased amounts stormwater runoff and high erosion rates (EPA 2009). The construction of roadways typically consist of mass clearing and grading leaving many site areas unstable, lacking ground cover to protect against rainfall induced erosion. During clearing and grading activities, topsoil is stripped away which can reduce infiltration and increase the amount of runoff and erosion from these unstabilized areas. Sediment discharging off-site, which is a nonpoint source pollutant (NPS), is a major concern for regulatory legislators (Fennessey and Jarrett 1997, Zech et al. 2009). Zech et al. (2008) states that accelerated erosion from construction sites can contribute large amounts of sediment to the stream network and degrade overall water quality, and can occur in just a short period of time. As linear roadway projects progress, unstabilized areas (i.e., roadbeds, cut and fill slopes, and other embankments) tend to be highly compacted, reducing infiltration. The amount of stormwater runoff for a particular construction site can vary throughout the construction life-cycle based upon changing site conditions. Stormwater runoff from unstabilized grading operations on construction sites can yield sediment losses of 17 to 45 ton acre^{-1} year^{-1} (ASCE 1992, Mamo and Budenzer 2004). Eroded sediment from construction sites is one of the most harmful pollutants to the environment resulting in over 80 million tons of sediment washing from construction sites into surface water bodies each year (Novotny 2003, Bidelspach et al. 2004). In linear construction, stormwater runoff is typically diverted to a series of constructed stormwater conveyances (i.e., berms, swales, and ditches), which may also be unstabilized prior to vegetative establishment.

Maintaining stormwater conveyances is crucial to stormwater management and flow control. If the stormwater is not properly controlled, it can cause major erosion within conveyance channels, and has the potential to transport heavy sediment loads off-site, potentially to nearby water bodies (Ooshaksaraie et al. 2009). Therefore, runoff control measures must be installed to minimize channel erosion, especially during peak periods of a storm event. Stormwater runoff control is the practice of managing concentrated flows and reducing peak runoff caused by modifications of the site topography.

Construction site managers are constantly trying to balance construction phases with sound stormwater, erosion, and sediment control implementation. McLaughlin et al. (2009a) states that most construction sites are required to have sediment control devices to intercept and treat runoff prior to discharge, but the devices typically installed are generally inefficient. It is imperative that designers understand the applications and capabilities of common erosion and sediment control practices throughout the various phases or stages of construction (Line and White 2001). As construction scheduling is evaluated and the stormwater pollution prevention plan (SWPPP) is developed, it is important for designers and contractors to understand the capabilities of these different practices in maintaining or improving stormwater quality.

1.2 Background

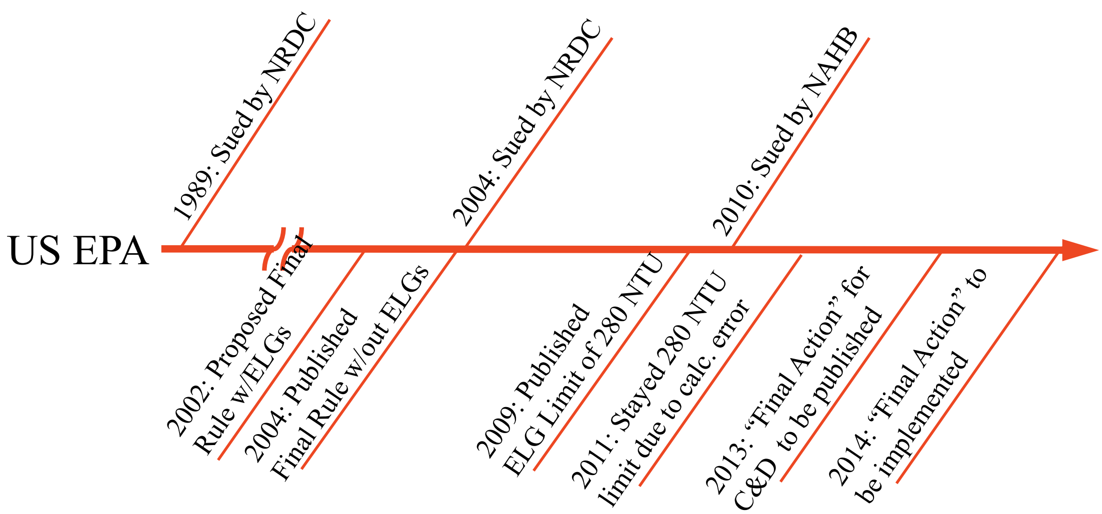
Controlling sediment-laden stormwater from construction sites is a major concern for the Environmental Protection Agency (EPA). In 1972 the EPA created the National Pollutant Discharge Elimination System (NPDES) as a part of the Clean Water Act (CWA). This system was enacted to regulate pollutant discharges into any U.S. water body for the purpose of restoring and maintaining the chemical, physical, and biological integrity of the nation's waters (CWA). Initially this regulation only considered point source pollutants and therefore pollution resulting from stormwater (i.e., nonpoint sources) was left unregulated. Upon determining the effects stormwater pollution had on water bodies and aquatic life, the EPA enacted regulations to include nonpoint source pollution (NPS) from stormwater runoff in its permitting programs. These regulations were passed by Congress amending the CWA and are known as the Water Quality Act of 1987. Phase I of these new regulations, developed in 1990, were applied to large and then medium sized municipalities (i.e., populations of 100,000 and above) including construction activities greater than 5 acres and ten categories of industrial activity. Phase II of the NPDES was enacted into law in 1999 and went into effect in 2003 as a regulatory device for small municipal separate storm sewer systems (MS4s) (i.e., populations less than 100,000) and also included construction activities of 1 to 5 acres (EPA 2005). The Pollution Prevention Act (PPA) was passed November 1990 making pollution prevention the national policy in the U.S. It states that pollution "should be prevented or reduced whenever feasible; pollution that cannot be prevented should be recycled in an environmentally safe manner, whenever feasible; pollution that cannot be prevented or recycled should be treated in an environmentally safe manner whenever feasible; and disposal or release into the environment should be employed only as a last resort (EPA 2009)."

A need exists for regulating construction activities that discharge sediment-laden stormwater from construction sites when compared to other sources. Construction sites can average 200 tons acre^{-1} year^{-1} of soil loss, where as one ton acre^{-1} year^{-1} of sediment is estimated to be lost from forestlands, and agriculture lands lose fifteen tons acre^{-1} year^{-1} (Faucette et al., 2005). The EPA uses a method called the best practicable control technology (BPT) to determine effluent limitations for conventional, toxic, and nonconventional pollutants (EPA 2009). The EPA considers 'cost to effluent reduction' relationship, the process used, and other engineering aspects for controlling point source pollution. The CWA requires the EPA to identify effluent reduction levels for conventional pollutants and perform a two-part cost-reasonableness evaluation. This led to the EPA assigning a best conventional pollutant control technology (BCT) for conventional pollutants which include biochemical oxygen demand, total suspended solids (TSS), fecal coliform, pH, and any additional pollutants deemed appropriate by EPA. New source performance standards (NSPS) were developed to determine achievable effluent reductions based upon best available demonstrated control technology. These NSPSs provide new facilities with the opportunity to install the best and most efficient production process and wastewater treatment technology. This reasoning has been applied to the construction industry whereas construction sites may be considered new facilities with the ability to implement the best and most efficient process and technologies to minimize pollution as a result of stormwater runoff, erosion, and sedimentation.

The Natural Resources Defense Council, Inc. (NRDC) and Public Citizen, Inc., filed an action against the EPA citing that the agency failed to comply with section 304(m) of the CWA which states:

> *"Within 12 months after the date of the enactment of the Water Quality Act of 1987, and biennially thereafter, the Administrator shall publish in the Federal Register a plan which shall (A) establish a schedule for the annual review and revision of promulgated effluent guidelines, in accordance with subsection (b) of this section, (B) identify categories of sources discharging toxic or nonconventional pollutants for which guidelines under subsection (b)(2) of this section and section 306 have not previously been published; and (C) establish a schedule for promulgation of effluent guidelines for categories identified in subparagraph (B) under which promulgation of such guidelines shall be no later than 4 years after such date of enactment for categories identified in the first published plan or 3 years after the publication of the plan for categories identified in later published plans."*

A settlement was agreed upon in January of 1992 and as a result the EPA agreed to propose and take final action for 11 point source categories identified in the lawsuit settlement and develop new or revised rules for 8 other point source categories. The C&D category was included in the new or revised rule category. Based upon this rule, EPA was required to propose an effluent limitation guideline (ELG) for the C&D category by May 15, 2002. Final action was to be taken before April 1, 2004. From this, EPA developed regulatory options for discharges emanating from construction, development, and redevelopment sites which are included in Phase I and Phase II of the National NPDES stormwater rules. The EPA was also required to develop numeric effluent limitations for sedimentation and turbidity and required to control other construction site pollutants such as discarded building materials, concrete truck washout, trash, etc. EPA was also to identify best management practices (BMPs) for construction activities, BMPs for post construction stormwater runoff, as well as require designers to maintain predevelopment runoff conditions when practicable. However, in 2004, EPA published a determination that national ELGs would not be the most effective way to control discharges from construction sites and reverted back to previous regionally adapted stormwater management rules. Due to this, NRDC, Waterkeeper Alliance, State of New York, and State of Connecticut filed a motion against EPA stating it failed to meet the ELG and NSPS requirements set forth by the CWA. In 2008, the court ordered EPA to publish proposed regulations by December 2008 and to adopt and publish ELGs and NSPS for the C&D category no later than December 2009. A numeric ELG of 280 nephelometric turbidity units (NTU), which is a measurement of the fine suspended sediment content of water (Davies-Colley and Smith 2001), was published in the Federal Registry December 1, 2009 as the numeric ELG for the C&D industry. Figure 1.1 shows a brief timeline of the progression of the ELG and NSPS development for the C&D category. After publishing this “C&D rule”, the Wisconsin Builders Association, the National Association of Home Builders (NAHB), and the Utility Water Act Group petitioned the EPA to reconsider the rule due to potential errors in the ELG calculation and misinterpretation of data used to develop the 280 NTU limit. The EPA acknowledged this error and stayed the limit on January 4, 2011 in order to gather more data. However, upon further review of the data, the EPA agreed to amend many of the non-numeric BMP requirements and withdraw the numeric limit altogether (EPA 2013). Final action is required to be published in 2013 and to be implemented by 2014.



Note: Natural Resources Defense Council (NRDC)
Effluent Limitations Guidelines (ELG)
Nephlometric Turbidity Units (NTU)
National Association of Home Builders (NAHB)
Construction and Development (C&D)

Figure 1.1: Time Line of Construction and Development Category Effluent Limitation Guidelines and New Source Performance Standards.

1.3 Stormwater, Erosion, and Sediment Control

Sedimentation and erosion are two major effects of stormwater runoff. Erosion is a catalyst for sediment transport and eventually sedimentation, the act where particles removed (or eroded) from the soil by stormwater (or other climatic means) are transported to another location and eventually settle out. Stormwater runoff can cause erosion and result in sediment transport, and leading to sedimentation via three hydrologic flow conditions: sheet flow (interrill), shallow concentrated flow, and open channel flow. These three flow conditions typically build upon one another as one condition transforms into another until full sediment load flow carrying capacity is reached. A steady-state rate of sediment transport is reached once the rate of entrainment equals the rate of deposition. For example, the steady-state flux for cohesionless soils is the maximum rate at which flow can transport sediment and therefore is the sediment transport capacity of that flow and sediment combination (Prosser and Rustomji 2000).

The erosion process typically involves interrill erosion which is the initial detachment process of soil material from the soil surface as a result of raindrop impact and runoff shear stresses applied to the soil surface which leads to the transport of sediment (Levy et al 1994, Zahng et al. 2002). Raindrop impact detachment is much greater than runoff detachment in this flow stage since the kinematic energy of the raindrop impact is much greater than the shear force of sheet flow runoff. However, as flow concentrates within exposed areas, this interrill erosion develops into rill erosion and in time will expand into larger gully erosion. As this process continues, the runoff created by the flow concentration is responsible for transporting sediment from the area down gradient to flow conveyances and to local water bodies.

As a result, stormwater pollutants, sedimentation, and turbidity are major environmental concerns when managing stormwater runoff (McCaleb and McLaughlin 2008). Many different measures are implemented to control and mitigate stormwater and sediment transport. Limiting travel times and travel paths by means of diversions are practices used to deter detachment of sediment which typically occur under concentrated flow conditions. However, once sediment-laden flow has entered channels, treatment time becomes limited and crucial. Structural controls, chemical treatment, and sediment basins are typical methods used to slow water velocities by impounding water and provide additional control of sediment-laden stormwater and sedimentation. Specific examples of erosion, sediment, and stormwater controls are listed in Table 1.1.

Table 1.1: Natural and Artificial Erosion, Sediment, and Stormwater BMPs

Technique	Natural	Artificial
Erosion Control	Temporary Seeding; Permanent Seeding; Sodding	Chemical Stabilization; Erosion Control Blankets; Turf Reinforcement Mats; Mulching
Sediment Control	Brush Barrier; Vegetated Buffers	Filter Berms; Fibrous Rolls; Sediment Basins and Traps; Silt Fences; Storm Drain; Inlet Protection; Ditch Checks; Check Dams
Stormwater Control	Grass Lined Swales; Streams	Inlet Structures; Porous Pavement; MS4s; Detention/Retention Basins; Diversion Berms

The Office of Water for the EPA states:

> *"…runoff controls are essential to preventing polluted runoff from roads, highways, and bridges from reaching surface waters. Erosion during and after construction of roads, highways, and bridges can contribute large amounts of sediment and silt to runoff waters, which can deteriorate water quality and lead to fish kills and other ecological problems. Heavy metals, oils, other toxic substances, and debris from construction traffic and spillage can be absorbed by soil at construction sites and carried with runoff water to lakes, rivers, and bays. Runoff control measures can be installed at the time of road, highway, and bridge construction to reduce runoff pollution both during and after construction. Such measures can effectively limit the entry of pollutants into surface waters and ground waters and protect their quality, fish habitats, and public health."* (EPA 1995)

Ditch checks, which are runoff controls, are defined as either permanent or temporary structures constructed across runoff conveyances, intended to slow and impound stormwater runoff, reduce shear stresses causing channel erosion, and create favorable conditions for sedimentation (EPA 2006, ALDOT 2012, NCDOT 2012, GWSCC 2000, ASTM 2007).

Practitioners typically space ditch checks based upon geometry (i.e., ditch check height, channel slope, and channel cross-sectional geometry) (NCDOT 2012, ASTM 2007, TDECA 2012, NDDOT 2004). However, this spacing practice does not take into consideration that ditch checks are typically not impervious barriers and have some degree of flow-through, which includes water flowing under, through, or over a structure. The amount of flow-through may depend upon the installation method, stormwater flow, and material composition (i.e., material type or density). As a result of this flow-through, during low flow conditions, ditch checks may not impound water the entire height of the ditch check. The impoundment length, therefore, will not extend the entire length upstream between ditch checks, as assumed during design, thereby exposing the channel to higher velocity flow, potentially making that section of a channel susceptible to erosion. A wattle, which may be used as a ditch check or slope intercept device depending on site-specific requirements, is a manufactured, tubular device composed of natural or synthetic fillers (i.e., compost material, wheat straw, excelsior [wood shaving], coir, carpet fiber, or recycled rubber tires) encased in a natural fiber or synthetic netting. The advantages of using wattles as ditch checks over other types of ditch checks (i.e., rock, hay bales, silt fence, etc.) include: (1) its biodegradability, (2) typically lightweight, (3) easily installed using minimum resources, (4) economical, and (5) are available in various dimensions making them adaptable to site specific constraints. Some limitations of using wattles as ditch checks include: (1) their elliptic shape may reduce surface area available for ground contact with the channel resulting in undermining and scour, and (2) the potential for lightweight wattles becoming buoyant, reducing adequate ground contact while securing the wattle in place under concentrated flows. Other ditch checks that could also be considered based upon site specific constraints include: rock, silt fence, sand bag, hay bales, and manufactured devices. Figure 1.2 shows the ditch check options typically used on Alabama Department of Transportation (ALDOT) highway construction projects. These are found in ALDOT ESC-300*: Ditch Check Structures, Typical Applications and Details* which can be found in Appendix A.

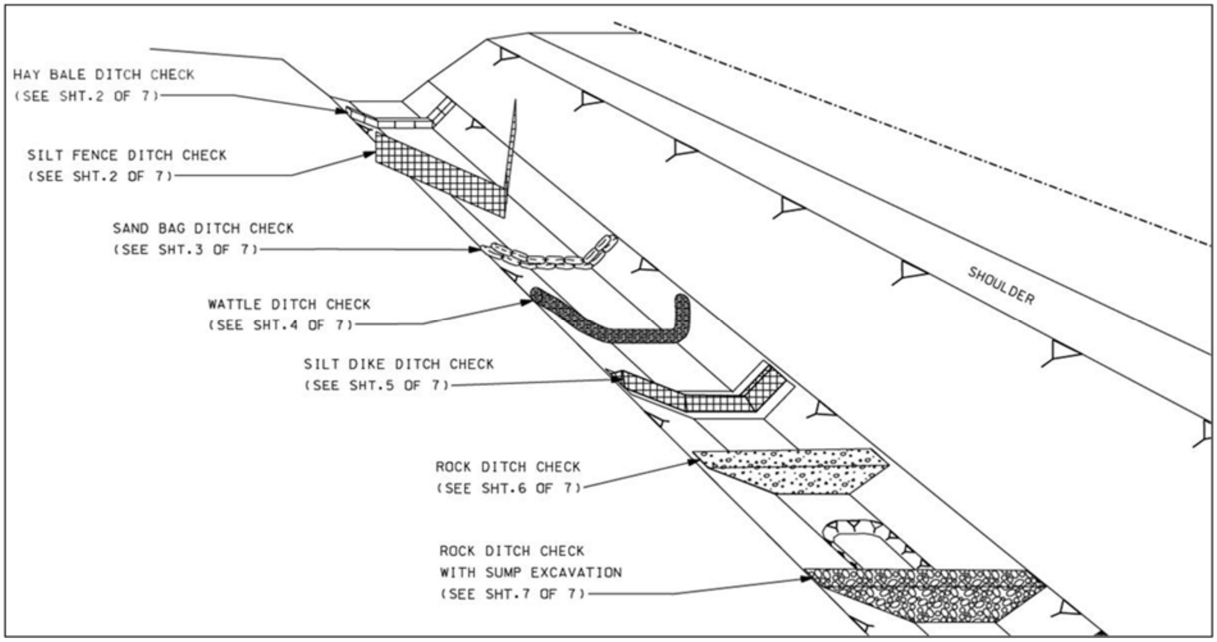


Figure 1.2: Typical Ditch Check Structures used by ALDOT (ALDOT 2012).

1.4 Research Objectives

The primary purpose of this research was to evaluate and improve erosion and sediment control practices typically deployed in channelized flow conditions. This research focused on improving the effectiveness of typical ditch check practices used on construction sites for reducing channel erosion and providing conditions that also allow sedimentation to occur. This is a major concern with discharge limitations becoming increasingly more stringent as the EPA attempts to reduce sediment load impacts on waters of the US. This required developing a controlled repeatable method for which to scientifically evaluate these typical practices. Once a repeatable methodology was developed, it was used to evaluate the overall performance of various ditch check practices under consideration as part of this research effort.

The first objective of this research was to design and construct a large-scale test facility at the National Center for Asphalt Technology (NCAT) Test Track for testing the effectiveness of selected erosion and sediment control practices deployed on highway construction sites. The development of this large-scale test facility, deemed the Auburn University Erosion and Sediment Control Testing Facility (AU-ESCTF), provides the ALDOT and other state highway agencies (SHAs) a controlled environment for testing stormwater, erosion, and sediment controls in a controlled large-scale environment.

The second objective of this research was to evaluate various channelized stormwater runoff controls used as ditch check applications typical to ALDOT projects. The purpose of this was to establish design, installation, and maintenance guidelines for ALDOT, and other erosion and sediment control practitioners, to use in development of their SWPPPs and proper implementation in the field. This objective involved developing a testing methodology that results in a consistent and repetitious testing regime that allows for comparative analyses between ditch check products and practices.

The third objective was to develop a performance criteria that can be used to evaluate the effectiveness of ditch check practices. This is important for determining products and practices that are deemed acceptable as ditch check practices. This will also provide a means for to evaluate manufactured products submitted for consideration to be used on construction sites.

The final objective was to evaluate the cost factors of various ditch check practices. The data collected for this portion of the research emanates from bid prices submitted by contractors to ALDOT for highway construction projects. This assists designers in specify ditch check practices more appropriately which can reduce costs SWPPPs.

The specific tasks to satisfy the abovementioned research objectives are as follows:

Task 1: Design and construct a large-scale testing facility specifically designed to evaluate channelized stormwater controls. Once constructed, a test methodology that is practical, repeatable, and representative of conditions that are encountered on highway construction sites was developed,

Task 2: Evaluate the performance of current ALDOT ditch check installation practices from the standard drawing to improve performance which will include: (1) wattles, (2) riprap, (3) silt fence, and (4) sand bags,

Task 3: Develop a performance criteria based upon ditch check capabilities for decreasing velocity and increasing impoundment depths and providing acceptable product approval thresholds, and

Task 4: Compare the cost of ditch check practices while taking into consideration performance which will help guide designers in selecting particular ditch checks for expected project specific site conditions.

1.5 Organization of Dissertation

This report is divided into nine chapters. Chapter Two: *Literature Review* examines the different facilities where large-scale testing and other relevant testing and evaluations have been performed with regards to ditch check applications. The chapter examines other research studies that have evaluated ditch check practices which include overall site descriptions and test methods. This chapter discusses the American Society for Testing and Materials (ASTM) standards used to test large-scale erosion and sediment control products and practices. Relevant SHAs standard ditch check installation practices are also discussed.

Chapter Three: *Design of the AU-ESCTF*, discusses: (1) the overall design, (2) the means and methodology used to develop the hydrologic evaluation, and (3) the detailed construction drawings of the facility.

Chapter Four: *Construction of the AU-ESCTF,* discusses and outlines the construction effort of the facility and explores the construction process along with problems encountered.

Chapter Five: *Testing Methodology* outlines the methods developed for testing different ditch check practices. This testing methodology was developed and then verified to ensure the methodology was repeatable and capable of properly evaluating each individual ditch check practice.

Chapter Six: *Ditch Check Installation Improvements* focuses on evaluating ALDOT's current ditch check installation practices, and developed enhancements for increasing the ditch checks performance capabilities. Recommendations based upon performance, feasibility, and cost are made for each ditch check practice.

Chapter Seven: *Product Performance* evaluates wattle products from the ALDOT approved materials list "List II-24: Temporary Erosion and Sediment Control Products" to evaluate performance of each wattle's ability to function as a ditch check. From this a performance criteria has been developed to assist practitioners in developing approval/rejection criteria for products on List II-24 and for new products submitted for approval.

Chapter Eight: *Cost Evaluation* evaluates the cost of particular ditch check practices and compare each practice to determine appropriate use of the practices based upon channelized stormwater control needs.

Chapter Nine: *Conclusions and Recommendations* summarizes the findings of this dissertation, provide recommendations for implementation, discusses limitations, and future research opportunities.

CHAPTER TWO

LITERATURE REVIEW

2.1 Introduction

The need for effective and scientific evaluation of erosion and sediment control practices, and devices has become of greater importance with more stringent EPA effluent guidelines and limitations. Determining the effectiveness of such practices and devices is difficult when monitoring field installations at construction sites. McLaughlin et al. (2001) states, "field testing of existing and new sediment and erosion control products or systems has been problematic when conducted on active construction sites. Uncertainty about runoff quantity and quality due to weather patterns and construction activities makes objective, replicated experiments difficult."

Several facilities both private and public dedicated to testing such devices and practices in bench- and large-scale conditions have been constructed and expanded over the past two decades. As a result, a large range of American Society for Testing and Materials (ASTM) standards have been developed to help govern testing of the different materials, practices, and devices. The majority of these ASTM standards concentrate on material testing in laboratory bench-scale settings. ASTM standards for large-scale testing of erosion and sediment control practices include:

- *ASTM D 6459-07: Test Method for Determination of Rolled Erosion Control Product (RECP) Performance in Protecting Hillslopes from Rainfall-Induced Erosion*
- *ASTM D 6460-07: Test Method for Determination of Rolled Erosion Control Product (RECP) Performance in Protecting Earthen Channels from Stormwater-Induced Erosion*
- *ASTM D 7208-06: Test Method for Determination of Temporary Ditch Check Performance in Protecting Earthen Channels from Stormwater-Induced Erosion*
- *ASTM D 7351-07: Test Method for Determination of Sediment Retention Device Effectiveness in Sheet Flow Applications*

The primary focus of this literature review was on examining: (1) existing test protocols used for evaluating product performance under channelized flow, (2) existing large-scale testing facilities, and (3) methods and systems used to test ditch check products and practices in channelized applications.

2.2 Ditch Check Testing Requirements per ASTM D 7208-06

ASTM D 7208-06 states it is to be used as a performance and comparative tool for evaluating the erosion control characteristics of different temporary ditch checks (by reducing channelized water velocity) and can be used for quality control to determine product conformance to project specifications. The channel design requirements for ASTM 7208 are: (1) a trapezoidal cross-section, (2) a 2 ft bottom width, (3) 2H:1V side slopes, (4) a minimum length of 60 ft, and (5) a bed slope of approximately 5%. The reason for the channel length is twofold. A minimum of 40 ft of actual testing area is required to allow for proper spacing between ditch checks. The remaining 20 ft of testing area is required to allow the water enough distance to reach stable, uniform flow before entering the testing area as well as enough area to properly convey flow leaving the test area into the discharge channel that feeds into the discharge channel.

The flow rate specified by this standard is 3 cfs. No specific guidance regarding the justification of this flow rate was provided.

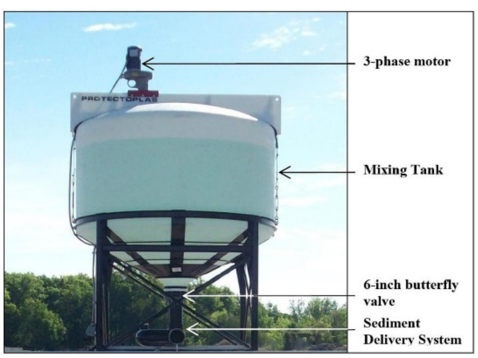
2.3 Large Scale Research Facilities

Ten large-scale facilities were researched for the purpose of this project. Six facilities have the capabilities to test ditch check practices. The Colorado State University performs large scale channelized hydraulic testing, however no specific discussion of ditch check testing was found. The University of Central Florida Stormwater Lab has a channel dedicated to evaluating flocculent dosing in concentrated flow applications. However, no literature specifically testing ditch checks were discovered. The following four labs conduct ditch check testing and were evaluated for the purpose of this research study: (1) The Texas Transportation Institute (TTI) at Texas A&M University runs the TTI/TxDOT Hydraulics, Sedimentation and Erosion Control Lab (HSECL), (2) the North Carolina Cooperative Extension has created Sediment and Erosion Control Research and Education Facility (SECREF), (3) the American Excelsior Company (AEC) has The ErosionLab, and (4) Texas Research International (TRI) Denver Downs Research Facility (DDRF).

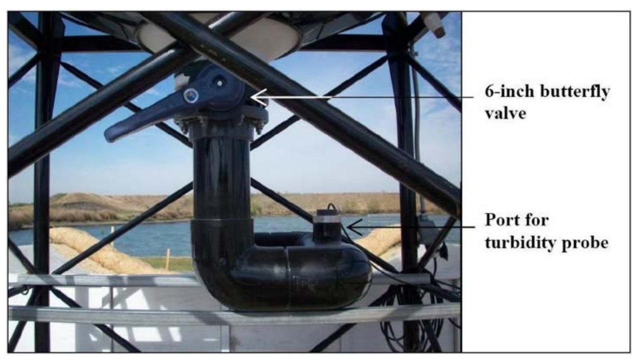
2.3.1 Texas Transportation Institute: The Hydraulics, Sedimentation and Erosion Control Lab

The TTI HSECL facility is located in College Station, TX. HSECL is a 19 acre facility encompassing: (1) an indoor rainfall simulator, (2) variable slope channel testing, (3) sediment retention device testing flume, (4) a climate controlled green house for vegetative testing, (5) 10 at grade channels, (6) outdoor embankment stability testing, (7) overland sheet flow testing, (8) a mobile rainfall simulator, and (9) a small footprint stormwater quality structure (Texas Transportation Institue 2013).

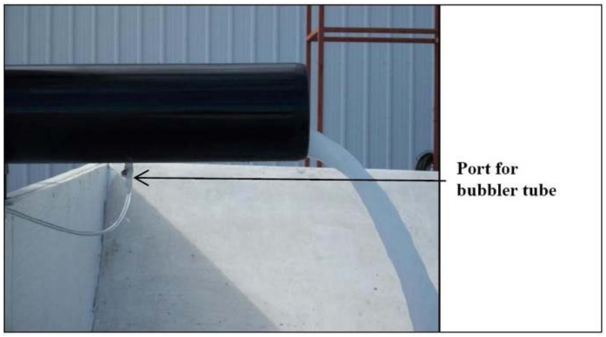
McFalls et al. (2010) describes the sediment retention device testing flume as a multi section parabolic shaped flume. This channel is specifically dedicated to testing ditch check practices and products for the Texas Department of Transportation. The channel is 15 ft wide and 18 ft long at a 3% longitudinal slope divided into: a retention zone, an installation section, and a collection section. The retention zone is 12 ft long and is constructed of concrete. The installation section is a 4 ft long earthen section. The collection section is a 2 ft concrete section down grade of the earthen section. The maximum water depth is 2.5 ft at the center. The water introduction system for this test consists of a mixing tank, valve, a turbidity monitor, and a flow meter. A 3-phase motor powering double mixing paddles is used for mixing. The tank is a 1,600 gallon polypropylene cylindrical tank. Flow is release by a 6 in. butterfly valve which releases sediment-laden runoff through the 6 in. conveyance pipe. Figure 2.1(a) - Figure 2.1 (c) illustrates this delivery system. Figure 2.1(d) is an illustration depicting the different zones of the flume system.



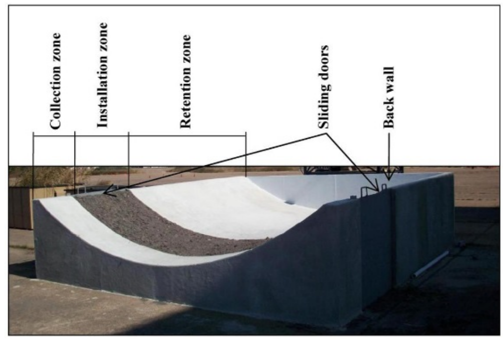
(a) slurry mixing tank



(b) mixing tank release valve and turbidity meter



(c) flow monitoring device and slurry discharge



(d) testing flume

Figure 2.1: TTI Sediment Retention Device Exp. Setup (McFalls et al 2010).

A sheet metal flume collects the runoff after it passes the sediment control device and conveys it to another 6 in. pipe for discharge. A turbidity probe and flow meter are also attached to this pipe. Figure 2.2 shows the setup for the water collection system.

The sediment used for testing is a 50-50 mixture by weight of SIL-CO-SIL® 49 and ball clay. SIL-CO-SIL® 49 is silica sand purchased from the manufacturer U.S. Silica Company and ball clay is kaolinite clay. This mixture was used to create a test slurry with a consistent gradation as well as flocculation capabilities for sediment control devices that uses flocculants. Figure 2.2 shows the outlet pipe discharging this slurry into a collection pool.

(a) flume collection system

(b) sediment-laden water discharge system

Figure 2.2: TTI Sediment Collection and Discharge System (McFalls et al. 2010).

The flow rate used for this testing effort was approximately 60 gals/min which equates to 0.13 cfs. Mcfalls et al believes that overtopping is considered a failure mode, and each device is therefore tested to determine the maximum flow rate allowable for which overtopping does not occur and classified as such. This testing method presumes that the primary purpose of a ditch check is to act as sediment control and limiting these devices to such low flow rates could limit the applicability of ditch check practices for extreme low flow conditions only. It should also be noted that this test is performed at a 3% slope and overtopping can vary based upon channel slope (i.e., it will require more flow for devices to overtop in shallower channels than steeper sloped channels). This limitation may also limit the effectiveness of these performance classifications.

2.3.2 North Carolina Cooperative Extension: SECREF

The North Carolina Cooperative Extension's Sediment and Erosion Control Research Facility (SECREF) is located at the Lake Wheeler Road Field Laboratory in Raleigh, North Carolina. It is part of the Soil and Water Environmental Technology Center at NC State University. The original intent of this facility as reported by McLaughlin et al (2001) was to test: (1) inlet protection, (2) sediment traps and rock dams, (3) sediment fence practices and (4) runoff conveyance measures. The inlet protection test evaluates fabric, block and gravel, and sod protection devices to determine the reduction of sediment from stormwater before being discharged into the inlet. The rock dams and temporary sediment trap tests evaluate different design practices under different design storm conditions to increase sediment removal efficiency, increase stability, and reduce washout. The sediment fence test evaluates silt fence performance when intercepting sediment-laden stormwater from hill slopes. The runoff conveyance measures test evaluates different channel stabilization practices such as fibrous mats, seeding, riprap, and check dam practices.

An 80,000 gal pond supplies test waters to the different testing areas of the facility. A 12 in. diameter pipe conveys the water from the storage pond to the different test locations by way of a computer controlled electric valve. This valve system is able to discharge water at a variable rate to mimic different flow conditions and patterns. Automatic samplers are installed at the inlet and outlet areas. A constructed wetland and forebay captures and treats all runoff before discharging. Figure 2.3 shows a plan view sketch of the SECREF facility. Figure 2.4(a) shows the ditch check spacing test and demonstration channel and Figure 2.3(b) shows the fiber check dam and polyacrylamide dosing test channel.

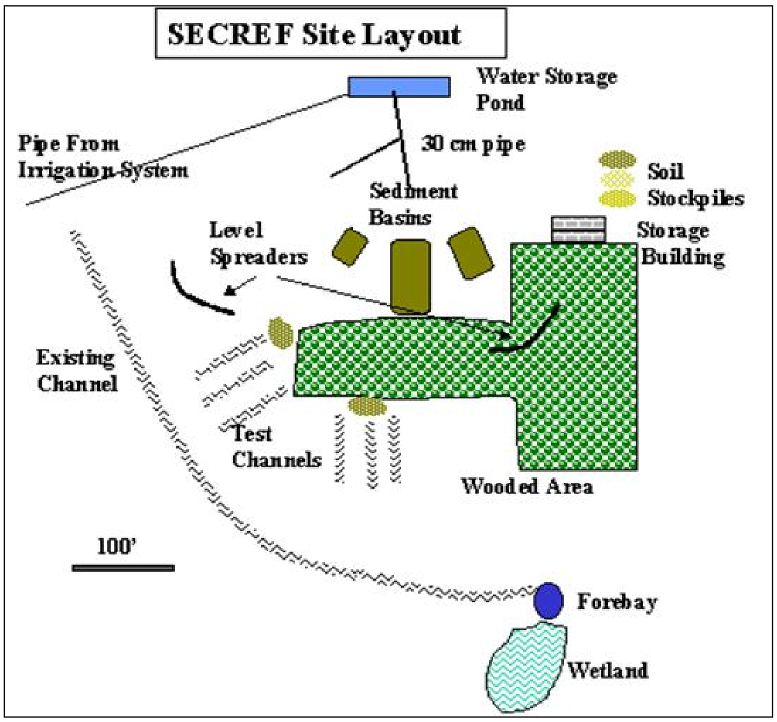


Figure 2.3: NCSU SECREF Facility (McLaughlin 2001).

(a) ditch check spacing performance channel (b) fiber check dam and PAM dosing test channel

Figure 2.4: SECREF Ditch Check Testing (McLaughlin 2010).

The focus of this facility appears to be directed towards sediment control. Wattles and other ditch check devices are frequently evaluated to determine reductions in stormwater runoff turbidity and TSS using flocculation aids. Though a modified wattle ditch check installation was developed by McLaughlin, these modifications appear to be based upon qualitative evaluations and not based upon specific performance improvement data. The overall goal of these installations are to minimize undercutting and force flow to overtop the wattles. This practice will passively dose the runoff with flocculants in an attempt to remove the sediment from the runoff.

2.3.3 American Excelsior Company (AEC): The ErosionLab

AEC is a privately owned company which manufactures products from packaging and cushioning to erosion control and engineered wood fibers (i.e., excelsior). The ErosionLab functions as AEC's large-scale facility located in Rice Lake, WI which tests rainfall erosion, channel erosion and sediment control performance on the products they manufacture. Cabalka and Clopper (1997) describe the facility as being built to satisfy AEC's need to: (1) document performance of existing AEC products, (2) examine competitive materials, (3) evaluate different method of installation, and (4) develop new solutions for erosion control applications. Figure 2.5 shows a plan view drawing of the test channels locations and set up.

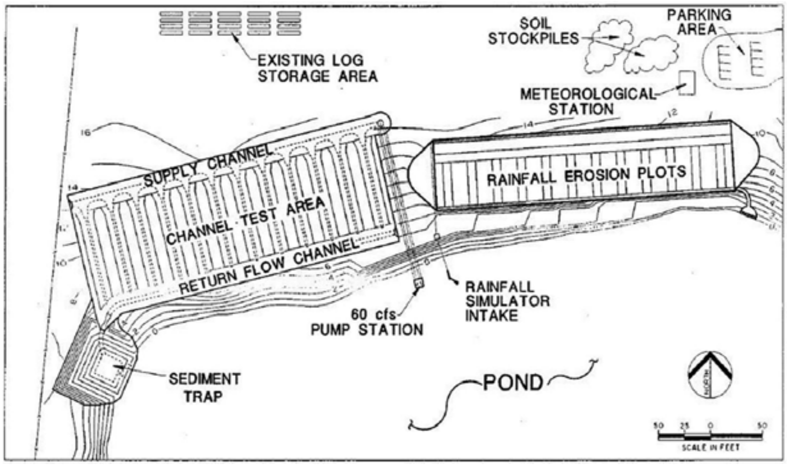


Figure 2.5: Plan View of The ErosionLab.

The *Channel Test Area* was originally designed to test RECP in channelized applications. The test area is comprised of 12 trapezoidal channels with 2 ft bottom widths and 2H:1V side slopes. Six of the channels were graded to a longitudinal slope of 5% and the other six were graded to a longitudinal slope of 10%. The channels are 85 ft long. However, only a forty foot middle section of each channel is actually used for product evaluation where as the sections above and below the forty foot sections are dedicated to inflow and outflow transition zones. A pump station located on the 5 acre pond capable of pumping a maximum discharge of 60 cfs was built for the open-channel flume testing. Figure 2.6 shows the profile view of the open channel testing sections.

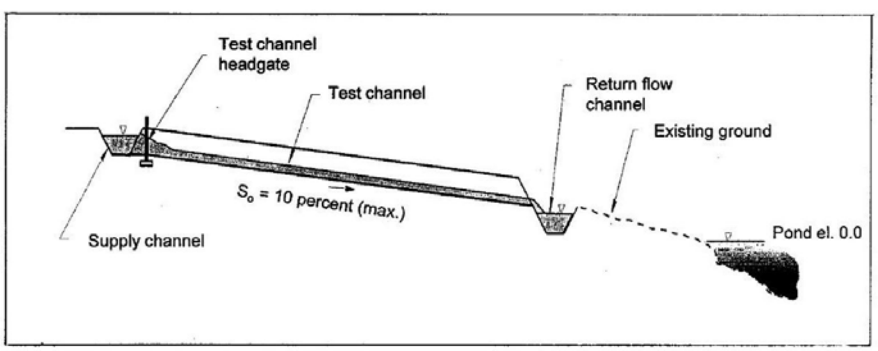


Figure 2.6: Profile View of AEC's Open Channel Flume Test Plots.

ASTM D 6460-07 which tests RECP performance in channel erosion applications as well as ASTM D 7208-06 which tests temporary ditch checks in earthen channels appear to have been developed by AEC though no literature specifically stating this was discovered. The ASTM setup for both tests identically matches the setup at The ErosionLab including the drawings shown in Figure 2.5 and Figure 2.6. The researcher in charge of The ErosionLab confirmed the method used at this facility was used to develop ASTM D 7208-06.

2.3.4 Texas Research International, Inc.: Denver Downs Research Facility

Texas Research International, Inc. (TRI) is a privately owned company dedicated to product testing only at both bench and large-scales. TRI Environmental at the Denver Downs Research Facility (DDRF), Figure 2.7, tests erosion and sediment controls including: rolled erosion control products (RECPs), sediment retention fiber rolls (SRFRs) also known as wattles, membrane barriers, plastic pipe, hydraulically applied erosion control product (HECP), erosion control mulches, logs, geotextiles, and inlet protection devices. This facility is dedicated to performing material and device testing per the ASTM standards. It has been setup to test all four previously mentioned large-scale tests: ASTM D 6459 (Figure 2.8), 6460 [Figure 2.9(a)], 7208 [Figure 2.9(b)], and 7351 (Figure 2.10) (TRI, 2010). TRI is also performing drop inlet testing as shown in Figure 2.11.

Figure 2.7: TRI's Denver Downs Research Facility (TRI 2010).

Figure 2.8: TRI's ASTM D 6459 Setup (TRI 2008).

(a) ASTM D 6460 ASTM D 7208

Figure 2.9: TRI's ASTM D 7208 Setup and Test (Courtesy of Earl Norton: 2008).

(a) sheet flow intercept pretest setup (b) wattle ponding sediment-laden sheet flow

Figure 2.10: TRI's ASTM D 7351 (TRI 2008).

(a) drop inlet test (b) drop inlet testing area

Figure 2.11: TRI's Drop Inlet Testing Area. (Courtesy of Earl Norton: 2008).

Table 2.1 shows a list of facilities that perform stormwater, erosion, and sediment control products and practices testing and training.

Table 2.1: Erosion and Sediment Control Testing Facility Capabilities

Facility	Capabilities and Attributes								
	Material Testing	Ditch Checks	Inlet Protection	Rainfall Simulator	Sediment Basins	Water Quality	Vegetative Studies	Runoff Control	Training
[1]TRI Environmental	X	X	X	X		X	X		X
[2]The Erosion Lab	X	X		X		X	X		X
[3]TTI-HSECL		X		X		X	X	X	
[4]NC State SECREF		X		X	X	X		X	X
[5]CSU Hydraulics Lab		X		X		X	X	X	
[6]SDSU Soil Erosion Research Lab				X		X		X	
[7]Shasta College Erosion Control Training Facility									X
[8]UCF-The Stormwater Lab	X	X	X	X		X		X	X
[9]City of Griffin, GA Sediment and Erosion Control Facility		X	X	X	X	X			X
[10]USDA-National Erosion Research Lab				X			X		

Locations of facilities: (1) Anderson, SC; (2) Rice Lake, WI; (3) Bryan, TX; (4) Raleigh, NC; (5) Fort Collins, CO; (6) San Diego, CA; (7) Redding, CA; (8) Orlando, FL, (9) Griffin, GA; (10) West Lafayette, IN.

2.3.5 Field Observations of Ditch Check Performance

McEnroe and Treff (1997) performed a study which investigated the effectiveness of Kansas Department of Transportation's (KDOT) temporary erosion control measures. This investigation was a qualitative study using field observations of different erosion and sediment control practices. The majority of the construction sites used silt fence and hay bales as ditch checks, as perimeter controls and as inlet protection. To qualitatively determine performance, McEnroe and Treff evaluated the measures by: (1) determining if substantial erosion was being prevented, (2) was sediment effectively trapped, (3) did soil migrate onto adjacent property, (4) did failure occur and if so, why, and (5) what improvements would make the measure more effective and less expensive. From this, the researchers realized the majority of the observed failures were caused by errors in implementation and installation. These errors included improper design and placement, use of substandard materials, and a lack of attention to detail during installation. Based on comments from field personnel during their study, the authors speculated that the reasons for most of these errors would be due to a basic misunderstanding of how temporary erosion control practices perform. From this, the authors surmised that the success of these practices is largely dependent on how well practices are installed and maintained.

After evaluating other SHA's erosion and sediment control practices, McEnroe and Treff implemented some new practices for field evaluation. A triangular silt dike (TSD) was one of two new ditch checks that were field tested and compared to hay bale ditch checks. The authors noted the ease of installation as the TSDs were installed in half the time of conventional hay bale ditch checks and required no heavy equipment. The TSDs came with a geotextile underlay attached to the bottom of the dike which protects the channel from scour near the ditch check.

This underlay apron helped minimize the scour on the downstream side from overtopping flows which was a major problem with hay bale ditch checks. Rock ditch checks were recommended for steep sloped channels and/or channels that will be conveying high flow rates. Rock ditch checks are structurally more stable than hay bale or silt fence ditch checks and did not fail in channels that had previously used hay bales that failed. However, these devices do require extra resources such as heavy equipment for installation. Bio-logs which were a primitive type of wattle were also field evaluated. These devices were essentially erosion control blankets rolled up and placed across the span of the ditch and secured using sod staples. They were deemed ineffective due to extensive undermining.

All of McEnroe and Treff's recommendations were based upon observations and no quantitative guidance was offered. This was most likely a function of performing field observations on construction sites that lacked the equipment and personnel to monitor specific storm events and flow conditions.

Though McLaughlin et al. (2001) discusses the many challenges to on-site field evaluation and monitoring, the NC Cooperative Extension still participates in extensive field testing and research of erosion and sediment control devices and practices at construction sites. Particularly onsite testing and monitoring is performed on roadside wattle ditch checks and sediment basins. McLaughlin et al (2009b) specifically discusses improving runoff emanating from construction sites by controlling sediment with fiber check dams (FCDs) and polyacrylamide (PAM). Note that check dams are considered to be the same practice as ditch checks for the purpose of this study.

McLaughlin et al. (2009b) performed a study to determine the effectiveness of passive dosing treatment of stormwater runoff using wattles and flocculants. This was performed by evaluating the effectiveness of wattles with and without PAM, for reducing sediment and turbidity in stormwater runoff on two construction sites. This study compares these practices to standard rock check dams on two North Carolina Department of Transportation (NCDOT) road paving projects.

Both sites employed small sediment traps constructed of rock check dams preceded by sumps. Two different wattle types composed of two different materials, coir and wheat straw, were also installed at both sites. One coir wattle was installed for every three wheat straw wattles. The coir logs were 12 in. in diameter and 10 ft long. The straw wattles were 9 in. in diameter and10 ft long. Both wattles were installed and secured using stakes and sod staples. Gaps between the wattles and ground were filled with cut pieces of erosion control blankets. Site 1 channels were lined with erosion control blankets due to the steepness of the channels. Only excelsior erosion control blanket underlays were installed for site 2 wattles leaving the remaining areas of the channel between the check dams unlined. These extended 3 ft downstream of the wattles to help alleviate downstream scour. Installation was estimated to take 15 minutes per wattle.

At site 1, from June 2006 to March 2007, the average turbidity for the stormwater runoff was 3,813 NTU for the rock check dams, 202 NTU for the wattles only, and 34 NTU for wattles which used PAM. Average turbidity in discharges at site 2 was reduced from 867 NTU for the rock check dams compared to 115 NTU for the wattle check dams with PAM. A decrease in sediment loading was also noticed between the rock and wattle check dams. At site 1, the rock check dam lost an average of 944 lbs of sediment per storm event compared to just 4.6 lbs for the FCDs-only and 2.0 lbs for the FCDs with PAM. At site 2, the rock check dams lost an average of 7.3 lbs per storm event compared with 1.8 lbs for the wattles with PAM.

Though this research focused more on the capabilities of check dams as sediment controls for reducing turbidity and total suspended solids (TSS) while evaluating the performance of PAM, some observations were made with regards to the performance of the check dams as erosion controls. Wattles tended to perform better in low flow conditions than rock check dams. The rock check dams typically had little to no pool in low flow conditions resulting in eroded channels directly upstream. As in the study performed by McEnroe and Treff, no specific flow data was included in this study to help actually quantify the actual flow constraints.

McLaughlin et al. (2009b) notes that proper check dam spacing is very important for erosion reduction. The ideal check dam spacing has water impounding back up the slope to the immediate downstream side of the upstream check dam. Therefore, the spacing is a function of check dam height/diameter, channel geometry, and channel slope. This creates a series of subcritical flowing pools that reduce the shear force within the ditch. Energy is transferred as water flows through, over and/or under the check dams. Because the greatest energy transfers occur at this interface, some type of channel armoring is required to dissipate energy and maintain channel integrity. Figure 2.12 shows an example of this type of installation with passive treatment.

(a) coir wattle w/PAM dosing check dam (b) straw wattle w/PAM dosing check dam

Figure 2.12: Installation of a Wattle Check Dams using PAM Dosing. (King 2011).

Dalton (2011) created a Public Works Technical Bulletin to provide guidance for the selection and use of check dams for erosion control. These check dams were tested for sheet flow and shallow concentrated flow conditions, and not necessarily for channelized applications, however, the check dams tested in this study were similar to the ditch checks used by ALDOT. The objectives of this study were to: (1) identify the types of commonly used and available check dam systems, (2) evaluate each structure in a field setting and a controlled rainfall simulator, (3) investigate the performance, durability, and installation ease of the check dams to increase land managers' understanding of the types of check dams available and the characteristics of each, and (4) provide costs associated with the check dam structures evaluated, and the pros and cons of those structures.

Dalton investigated the effectiveness of five types of check dams: rip-rap, compost filter berm, plastic grid dam, triangular foam berm, and compost sock. These were tested under three different slope conditions, 6:1, 9:1, and 12:1. Quantitative analysis was conducted by comparing the runoff volume and sediment load from the check dams in both laboratory and field conditions.

The small-scale testing was performed using two horizontal tilting soil chambers measuring 12 ft long, 5 ft wide, and 1 ft deep. All five of the different check dams were tested under three different slopes for the study to determine their performance. The check dams were installed based upon previously identified installation and maintenance specifications. A computer-controlled laboratory rainfall simulator installed 33 ft from the floor applied the rainfall to the soil. An intensity of 1.71 in./hr for 30 minutes was used to represent a 10 year, 30 minute rainfall event in Central Illinois. The entire runoff volume was collected while a single sample volume was dried to calculate sediment load based on ASTM D 3977.

For each test run, the control, which was a bare earthen slope, yielded higher runoff and sediment loads than the tests with check dams. Typically, as slope increased, runoff volumes and sediment loads also increased. The compost sock seemed to yield the highest runoff and sediment values. However, based upon data evaluation, the compost berm and the plastic grid dam performed well consistently at all slope conditions tested. The foam berm performed well at the lower slope, but lost its effectiveness as slope increased. The results of this small scale testing effort is shown in Figure 2.13.

Dalton reports that for all cases, the control yielded higher runoff and sediment loads than the check dams. Also with the exception of the runoff volumes for the 12:1 slope condition, as slope increased runoff volumes and sediment loads also increased. The compost sock yielded higher runoff and sediment values for all tests whereas the plastic grid dam performed consistently better than the control and all the other check dams for slopes of 6:1 and 9:1. Dalton et al. does acknowledge that they later realized the compost socks filler did not meet manufacturer specifications and may have contributed to its poor performance.

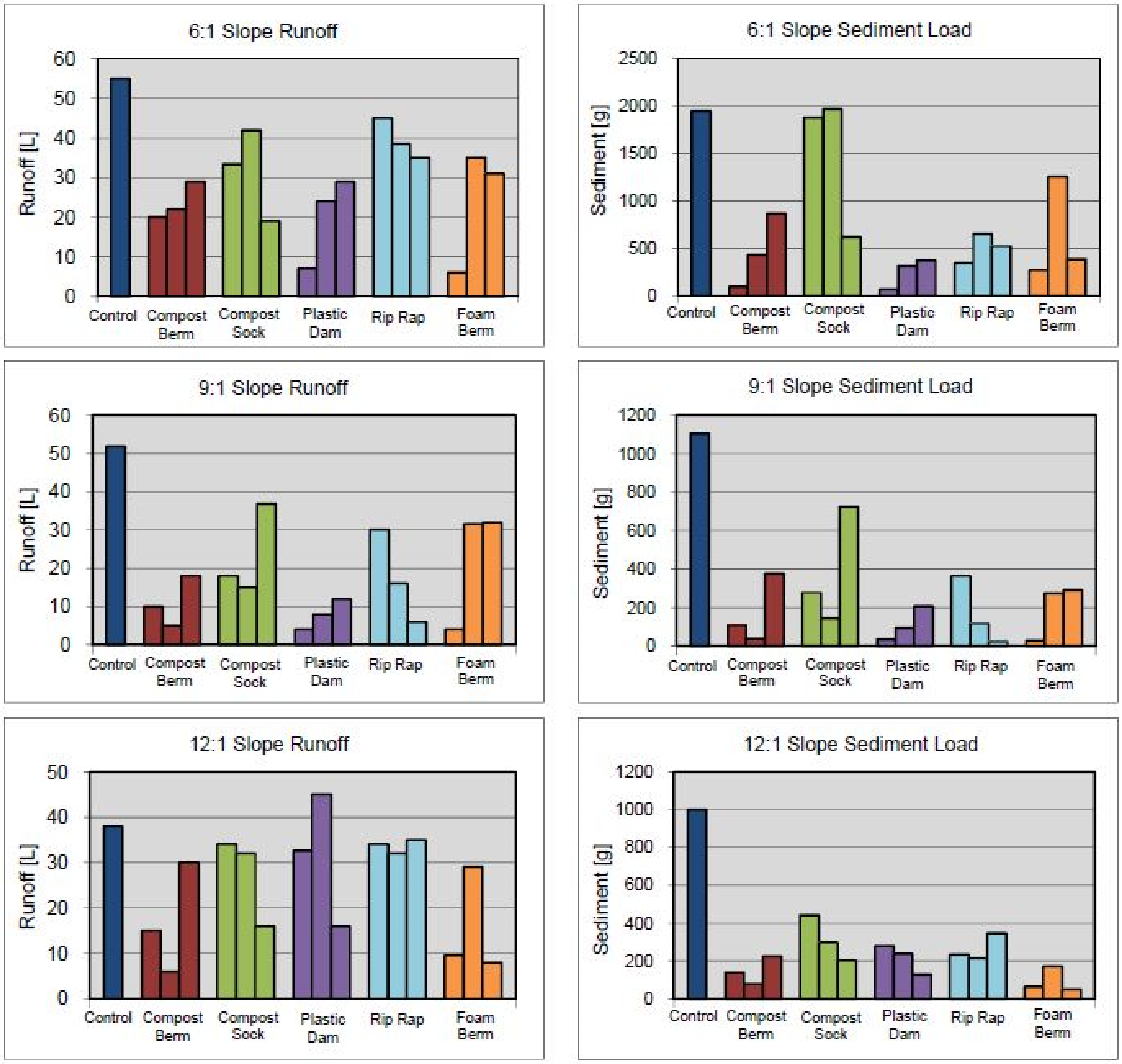


Figure 2.13: Small Scale Test Results of Check Dams Tested at 3 Slopes. (Dalton 2011).

For the field experiments, each check dam was installed 16 ft from the beginning of the plots. Three repetitions for each treatment were installed. A single bare plot was installed as a control for all of the replications since space was limited. There were six storm events that occurred during the study. However, only four produced any measurable runoff. These are shown in Figure 2.14.

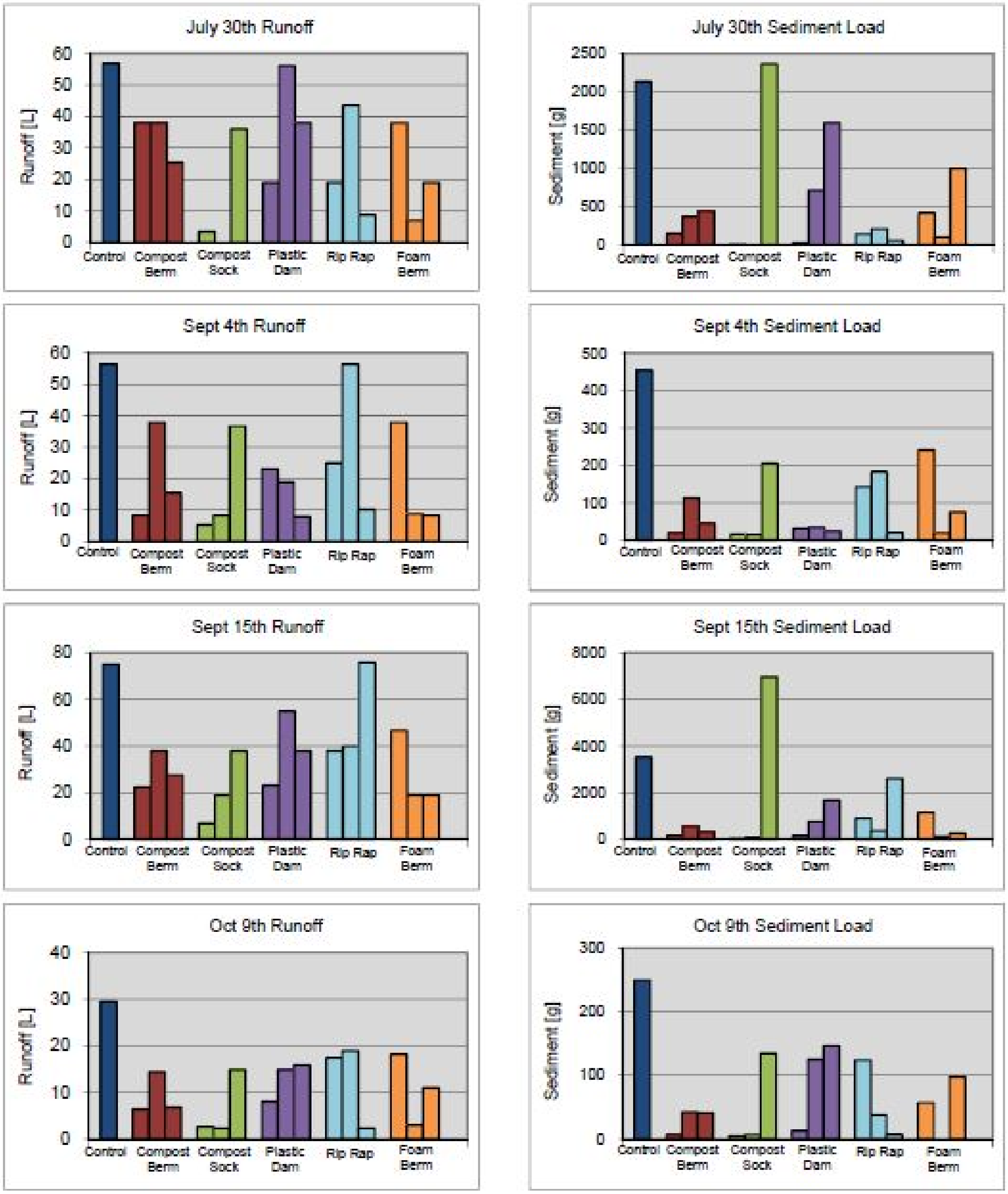


Figure 2.14: Field Scale Check Dam Results for Four Storm Events (Dalton 2011).

The compost sock installation was an arc shape concave upstream and anchored into the ground per the company's recommendations. This installation may have led to better stability and durability during high flows but also led to higher amounts of runoff and sediment as the arc shape would funnel or focus the flows creating more channel scour and higher concentration of flow to the middle of the check dam. Riprap berms also turned sheet flow into several streams of water between the rocks at the base of the check dam and thus concentrating the energy and volume of the original flow. Dalton notes that the rock size used in the berm is a major factor in this aspect. Dalton also notes that a geotextile fabric is required under the rock check to prevent erosion from flow through the check dam. The triangular foam berm was highly unstable due to its light weight for the high flow conditions and is recommended for low slope, low flow conditions. This outcome led to some of the variability observed in the data. The plastic grid dam was hard to qualify because it was installed on top of a compost blanket.

The testing performed by Dalton on the check dams allowed for direct comparison between check dam applications. Neither slope for the small scale or between rain events for the large scale testing created consistent results. Actual flow data for the large scale tests were not reported. It should also be noted that these tests were all performed on smooth slopes and were used to intercept sheet flow much like a slope interrupter device and not tested in channelized applications for use as ditch checks.

Cleveland and Fashokun (2006) discuss a roadside ditch monitored along a 2.3 mile stretch of NASA Road 1 in Harris County, TX. A study was performed to monitor the effect of a temporary rock-filter dam as a pollution prevention device for stormwater runoff. Two sites within the ditch were monitored; the first directly upstream of the rock-filter dam and the second 300 ft downstream of the rock-filter dam. These locations were monitored preconstruction, during construction and post construction. Turbidity, total suspended solids (TSS), total solids (TS), nutrients (NO3, NO2, NH4, and PO4) and soil particle gradations were measured during these samples.

These samples were used to compare differences between preconstruction, during construction and post construction using the mean values. During construction mean values were also compared upstream and downstream of the rock-filter dam. Samples were taken during storm which were classified as first-flush samples. TSS increased 477% from preconstruction conditions to during construction conditions upstream of the ditch checks. TSS increased further downstream of the ditch check by 15% when compared to upstream of the ditch check which leads one to believe that the rock ditch checks did not perform as a sediment control measure.

The preconstruction TSS levels returned during the post construction phase once vegetative stabilization occurred. This study concluded that the rock-filter dam failed as a pollution prevention device for controlling TSS. Large amounts of sediment were noted to have been deposited behind the dam, however, per the SWPPP, the contractor cleaned out the sediment once reaching capacity and these volumes were not recorded for analysis. The authors note that future research should evaluate retention volumes to estimate solids removed by the sediment control device.

Kang et al. (2013) compared different check dams (a.k.a, a ditch check) for use as sediment control devices. An excelsior wood fiber wattle, a rock check dam, and a rock check dam wrapped with an excelsior wood fiber erosion control blanket were tested using large-scale testing. Each check dam was reduced in length to fit in a 0.9 m wide and 0.9 m deep rectangular channel. This channel is 24 m long and ranges from 5 to 7% longitudinal slope and is lined with plastic sheeting. Each test used three of the same ditch checks in series with the spacing varying due to the height of the ditch check and slope of the channel. Three different test conditions were performed: (1) no PAM added to the devices with loam sediment added to test flow, (2) 60 g of PAM per device with loam sediment, and (3) 60 g of PAM per device with clay loam. After installation, three consecutive storm events were simulated. Each storm was a duration of 20 minutes consisting of four minutes of flow at 14, 28, 57, 28, and 14 L/s with a total of 242 kg of sediment was added and test results were averaged. When tested using no PAM, the rock check dam performed significantly different (for $p = 0.05$) by having significantly higher turbidity when compared to the rock with excelsior blanket and the excelsior wattle. The turbidity for the rock check dam increased 300% by the third storm, whereas the excelsior wattle and rock with excelsior blanket increased between 122 to 144%.

When comparing each storm event, all three were not significantly different for the first storm event, however for the subsequent storms, the turbidity for the rock ditch check was significantly higher than the other two devices. After applying PAM treatments, it was determined that effluent turbidity measured at the channel outlet was reduced by 78% to 93% for all check dams. The rock check dams lost flocculation potential faster than the other two devices that used excelsior. The excelsior creates a surface that PAM can attach to once wetted allowing the flow to be treated as it comes in contact with the PAM. The results of this study were based upon sediment control using a flocculation aid. Sedimentation deposition was also measured based upon the greatest depth of sediment deposit and length of deposition. A sediment deposition index (SDI) was developed by multiplying depth and length of the sediment deposition area. Therefore, SDIs with greater numbers created larger deposition areas and volumes. The total SDI for each ditch check was 1.16 for excelsior wattle, 1.07 for rock with excelsior blanket wrap, and 0.34 for rock check dam. This shows the excelsior blanket also promoted a greater deposition area when compared to the rock check dam without the excelsior blanket.

2.4 ALDOT Traditional ALDOT Sediment Control Ditch Check Practices

ALDOT Special Drawing ESC-300 (Sheet 1 of 7) specifies seven ditch check practices which can be used on ALDOT construction sites. Sand bag ditch checks are to be used for concrete or rock bottom channels, and six ditch checks that are recommended for bare earth channels: (1) hay bales, (2) silt fence, (3) wattles, (4) silt dikes, (5) rock and (6) rock with sump excavations.

2.4.1 Sand Bag Ditch Checks

Sand bag ditch checks are typically reserved for channel applications with hard channel bottoms such as concrete or rock. This is because sand bags ditch checks do not require stakes or posts to be driven into the ground to secure the devices in place. However, ALDOT has used these devices on earthen channel bottoms as well as shown in Figure 2.15.

(a) downstream view (b) post storm event

Figure 2.15: Sand Bag Ditch Check on ALDOT Project in Franklin County, AL.

2.4.2 Hay Bale Ditch Checks

Hay bales are an agricultural product that has been adopted by the construction industry for use as flow interceptor devices. Hay bales are to be used to intercept low volume flows in low to moderate grade ditches Abutting hay bales tightly together in two rows slows flow velocities in an attempt to reduce the erosive, scouring effects of concentrated and channelized flows. Hay bales can be secured with wooden stakes within a ditch or swale as a means to intercept and pond stormwater runoff. These bales are to be keyed or embedded into the soil a minimum of 4 inches to minimize undercutting the ditch check. Figure 2.16 shows the typical installation of hay bale ditch checks.

(a) typical hay bale installation on bare soil

(b) typical hay bale installation in veg. channel

Figure 2.16: Typical Hay bale Ditch Check Installations.

2.4.3 Silt Fence Ditch Check

The primary function of silt fence is to act as a temporary stormwater retention basin, reducing flow velocity, causing deposition of suspended sediment behind the structure (Jiang et al. 1997). ALDOT requires silt fence be installed on construction sites as required by the project plans. Silt fence consists of a geo-textile filter fabric that meets the requirements of AASHTO M288, supported by t-posts and wire backing placed in a way as to control sheet flow from disturbed sites. Its purpose is to retain sediment from small areas by providing detention time that allows for the deposition of suspended particles (Smolen et al., 1998; EPA 600/R-04/184, 2). Silt fence can be used as sediment barriers, ditch checks, or inlet protection devices on ALDOT projects. A silt fence used as a ditch check is a temporary dam constructed across a swale or drainage ditch to reduce the velocity of storm water runoff. Figure 2.17 shows improperly and properly maintained silt fences used in ditch check applications.


(a) improperly maintained silt fence ditch check


(b) properly maintained silt fence ditch check

Figure 2.17: ALDOT Specified Silt Fence Ditch Check.

2.4.4 Wattle Ditch Checks

Wattle ditch checks are appropriate for reducing velocity of stormwater runoff and controlling of sediment transport under low to medium flow conditions. Wattles are used as ditch checks, sheet flow interceptors, and inlet protections devices to control sediment. These devices are coveted by today's industry because they are biodegradable, they are easy to install and have a comparable or lower cost than traditional ditch checks. McLaughlin et al. (2009) reports a cost of \$1.70 per linear ft of construction when using wattle check dams as compared to a cost of \$1.98 per linear ft when using rock check dams with sump. Maintenance costs of rock ditch checks with sumps were estimated at over \$400 per maintenance action as compared to less than \$80 per maintenance action with wattles however these estimates are only based on the one study performed by McLaughlin et al. (2009). Figure 2.18 shows two wattle ditch check installations, one in a wide bottom ditch and the second in a narrow bottom ditch.

(a) wattle ditch check in a wide bottom ditch

(b) wattle ditch checks in series in a shallow ditch

Figure 2.18: Wattle Ditch Check Applications.

2.4.5 Silt Dike Ditch Check

Silt dikes are to be used in ditches with concentrated flows within the clear zone where riprap cannot be used. A silt dike can be used in ditch check applications to intercept and pond concentrated flow to allow sediment deposition. These devices are typically constructed of triangular foam wrapped in a geotextile filter fabric. An apron is attached to the bottom of the silt dike and draped upstream and downstream for anchoring purposes while preventing undercutting. Figure 2.19 shows two applications of silt dikes. Figure 2.19(a) is used in a bare earth swale where as Figure 2.19(b) shows a silt dike installation in a narrow channel acting as protection for a culvert.

(a) silt dike in a bare earthen swale

(b) silt dike ditch check protecting culvert

Figure 2.19: Silt Dike Ditch Check.

2.4.6 Riprap (Rock) Ditch Check

Rock ditch checks are designed based on expected flows and velocities which dictates the types and size of rock to be used. Sediment trapping effectiveness may be adjusted by choking with fabric or rock placed on the upstream side of the ditch check. The inclusion of an excavated sump upstream of a rock ditch check is to assure on-site sediment trapping of eroded soils in critical areas (e.g., cuts and fills) where erosion is expected. ALDOT has allowed rock ditch checks in conjunction with excavated sumps to be used as a sediment control applications in concentrated runoff applications. Where sediment retention is required, a drainage sump can be constructed below the ditch bottom elevation at discharge points allowing adequate detention time for suspended solids to settle out of the stormwater runoff and deposit within the sump itself. Several rock ditch checks with sumps may be used in series to increase overall on-site sediment trapping efficiencies. Figure 2.20 shows these applications in the field.

(a) rock ditch check ponding flowing runoff

(b) rock ditch checks detaining water


(c) rock ditch checks retaining sediment

Figure 2.20: Rock Ditch Checks.

2.5 Ditch Check Application Guidance and Recommendations

Limited specific recommendations for applications of ditch checks were found in the literature review. Specifically, recommended guidance for flow conditions applied to ditch check selection were not found through peer reviewed studies. Therefore, SHA's standard drawings and specifications applicable to ditch check selection, implementation, and maintenance were investigated to determine common practices typically used in the field.

2.5.1 Alabama Handbook

The Alabama Handbook for Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas (Alabama Handbook) was developed by the State of Alabama Soil and Water Conservation Committee and is referenced by both the Alabama Department of Environmental Management (ADEM) and ALDOT. The Alabama Handbook defines check dams as a small barrier or dam constructed across a swale, drainage ditch or other area of concentrated flow for the purpose of reducing channel erosion by flattening the gradient of the flow channel and slow the velocity of the channel flow. It also specifies that, contrary to popular belief, most check dams trap insignificant volumes of sediment. The

Handbook lists rock, logs, hay bales, or other suitable materials which include manufactured products for use as a check dam. No specific flow rate guidance was listed for any check dams, however rock and log check dams were specified appropriate for a drainage area of ten acres or less. Spacing was recommended so that the elevation of the toe of the upstream dam is at or below the elevation of the downstream dam.

2.5.2 California Department of Transportation

The California Department of Transportation (CalTRANS) developed a BMP database to be used as guidance for specifying different erosion and sediment control practices. Brush check dams are described as small incised gullies and slope drains that are stabilized with onsite brush material. This approach places brush dams in gullies and channels to intercept flow and cause sediment to deposit upstream of the brush. This is recommended for up to 6.5 ft and 16 ft wide ditches and is recommended for use in series to treat runoff. Spacing is recommended by running a level line from the top of the downstream ditch check to the bottom of the gully upslope to locate the next upstream ditch check. The drainage area constraint for this practice is limited to 2.5 acres.

The database also included the use of wattles for slope intercepts and mini ditch checks. There were no specific guidance for these products except to include that correct installation is critical to meet the effectiveness criteria and to be used for maximum steepness of 1H:1V.

2.5.3 Georgia Erosion and Sediment Control Manual

The Georgia Department of Natural Resources has created the Georgia Erosion and Sediment Control Manual. This manual specifies check dams as small, temporary barrier, grade control structure, or dam constructed across a swale, drainage ditch or areas of concentrated flow. The purpose is to minimize the erosion rate by reducing the velocity of stormwater in areas of concentrated flow. The manual specifies stone or hay bales as check dams. The stone check dams are for drainage areas no greater than two acres and the hay bale check dams no greater than 1 acre. No specific flow rate was provided for either type of check dam.

2.5.4 North Carolina Department of Transportation

NCDOT specifies riprap or wattles be used as ditch checks. The primary purpose of each is to reduce erosion in a drainage ditch by restricting the velocity of flow in the channel. The riprap ditch checks are specified to be used for drainage areas of 0.5 to 1 acres of drainage. No specific drainage guidance was provided for wattles. NCDOT has details for incorporating PAM to increase the overall sediment control capabilities of the ditch check. No specified flow rate for either ditch was provided.

2.6 Review Summary

Limited research has been conducted on controlled, large-scale testing of ditch checks in channelized applications. No literature was found that describes how the installation of a ditch check can affect overall performance of the practices. Specific guidance was also lacking as to optimal hydrologic conditions for each device. Where guidance was given, variations were evident as flows drainage areas varied for riprap from 0.5 to 10 acres. As the move from riprap and other traditional ditch checks to temporary manufactured ditch checks is becoming more prevalent, determining specific guidance regarding the applicable drainage area, flow rate, and for when to use various ditch check practices properly is necessary. Also, optimizing the installation of a ditch check may maximize the ability for a ditch check to reduce channel erosion and create favorable conditions for sediment deposition to occur within the channel and is crucial for determining applicability. As McLaughlin et al. (2009) noted, when ditch checks effectively impound stormwater, the energy from the runoff is transferred from the potential energy stored up by the impounded stormwater to kinetic energy within the channel directly downstream of the ditch check as flow overtops, flows through, or undercuts the device. Therefore the channel must be adequately protected at the ditch check - channel interface to reduce the risk of creating a secondary source of sediment as a result of downstream channel erosion after the ditch check.

Researchers at the AU-ESCTF evaluated the overall effectiveness of standard ditch check practices and whether these practices can be modified to improve performance. This dissertation examines the effects of various ditch check practices, installation configurations, and performance evaluations on wattle, sand bag, silt fence, and riprap ditch checks. The purpose was to enhance performance, develop flow criteria, and develop performance criteria. A testing methodology was developed to accomplish this task and is described in Chapter 3.

CHAPTER THREE

AU-ESCTF DESIGN AND CONSTRUCTION

3.1 Introduction

Many methods have been used to evaluate the performance of ditch checks and other filtering/impounding devices under large-scale conditions. The standard test method ASTM D 7208-06 is used by third party testing facilities to evaluate ditch check products. However, other testing facilities that are associated with specific state highway agencies (SHAs) typically use testing methods geared more towards local or SHAs specific design needs. The goal of this facility is to test devices typically used on construction sites as ditch checks under large-scale conditions while maintaining control of several variables control (i.e., slope, channel geometry, flow rates, sediment load, channel erosion, installation practice, sampling practices) and without creating an environment unrepresentative of field-like conditions. The purpose of this chapter is to describe the design efforts of the Auburn University Erosion and Sediment Control Testing Facility (AU-ESCTF) with a focus to evaluate the performance of erosion and sediment control practices typically used in highway construction. Chapter 4 will describe the overall construction effort.

3.2 Test Facility Location

Finding a suitable location with field-like conditions to accommodate such a facility requires, among others, three necessary attributes: (1) a renewable water supply, (2) topography indicative of road side conditions, and (3) the necessary land area to encompass all facility needs. The National Center for Asphalt Technology (NCAT) Test Track located in Opelika, AL contains an area with similar characteristics to the aforementioned attributes. The NCAT Test Track, shown in Figure 3.1, is a 1.7 mile oval test track on a 300 acre plot of land dedicated to accelerated pavement performance testing. The NCAT Test Track facility, therefore can provide an adequate amount of land with a large enough drainage area to aid in developing the AU-ESCTF. The design of the test track provides topography that mimics highway roadside conditions needed for the AU-ESCTF while also encompassing land for possible future expansion.

Figure 3.1: Preconstruction Aerial View of the Facility Location (Google Earth).

The location of the AU-ESCTF is a 2¼ acre site outlined in orange in Figure 3.1 which is encompassed by two connecting access roads along with a 5.8 acre asphalt parking lot up gradient and east of the AU-ESCTF boundaries. This area of land was originally intended as a reach to drain runoff from the impervious asphalt parking lot. This site fulfilled two important needs for the facility: (1) it made available a renewable water source by collecting runoff from the parking lot and (2) provided topographical characteristics similar to highway roadside drainage areas.

3.3 Facility Design

With the first two conditions met, the site location needed to be further evaluated to determine if the remaining needs of the facility could be met. Therefore, all the facets of a functional facility had to be identified. These additional needs included: (1) a way to collect and store stormwater runoff from the parking lot for experimental use, (2) the ability to hydrologically manage stormwater runoff not collected for storage, (3) identifying an appropriate area to locate three experimental channels, (4) provide control and treatment of sediment-laden runoff used in experiments, and (5) hydrologically designing the facility while not adversely affecting the downstream watershed.

3.3.1 Topographical Survey

The first step in evaluating the suitability of the site was determining the topography and area of the proposed facility location. A real time kinematic (RTK) global positioning system was used to perform the site survey. The area was surveyed from the upstream watershed through the proposed facility location to the discharge point of the watershed. Figure 3.2 shows an aerial view of the preconstruction facility and Figure 3.3 shows the site survey as rendered by Micorstation™ v8 XM.

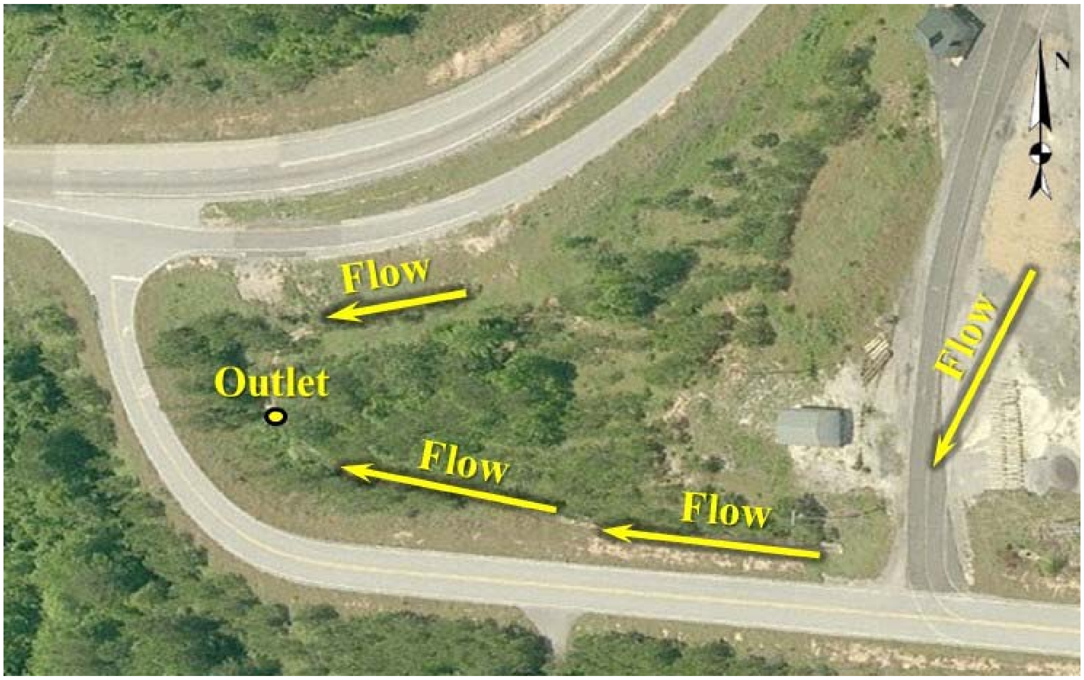


Figure 3.2: Aerial View of Proposed Site Conditions (Google Earth).

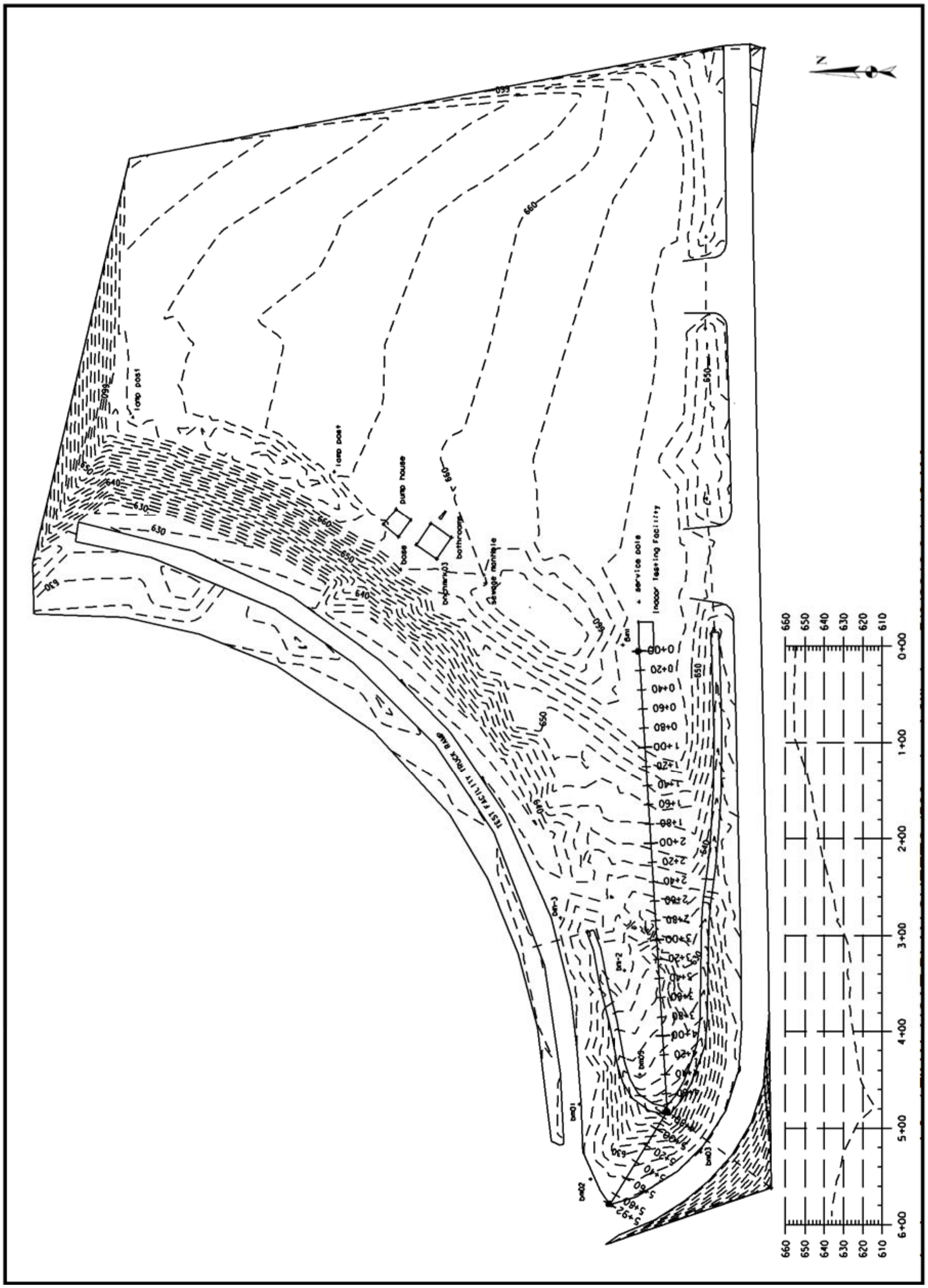


Figure 3.3: Topographical Rendering of the Proposed Site's Watershed.

Figure 3.3 suggests that rainfall runoff from the parking lot will flow to the main concrete ditch and be conveyed parallel to the proposed facility location to the outlet culvert. Physical observations of the runoff conditions were investigated during a storm event. It was determined that a large portion of the runoff from the parking lot does discharge through the proposed facility location. Figure 3.4 shows a closer view of the area's preconstruction topography. The preconstruction elevation difference from the top of the hill to the bottom culvert was about forty-one feet. The length of this elevation change was about 380 feet resulting in an average slope of 10.8%.

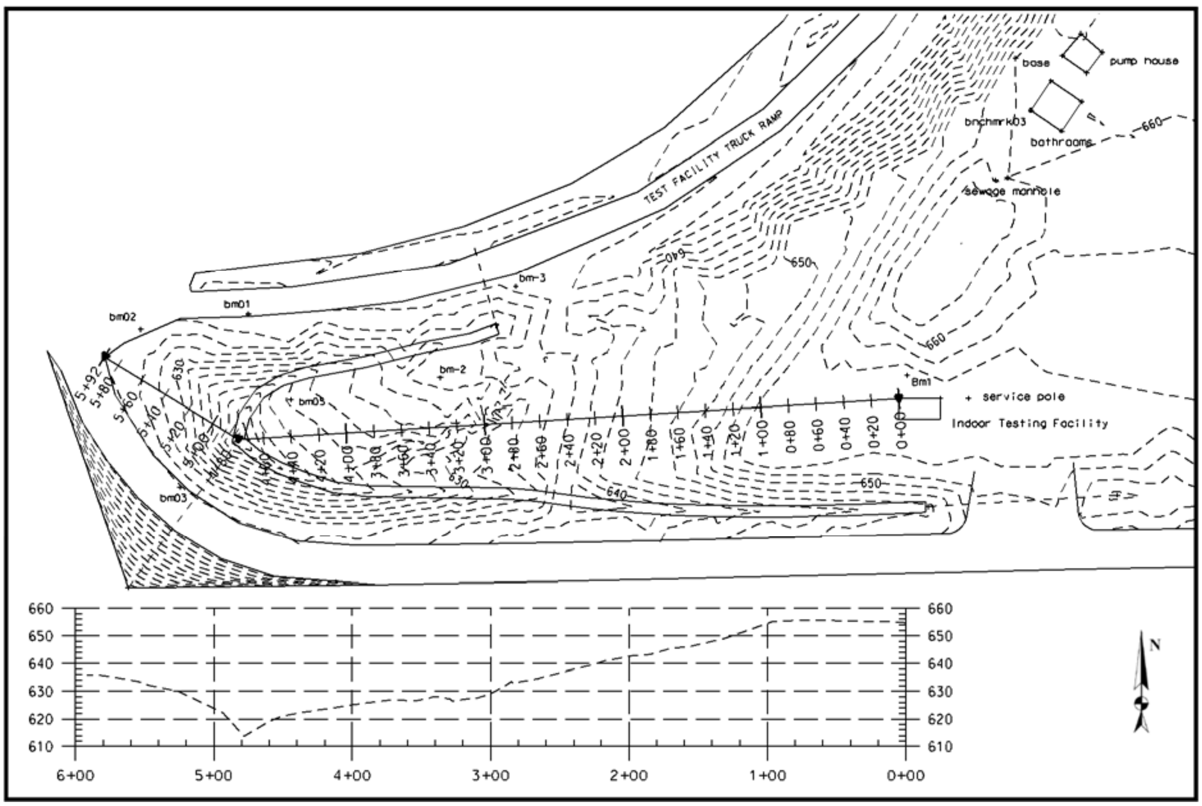


Figure 3.4: Topographical and Profile View of the Proposed ESC Facility Location.

3.3.2 Facility Design

The design of the facility included three water storage structures: (1) an upper supply pond, (2) a sediment basin, and (3) lower retention pond. An upper supply pond is needed to intercept, collect, and store runoff emanating from the parking lot for use during channelized flow testing efforts. A sediment basin would be needed to collect all sediment-laden water used in testing to allow for detention time and sedimentation. Since the overall characteristics of the area would be modified to suit the needs of the facility, a lower retention pond was required to retain water for use as an auxiliary water supply as well as detain stormwater to match or better preconstruction hydrologic conditions. Riprap lined channels would be used in place of the original concrete lined channels as means to dissipate energy while conveying stormwater runoff as previously designed. Figure 3.5 shows the final topographical design of the facility and Figure 3.6 shows a three dimensional rendering of the facility.

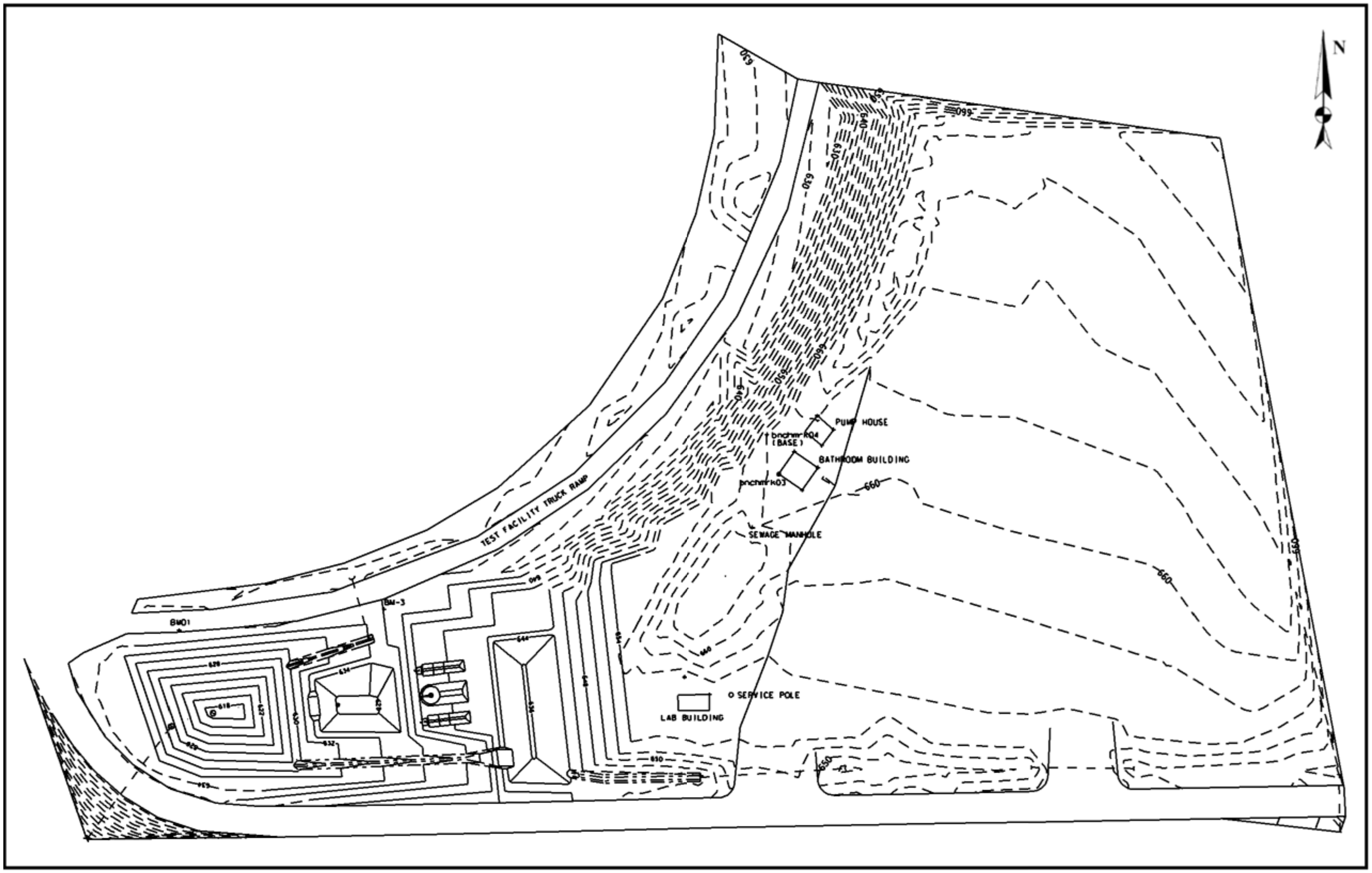


Figure 3.5: Topographical Plan View Rendering of the AU-ESCTF Design.

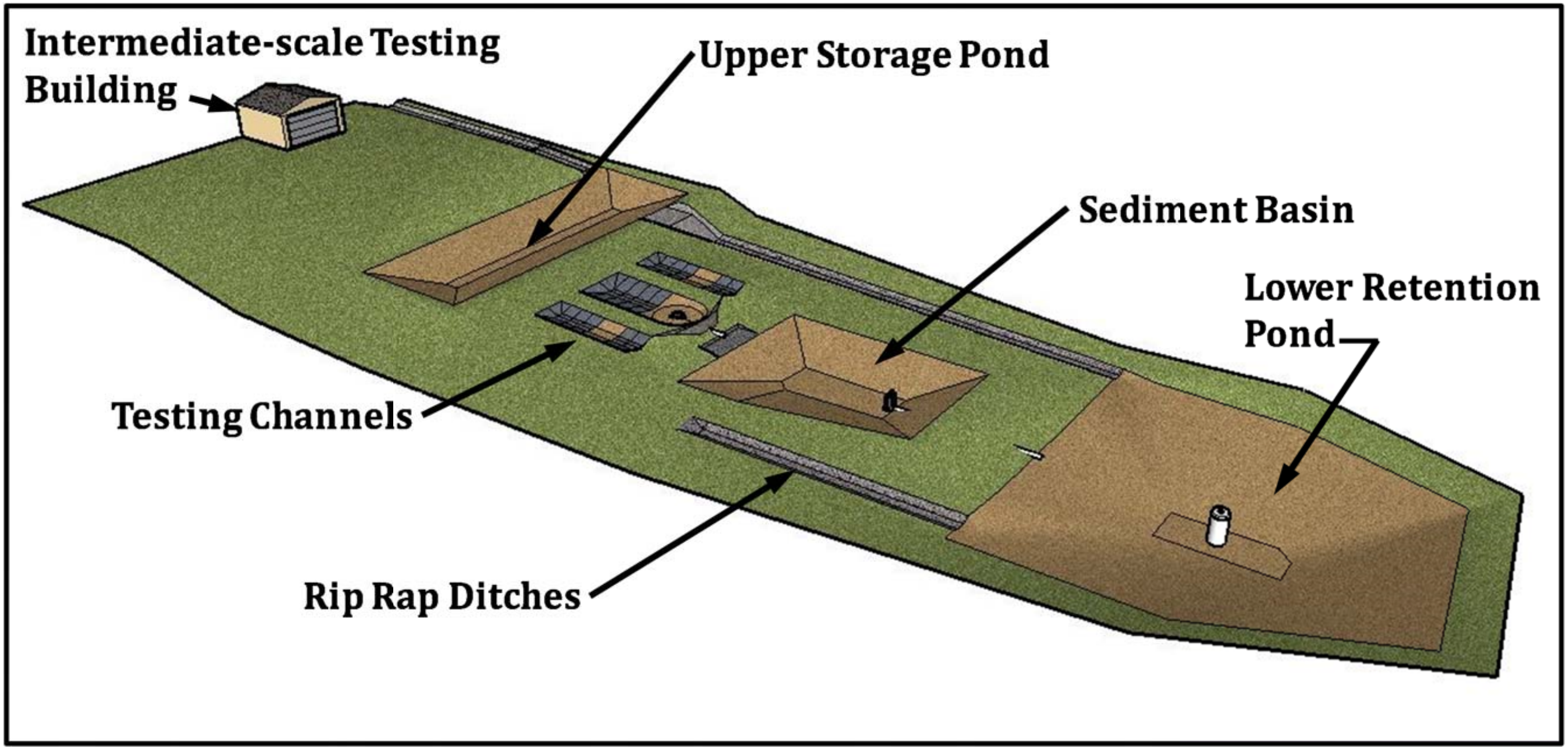


Figure 3.6: Three-Dimensional Rendering of AU-ESCTF Layout.

3.3.2.1 Facility Water Storage Structures

The upper supply pond was designed to retain enough water to use in three full ditch check tests per the water volume specified in ASTM Standard D 7208-06: *Standard Test Method for Determination of Temporary Ditch Check Performance in Protecting the Earthen Channels from Stormwater-Induced Erosion*. ASTM D 7208-06 requires 3 cubic feet per second (cfs) of sediment-laden water for a maximum 30 minute duration. Therefore, for three test runs of 30 minutes at 3 cfs, the pond is required to store at least 23,130 ft^3 of stormwater runoff. The actual pond storage capacity design is 28,000 ft^3. The depth of the pond is 6 ft. The bottom length and width is 90 ft and 6 ft respectively with 3:1 side slopes.

Once the storage pond reaches capacity, a 14 ft wide spillway at the head of a riprap lined channel will convey the remainder of the runoff to the lower retention pond. The retention pond's capacity was designed to hold 45,000 ft^3 of water. The outlet structure for the retention pond is a 48 in. diameter concrete riser pipe. This pipe is connected to the existing 24 in. reinforced concrete pipe (RCP) culvert. A geosynthetic clay liner (GCL) was installed in the retention pond to protect the test track from seepage forces of infiltrating water. The CETCO GCL liner Akwaseal was chosen because of its durability and low seepage rate. CETCO specifies a seepage rate of 40 gallons/acre/day for Akwaseal. The lower retention pond has a standing pool wetted area of 8,000 ft^2 (0.18 acres). A seepage rate of 7.2 gallons per day was estimated for the standing pool of the lower retention pond.

A sediment basin to properly control and detain sediment-laden water used during testing was also identified as a major need. A flashboard riser and a four inch Faircloth© skimmer were installed to promote detention time and sedimentation. The four and a half foot tall flashboard riser acts as the emergency spillway for the system. The basin relies primarily on the skimmer to discharge the water during normal hydrologic conditions. The skimmer is capable or discharging the stormwater in less than two days with no orifice cap. The water in the sediment basin will be discharged into the retention pond over a period of time dependent on an interchangeable orifice opening utilized with the skimmer design. The sediment basin is designed to hold at least 12,000 ft^3 of water. The typical recommended sediment basin for linear construction sites is 3:1 length to width ratio. However, the design for the AU-ESCTF sediment basin was approximately 75 ft by 60 ft length to width which results in a ratio of only 1.25:1. Initial designs included multiple basins which had shorter widths but maintained the 75 ft length, however this decreased the treatment capacity required for testing, therefore a basin that did not meet the recommended design parameters, but did meet the required volume requirements was chosen to meet the needs of the facility. It should be noted that the sediment basin will discharge into the lower retention pond which may act as a secondary basin to further treat the test water before discharging through the culvert.

During the initial design of the sediment basin and storage pond, Akwaseal was to be used as the pond liner. However, reclaimed asphalt pavement (RAP) was made available from the milling and repaving of multiple sections of the NCAT Test Track. It was decided that compacted RAP would be a more durable liner for equipment to run over during the maintenance of the sediment basin and storage pond. Please refer to Appendix A or this report for the complete set of final construction drawings and details.

3.3.3 Hydrologic Design

The design of the facility was heavily dependent upon its hydrologic performance. Three hydrologic programs were used throughout the design process to evaluate the hydrologic performance of the facility design which included: (1) the United States Department of

Agriculture's (USDA) Windows Technical Report-55 (Win TR-55), (2) HydroCAD, and (3) Bentley's PondPack^{TM}. Win TR-55 was used as a method to determine the runoff conditions of the facility during the initial design phases. HydroCAD was the first pond routing program used during the initial design process, however PondPack was used during the final design stage.

3.3.3.1 Stormwater Analysis

Win TR-55 was used to determine runoff conditions and hydrologic site characteristics. Many land use details were needed to properly evaluate runoff conditions associated with the site. These details included: (1) the facility as well as the overall upstream watershed acreage, (2) the pre and post construction ground cover conditions, (3) the pre and post construction topography, and (4) regional rainfall patterns. The total acreage, ground cover conditions, and topography were all determined using the topographical survey and design generated in MicroStation. Determining the regional rainfall patterns is a built-in function of the Win TR-55 program. Lee County, AL falls within the National Resource Conservation Service (NRCS) Type III hydrologic rainfall pattern. Figure 3.7 shows the storm data for Lee County from Win TR-55.

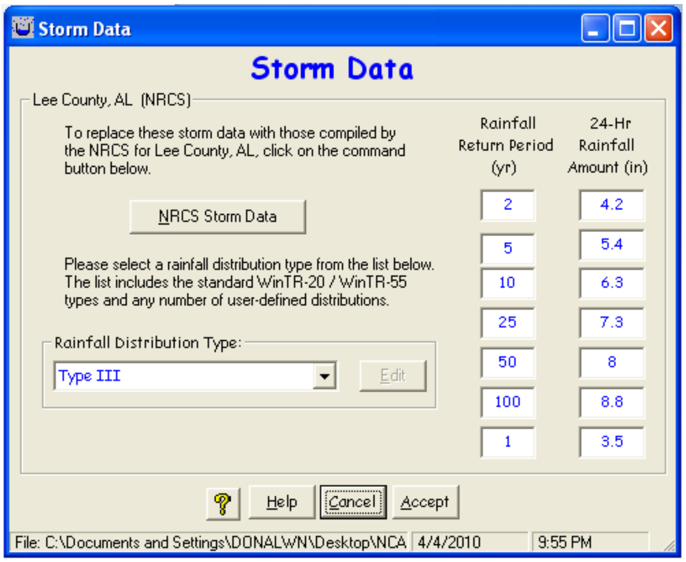


Figure 3.7: Win TR-55 Storm Data for Lee County, Alabama.

The preconstruction hydrologic analysis divided the AU-ESCTF watershed into three subareas as shown in the Win TR-55 screen shot in Figure 3.8. The parking lot (‘Parking Lo’) drains into the main concrete ditch (‘reach: CC-1’) which discharges to the outlet. It has the largest area of the three subareas at 7.7 acres and is comprised of about 5.8 acres of impervious area and 1.9 acres of brush which drains to ‘reach CC-1’. The sub-area which encompassed the facility, named ‘Grass/Tree’, was 1.28 acres of a woods-grass combination ground cover. The third subarea called ‘Median’ was about 0.64 acres with grass as well as impervious groundcover. Table 3.1 shows the preconstruction runoff characteristics determined in Win TR-55.

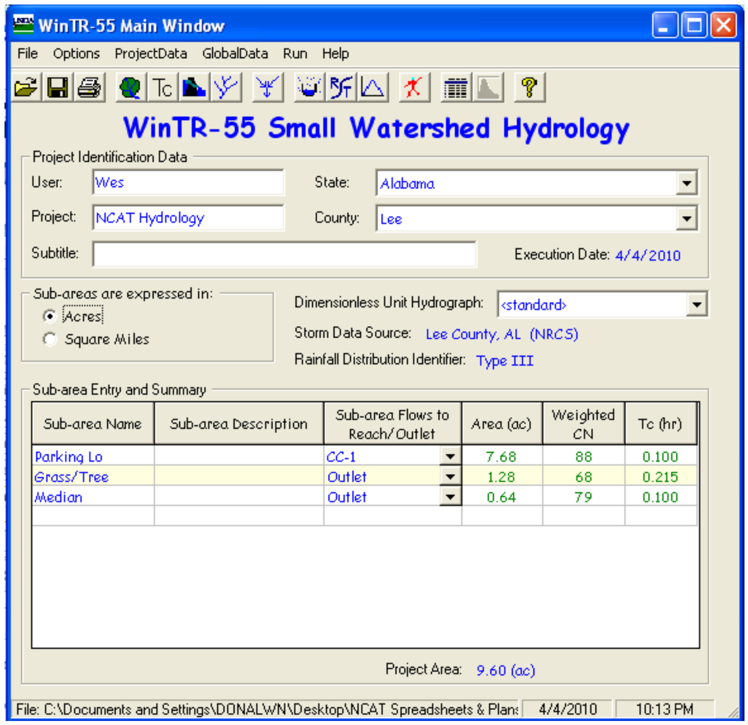


Figure 3.8: Win TR-55 Input of the ESC Facility Preconstruction Hydrologic Details.

Table 3.1: ESC Facility Preconstruction Runoff Characteristics

Preconstruction Hydrograph Peak Flow Rate (cfs)							
Subarea/Reach Name	**Design Storm**						
	1-yr	**2-yr**	**5-yr**	**10-yr**	**25-yr**	**50-yr**	**100-yr**
Parking Lo	15.89	20.46	28.33	34.23	40.76	45.33	50.51
Grass/Tree	1.01	1.57	2.64	3.50	4.52	5.24	6.08
Median	0.58	0.79	1.16	1.44	1.76	1.98	2.24
CC-1	15.89	20.46	28.33	34.23	40.76	45.33	50.51
Outlet (Total)	17.35	22.66	31.92	38.91	46.72	52.19	58.44

Once the design was complete, a post construction hydrological analysis was performed using Win TR-55. Subareas ‘Parking Lo’ and ‘Median’ were unchanged by the design. Subarea ‘Grass/Tree’ changed due to construction of the facility. However, due to the intricacies of the design, determining an accurate time of concentration (T_c) of the subarea was difficult. The preconstruction T_c was determined to be 0.215 hrs and the post construction runoff condition of this subarea was a T_c of 0.1 hrs. The main concrete lined channel (‘reach CC-1’) was replaced by two riprap lined channels named ‘Riprap1’ and ‘Riprap2’. Table 3.2 shows the post construction runoff characteristics.

Table 3.2: ESC Facility Post Construction Runoff Characteristics

Post Construction Hydrograph Peak Flow Rate (cfs)							
Subarea/Reach Name	**Design Storm**						
	1-yr	**2-yr**	**5-yr**	**10-yr**	**25-yr**	**50-yr**	**100-yr**
Parking Lo	15.89	20.46	34.23	34.23	40.76	45.33	50.51
Grass/Tree	1.20	1.84	3.05	4.03	4.16	5.96	6.90
Median	0.58	0.79	1.16	1.44	1.76	1.98	2.24
Riprap1	15.89	20.46	28.33	34.23	40.76	45.33	50.51
Riprap2	15.89	20.45	28.33	34.22	40.76	45.30	50.49
Outlet (Total)	17.66	23.07	32.53	39.68	47.65	53.25	59.62

3.3.3.2 Pond Routing and Retention Pond Design

Two pond routing programs were used during the design process, Hydrocad and PondPack. The upper storage pond and spillway are capable of conveying a 50-year storm event based upon the pond volume and surface area and based upon the spillway capacity. The efficiency and performance of the lower retention pond and outlet structure dictates level of protection provided to the adjacent road embankments. Therefore maintaining the pond height during storm conditions within the freeboard limits became the most critical design parameter. A 24 in. diameter RCP was the discharge outlet for the preconstruction condition of the facility. It was located at elevation 613.8 ft at the end of the two concrete ditches. This point became the control point for the lower retention pond design. The bottom of the lower retention pond in the original design was at elevation 614 ft. A 48 in. concrete stand pipe became the new outlet structure for the watershed by tying it into the 24 in. RCP. To limit the amount of earth movement during the construction of the facility, the original pond design would use the existing topography of the road bank as the pond side slopes. Under this condition the pond side slopes were approximately 2:1 and would allow the stand pipe to be tied directly into the RCP as shown in Figure 3.9.

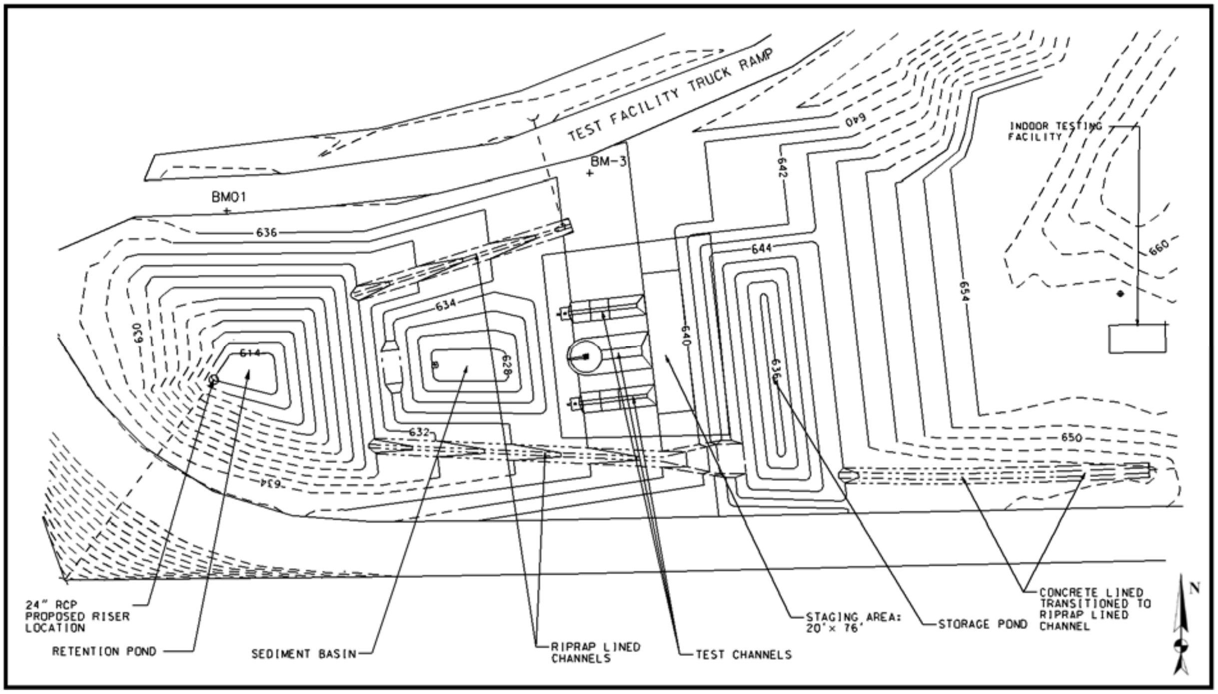


Figure 3.9: Original Retention Pond and ESC Facility Design.

A new pond design with 4:1 slopes for all pond sides was investigated due to the slope requirements of the Akwaseal pond liner as well as the grading equipment during construction. Changing the pond design required a completely new topographical design of the entire facility. Figure 3.10 shows the second facility design. This design required the bottom elevation of the retention pond to be raised four feet to 618 ft. The riser pipe was then moved from directly in front of the RCP to the center of the pond and away from the side slopes, which will allow equipment to easily navigate around it. Forty feet of 24 in. RCP was needed to tie the riser pipe into the existing discharge pipe.

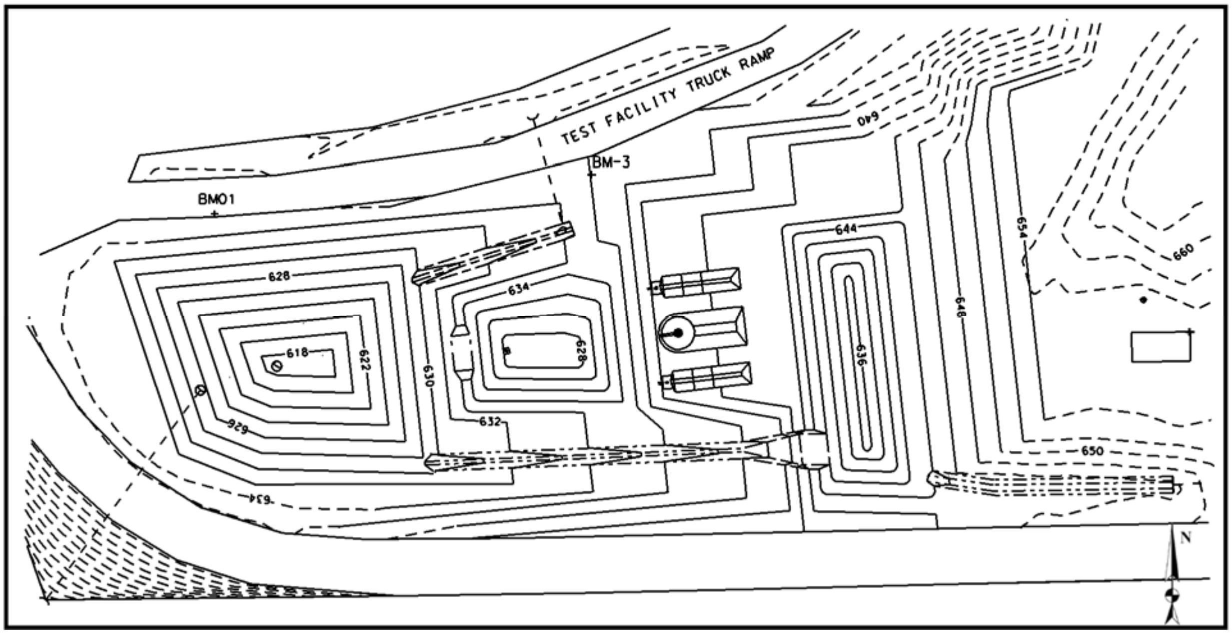


Figure 3.10: Final ESC Facility Topographical Design.

A third change to the overall grading plan was also performed. Upon analysis of the elevation of the outlet pipe connected to the drop inlet in the center testing channel, it was determined that the elevation of the testing channels needed to be raised. Therefore the staging area and the testing channels were raised four feet to the top elevation of the storage pond. This change reinforced the embankment that encompassed the storage pond but increased the fill dirt required for the facility.

These final designs were evaluated using PondPack to determine the performance of the lower retention pond. The 48 in. diameter riser pipe installed in the lower retention pond used two orifices. The first orifice was a 12 in. radial semicircle with a bottom crest located at elevation 625 ft. A 24 in. diameter orifice was also placed in the top of the riser pipe to act as an emergency spill-way for high rain events. The crest height of the pond (including freeboard) was set at 632 ft. A tail water condition would be created if the pond surface height reached the bottom elevation of the riprap channels at 630 ft. However, because the top of the GCL liner was installed at elevation 632 ft, two feet of freeboard would remain. The retention pond is designed to be able to pass a 100-year storm event before a tail water effect in the inlet channels is created. Refer to Table 3.3 for the lower retention pond routing information. When comparing the outflow to Table 3.2, the discharge through the culvert has decreased due to the capacity of the discharge pipe and the capability of the water to spread out over the surface area of the pond.

Table 3.3: Retention Pond Routing Information

Return Event (yr)	Q_{peak} Inflow (cfs)	Q_{peak} Outflow (cfs)	Pond Surface Elevation (ft)
1	21.84	15.74	627.96
2	27.3	19.05	628.31
5	36.83	22.62	628.85
10	44.09	25.11	629.31
25	52.24	27.72	629.99
50	57.98	29.95	630.26
100	64.56	31.53	630.68

3.3.4 Test Channels

Three test channels with the ability to test erosion and sediment controls in channelized conditions were designed as the central feature of the facility and the overall purpose of the design. Two identical channels dedicated to channelized erosion and sediment control performance and one channel dedicated to drop inlet protection devices were designed. Each channel has been designed to mimic roadside drainage ditch characteristics on typical ALDOT highway construction sites. ASTM D 7208-06 was referenced for the channel designs, however, design was adjusted to address regional design practices versus the ASTM standards design. No ASTM standard is available for performance testing of drop inlet protection; therefore the drop inlet was designed to similar features of a highway median drainage. Table 3.4 provides a tabular comparative breakdown of the geometry of the AU-ESCTF test channels versus the channels described in ASTM D 7208-06.

Table 3.4: Comparison of AU-ESCTF Channels vs. ASTM D 7208-06 Standard

Characteristic	AU-ESCTF Drop Inlet Test Channels	AU-ESCTF Ditch Check Test Channels	ASTM D 7208-06
Cross Section	Trapezoidal	Trapezoidal	Trapezoidal
Bottom Width	4 ft	4 ft	2 ft
Side Slopes	3.75H:1V	3H:1V	2H:1V
Length	45 ft	45 ft	60 ft
Slope	5 %	5 %	5 %
Channel Lining	Multi Section (42% Earthen | 58% Removable Sheet Metal)	Multi Section (22% Earthen | 78% Removable Sheet Metal)	Earthen

Construction drawings were developed for these channels as shown in Figure 3.11 and Figure 3.12. A 3-dimensional scale rendering of the channels were also developed to determine their placement functionality within the facility shown in Figure 3.13 and Figure 3.14.

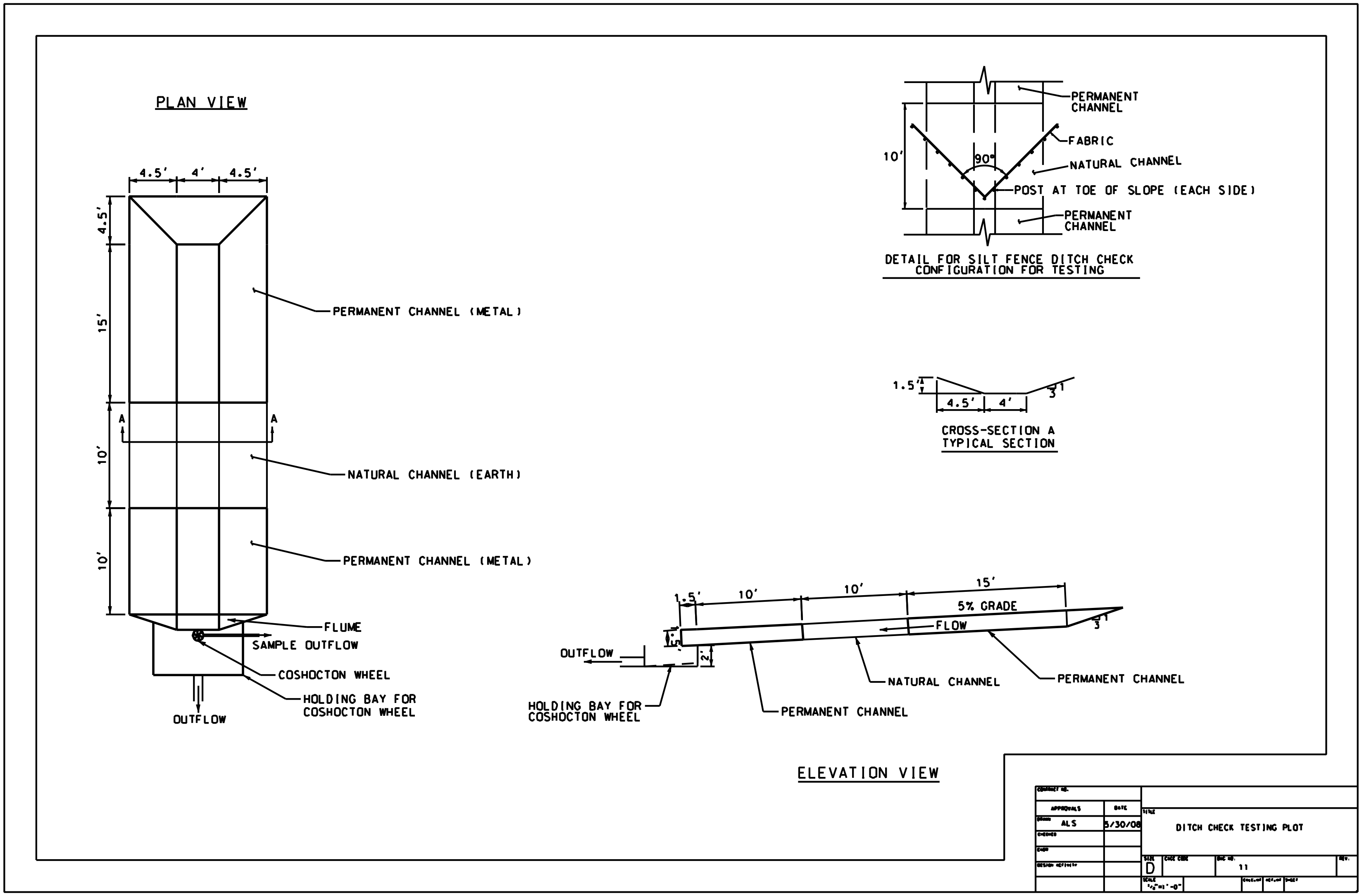


Figure 3.11: MicroStation Drawing of the Erosion and Sediment Control Test Channels.

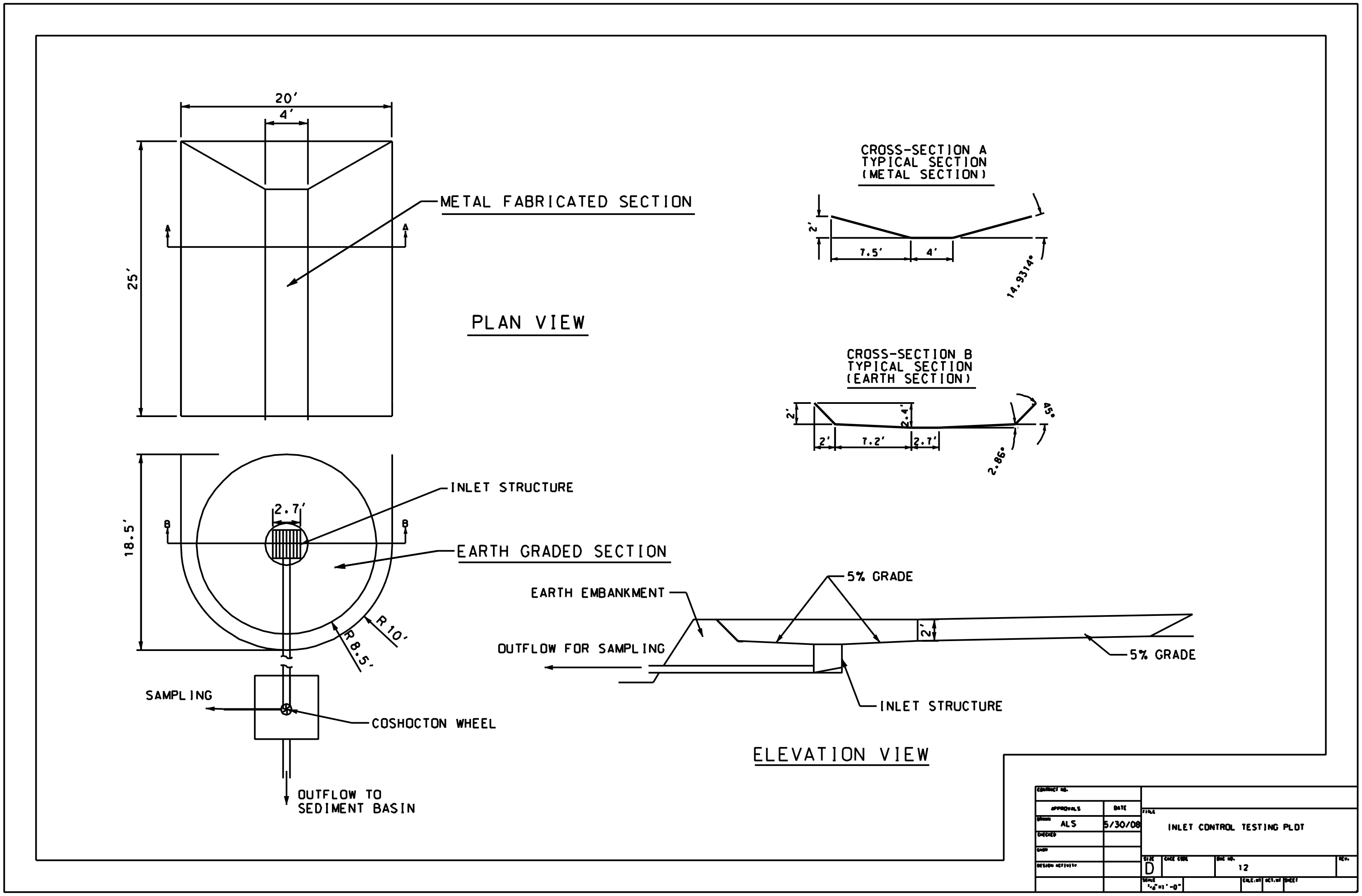


Figure 3.12: MicroStation Drawing of the Inlet Protection Device Test Channel.

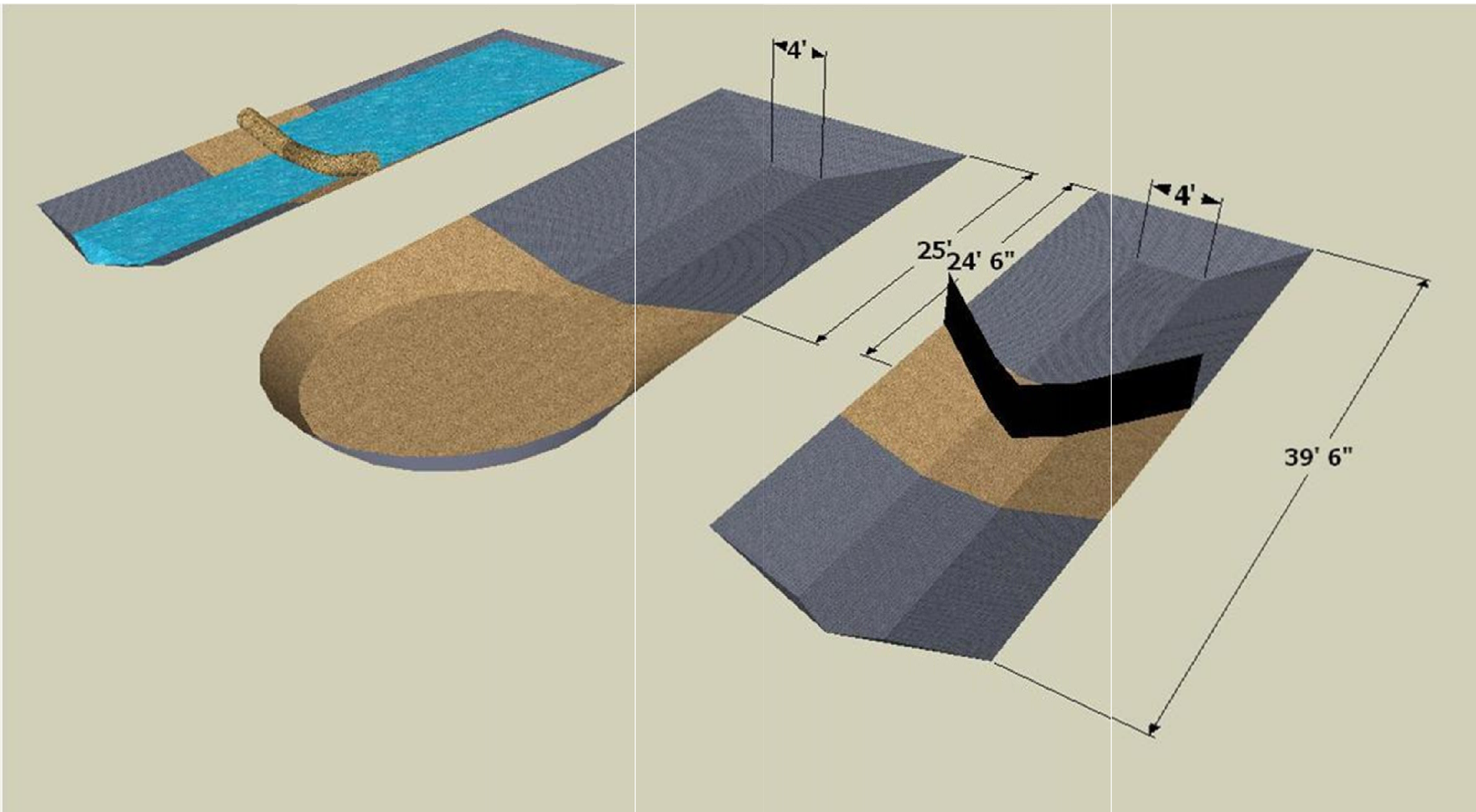


Figure 3.13: 3-Dimensional Rendering of the ESC Facility Test Channels.

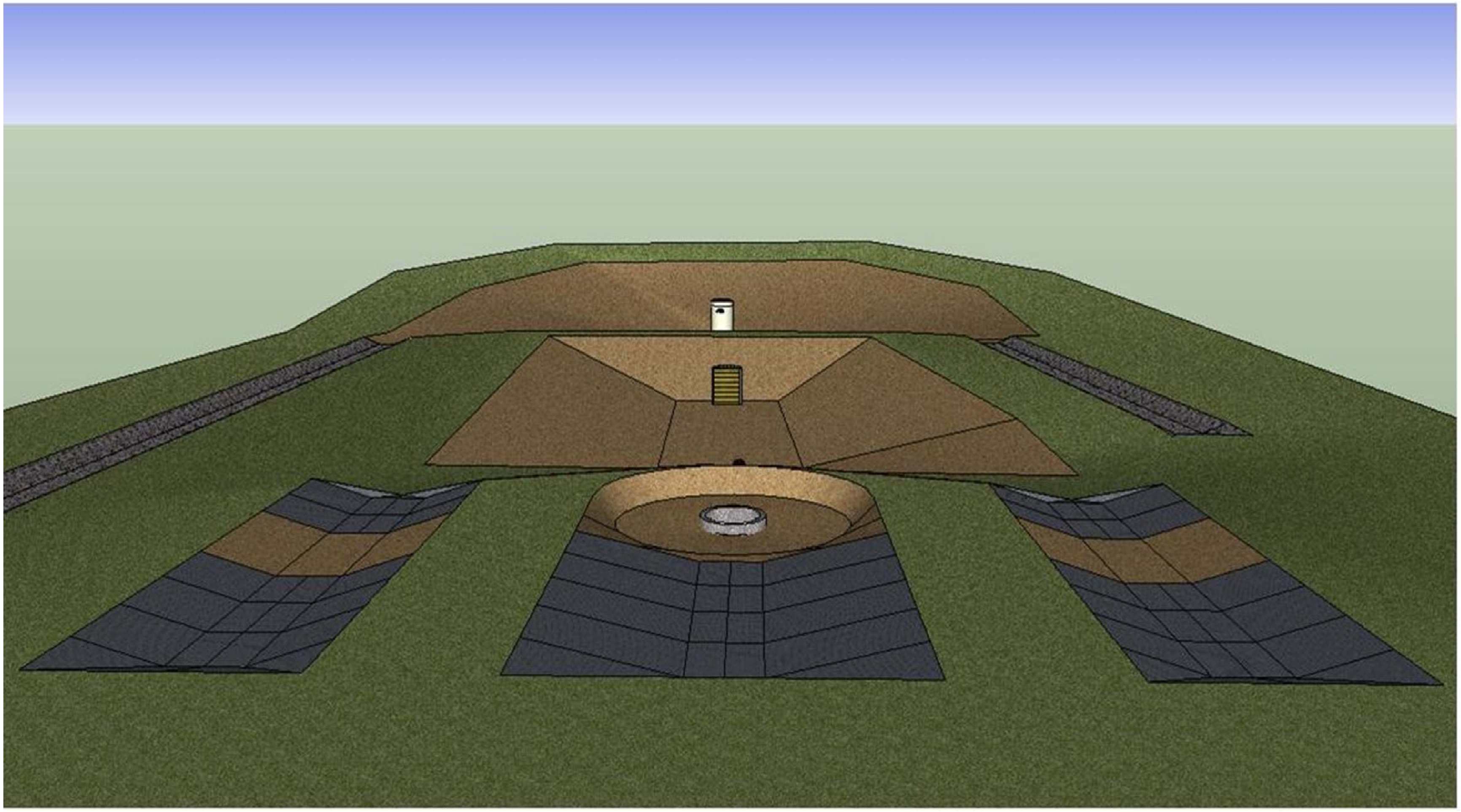


Figure 3.14: Isometric View of the AU-ESCTF Experimental Channels.

3.4 Summary

The overall design of the AU-ESCTF used several different design programs. The two graphical design programs used were MicroStation XM and Google Sketchup. MicroStation XM was used to develop construction drawings based on-site surveys performed at the proposed facility location. Sketchup was used to graphically enhance the visualization of the facility for educational purposes as well as developing a model that was used to check the different design aspects and layouts. This was necessary to determine overall effectiveness and applicability to the functionality of the facility. Win TR-55 was used in determining pre and post hydrologic conditions based on the changes to the facility drainage patterns and conveyance measures. This information was used to help develop simulations in both hydrologic pond routing programs PondPack and HydroCAD.

Base on the hydrologic performance of these models the design of the facility was changed and transformed to satisfy the hydrologic needs as well as the constructability of the facility. Redesigning some elements to improve the constructability and meet the capabilities of the construction crew became the main prerogative in many of the design changes made throughout the construction process. These changes made the construction efforts more manageable which allowed for a better product.

CHAPTER FOUR

CONSTRUCTION OF THE AU-ESCTF

4.1 Introduction

Construction on the AU-ESCTF began in January 2009. All construction activities were performed and monitored by the NCAT Test Track staff. The NCAT Test Track truck drivers were used as the construction workers and equipment operators since pavement testing had ceased to allow for track repaving. Blaine Guidry and Blake Lockhart oversaw construction operations on-site. There were three main construction phases: (1) clearing and grubbing, (2) rough grading and excavation, and (3) fine grading and installation.

4.2 Clearing and Grubbing

Before construction activities began, an erosion and sediment control plan was developed and implemented at the site. Sandbag ditch checks were placed in the concrete ditches to slow water velocities and promote sedimentation. A perforated, corrugated metal pipe wrapped with filter fabric and surrounded by riprap and 57 stone was installed to also aid in sedimentation as shown in Figure 4.1. The riser pipe was installed at the inlet of the concrete culvert which ran under the access road as shown in Figure 4.1. At the outlet end of the culvert, energy dissipaters and silt fencing were installed to prevent erosion and to control the sediment load leaving the construction area as shown in Figure 4.2. The main facets of the clearing and grubbing phase included the removal of all the trees on the facility and the demolition and removal of the concrete drainage ditches. Figure 4.3 shows preconstruction thru the clearing phase of the project.

(a) perforated riser pipe wrapped in filter fabric with a wire trash guard

(b) riprap placed around riser pipe for support during ponding periods

(c) final installation of ESCP temporary riser

(d) sand bag ditch checks

Figure 4.1: Installation of the ESC Plan Temporary Riser Pipe.

(a) perforated riser pipe installed over existing 24 inch reinforced concrete pipe

(b) outlet side of concrete culvert

Figure 4.2: Inlet and Outlet of Control Structures Used for the ESCP.

(a) preconstruction retention pond area

(b) preconstruction view of the AU-ESCTF from lower end of access road

(c) clearing and grubbing stage

(d) completion of clearing and grubbing

Figure 4.3: Clearing and Grubbing Stages of the AU-ESCTF.

4.3 Grading and Excavation

Most of the area designated for the facility was comprised of unsuitable soil that contained a high amount of organics (e.g. tree stumps, spoil, etc.) left over from the original construction of the NCAT Test Track in 2000. A cut-and-fill analysis for soil quantity was performed multiple times during the construction process as several design changes were required based on the complications encountered during construction. These included: (1) excavation of old stumps and other trash left over from the track excavation which resulted in loss of fill material, (2) change in position, elevation and geometry of the lower retention pond,

(3) repositioning of the middle sediment basin and upper storage ponds, and (4) redesign of the staging area for the experimental channels. The first design resulted in an excess of 6,000 yd^3. After many redesigns before and during the construction process, an estimated 300 yd^3 of soil was needed to fulfill all the grading requirements.

There were six elements of the facility that required fine grading: (1) the upper storage pond, (2) the sediment basin, (3) the lower retention pond, (4) the staging area for the experimental test channels, (5) the short riprap lined channel which collects runoff from one median of the test track, and (6) the two, riprap lined channels that collects and transports runoff from the asphalt parking lot to lower retention pond. Two riser pipes were also installed: (1) a concrete riser pipe for the lower retention pond and (2) a flash board riser for the sediment basin. The lower retention pond was lined with an Akwaseal™ geosynthetic clay liner (GCL). The sediment basin and the upper storage pond were lined with recycled asphalt pavement (RAP) due to availability, durability, decreased installation difficulty, and cost effectiveness.

4.3.1 Retention Pond

The lower retention pond was the first element to be completed. Six stages were needed to complete the retention pond: (1) initial grading, (2) installation of the 40 ft long, 24 inch RCP, (3) installation of the temporary riser pipe, (4) installation of the permanent 48 inch riser pipe, (5) installation of the GCL, and (6) final grading of the pond liner cover soil.

Fill dirt for the subgrade of the pond was taken from cut areas of the sediment basin and upper storage pond. While the cut and fill process was proceeding, the 40 ft long, 24 in. RCP was tied into the existing RCP culvert as shown in Figure 4.4.

(a) 24 inch RCP tied into existing culvert (b) relocation of the riser pipe

Figure 4.4: Installation of the Retention Ponds Riser and Outlet Pipe.

The temporary riser pipe originally installed over the existing culvert was temporarily removed during the installation of the 24 in. RCP and the bottom section of the permanent riser pipe. The permanent riser pipe was constructed of four, 48 in. diameter RCP sections. The bottom three sections were 4 ft in height apiece while the top section was 1 foot tall. The bottom section tied into the RCP and was buried by the subgrade material and the cover soil over the GCL pond liner. Once the subgrade was excavated to the required elevation, the temporary riser pipe was reinstalled over the bottom section of the permanent riser pipe as shown in Figure 4.5.

(a) temporary riser pipe relocation (b) ponding around temporary riser

Figure 4.5: Relocation of the Temporary Riser Pipe for Sediment Control.

The permanent riser pipe was installed once final grading and preparation of the lower retention pond was completed. Three more sections of the riser pipe were installed on top of the existing base. The first section, which was installed directly on top of the base, was a 4 ft tall section with no orifices. The next section was a 4 ft tall section with a 24 in diameter vertical orifice. The top crest of the vertical orifice is located 3 inches below the top of this section. A rebar trash rack was cast in place in the orifice. A 1 ft tall top section with a 24 inch diameter horizontal orifice with a rebar trash rack cast in place was installed on top of the third section. Upon evaluation of the riser pipe, two changes were made post installation. The lower half of the vertical orifice was concreted in to add additional storage within the pond. Also the permanent trash rack on the horizontal orifice located in the top section was cut off with a torch to allow maintenance access to the riser pipe. Figure 4.6 shows the final installation of the riser pipe.

(a) final location of the permanent riser (b) upper three sections of the riser

Figure 4.6: Permanent Riser Pipe for the AU-ESCTF Lower Retention Pond.

After the riser pipe was installed and the subgrade for the Akwaseal GCL was finished, installation of the pond liner began. The uppermost elevation of the liner location was staked out and a trench was dug along this elevation in order to key-in the liner. A 4 in. diameter, 20 ft long solid metal bar was used as the core bar to unload and roll out each roll of Akwaseal. Each roll weighed an estimated 1,500 lbs. A track excavator was rented during the construction process to excavate the ponds. Due to the weight of the rolls, the excavator was needed to pick up and hold the rolls while the construction crew unrolled the liner as shown in Figure 4.7. A metal pipe was mounted between the chains above the core bar to act as a spreader bar, spreading the chains apart to keep the GCL from chaffing against the chain while unrolling.

Figure 4.7: Pond Liner Installation Set Up.

Each roll of GCL had to be overlapped 1 ft with the liner adjacent to the section and with any liner intercepted. Granular bentonite was placed between each overlap to seal and seam the sections together. Once the liner was in-place and seamed together, 2 ft of cover soil was spread over top of the liner to protect the liner material and add constant compressive force to the bentonite. The constant compressive force was needed to densify the bentonite once wetted and allowed to swell. This spread soil was stockpiled around the upper perimeter of the trench. Because a large majority of the cut soil was deemed unsuitable as a fill material, outside material had to be trucked in. Figure 4.8 shows the progression of the liner installation.

(a) truck bed of Akwaseal rolls

(b) first set of installed liner

(c) liner rolled out after the first day of pond liner installation

(d) retention pond liner completely unrolled

(e) seaming together of liner using granular bentonite

(f) keying in of and final installation of the pond liner

(a) spreading of cover soil

(b) final grade of the retention pond

Figure 4.8: Progression of the Akwaseal GCL Installation.

Stabilization and erosion control of the retention pond banks became the next and final step of completing the retention pond construction. A silt fence barrier was installed up gradient of the retention pond to capture and detain all runoff emanating from the facility. The banks retention pond were treated with the following soil amendments: lime, fertilizer and polyacrylamide. After the soil was treated, the banks were seeded with one part Bermuda, and three parts Bahia grass seeds at a spread rate of 3 lbs total seed per 3,000 sf. Finally the banks were hydromulched with Geoskin donated from Mulch and Seed Innovations, LLC. Figure 4.9 shows these processes which occurred Friday March 13, 2009.

(a) broadcast seed spreader (b) seeding retention pond banks

(c) hydromulching pond banks (d) hydromulch application and silt fence

Figure 4.9: Initial Erosion Control and Slope Stabilization Measures on Retention Pond.

From March 14-March 16, 2009, 2.85 in. of rain was recorded for the Auburn-Opelika, AL area. During the first 24 hrs, 0.33 in. of precipitation occurred. The area encountered 2.52 in. of rain over the next 48 hrs with many problems developing at the facility. Because the hydromulch was not given ample period to dry, the majority was washed off the banks and into the pond during the more intense rain period. The stormwater runoff that ponded behind the silt fence undercut the center of the fence causing concentrated flow to wash out part of the soil cover of the pond liner resulting in a slope failure. Soil from the slope failure washed into the retention pond severely silting in the pond. Many rills were also formed as a result of stormwater emanating from the surrounding access road as well as the runoff emanating from the banks directly. Figure 4.10 shows the retention pond during the initial, lighter rainfall event as well as the subsequent more intense rainfall event.

(a) during the initial rain events: March 14, 2009

(b) during the heavier rain events: March 15, 2009

Figure 4.10: Initial Retention Pond Stabilization and Erosion Problems.

The retention pond was pumped dry to assess the overall condition of the pond and determine the best course of action to stabilize the banks. Skip Ragsdale from Sunshine Supplies in Birmingham, AL evaluated the facility's runoff conditions and erosion issues. Mr. Ragsdale made the following recommendations: (1) use erosion control blankets (ECB) to stabilize all disturbed areas around the pond, (2) reinstall the silt fence and reinforce the fence at the lowest elevation to prevent undercutting and structural failure, (3) install turf reinforcement mats (TRM) perpendicular to the silt fence at the lowest elevation to protect the cover soil from eroding and washing out, and (4) place a berm around the edge of the access road surrounding the retention pond to divert flows around the banks to other TRMs where flow can be safely conveyed into the retention pond. These new practices are shown in Figure 4.11. The entire area was reseeded as previously performed to prompt that long term vegetation establishment. Soon after these new erosion control practices were implemented, another large rain event occurred at the facility. These new practices greatly improved the soil stabilization around the retention pond. Figure 4.12 shows the retention pond and silt fence after the rain event.

(a) installation of new ECBs (b) installation of TRM

(c) RAP berm constructed to convey runoff (d) complete stabilization of pond banks

Figure 4.11: New Temporary Stabilization of the ESC Facility's Retention Pond.

(a) silt fence impounding facility runoff (b) TRM conveying impounded runoff

(c) rills forming below ECBs (d) ECBs after rain event

Figure 4.12: Retention Pond After Rain Event.

Figure 4.12 shows the retention pond at near half pool. This was due to the seam connecting the two middle sections of the riser not being sealed properly. The seam leaked for the first few months of operation until it was properly sealed. Upon the completion of the retention pond, the construction operation focused on completing the sediment basin, upper storage pond, riprap channels, and test channels area.

4.3.2 Sediment Basin

The sediment basin was the next major element of the facility to be constructed. The sediment basin was much less labor intensive since the GCL pond liner was no longer going to be installed in this pond. The majority of the area excavated for the sediment basin was an unearthed trash pile of trees and stumps from the clearing and grubbing phase of the original test track construction effort. Therefore, some of the material excavated from the upper storage pond had to be used as berm material for the sediment basin. Once the berms were built up, a trench was cut through the berm separating the sediment basin from the retention pond. A 12 in. diameter and a 4 in. diameter schedule 40 polyvinyl chloride (PVC) pipe were installed in the trench to connect the sediment basin discharge devices to the retention pond. A flash board riser was attached to the 12 in. pipe. A Faircloth skimmer would later be attached to the 4 in. pipe.

The flashboard riser was constructed of a 36 in. diameter corrugated metal pipe and has an installed height of 4.5 ft. Two columns of tongue-in-groove flashboards are dropped in place in the front of the riser. These boards can be removed as needed to control the water depth of the basin. The riser will be used as the emergency spillway for the basin as the skimmer will be the primary dewatering device. Once the riser and outlet pipes were installed, RAP was spread and compacted in the basin to act as the basin's liner. Figure 4.13 shows the construction progression of the sediment basin.

(a) flash board riser (b) 12 in. PVC outlet pipe

(c) rough grade of the sediment basin (d) final condition of the sediment basin

Figure 4.13: Construction Progression of the Sediment Basin.

4.3.3 Upper Storage Pond and Riprap Channels

The upper storage pond and riprap channels were the last two major construction efforts of the AU-ESCTF. The upper storage pond was excavated in phases throughout the construction process as fill soil was required to fill other construction needs. This pond was the most difficult to excavate due to the densely compacted clay. RAP was also used as the liner material for this pond. However, compacting the RAP was very difficult due to the oblong shape and steep bank on the inlet/outlet end of the pond. As an end result, the seepage rate for the upper storage pond is higher than the lower retention pond. This has been visually observed by comparing the water level decrease between the two ponds. The upper storage pond level noticeably decreases while the lower retention ponds pool height stays more consistent. Figure 4.14 shows the progression of the storage pond.

(a) initial excavation of the storage pond

(b) first half of the pond lined with RAP

(c) majority of the storage pond lined (d) completed lining of the storage pond

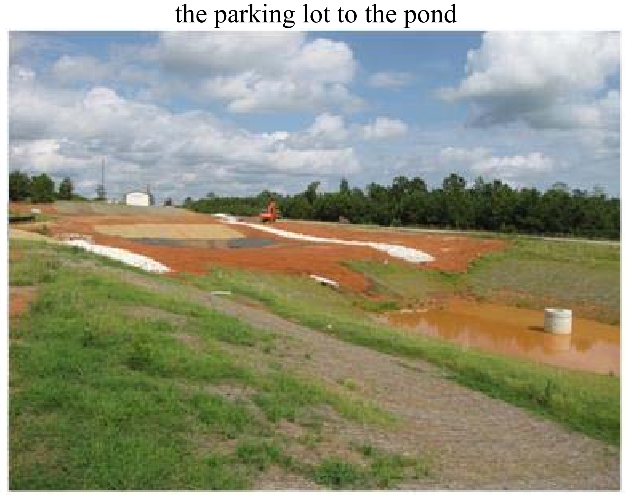
Figure 4.14: Construction Progression of the Storage Pond.

As shown in Figure 4.14 (d), once the storage pond was completed, the riprap lined channels were constructed and lined. The channel was excavated per the construction drawings located in Appendix A. The bottom widths of the channels are 3 ft wide with 1V:2.3H side slopes. This side slope was chosen to mimic the channel dimensions of the original concrete lined channels. Each channel was lined with a nonwoven filter fabric to protect bottom from scour. Four inches of ALDOT No. 4 crushed stone was used as the sub base for the channel. Finally, 2 ft of ALDOT Class 1 riprap was used to further line the channel. A third riprap lined channel was installed on the opposite side of the facility to replace the smaller concrete channel which drained the off-ramp and a portion of the median separating the test track and the off-ramp. Figure 4.15 displays the finished channels.

(a) location of the inlet and outlet channels

(b) inlet channel responsible for conveying runoff from the parking lot to the pond

(c) outlet channel responsible for conveying discharge to the retention pond



(d) overall view of the facility after installation of the drainage channels

Figure 4.15: Location of Riprap Lined Drainage Channels.

4.3.4 Test Channels Excavation and Construction

After the completion of the storage pond and riprap drainage ditches, the final step in completing the facility construction was completing the test channels. During the construction process, it was determined that a final change to the construction plans was needed proper installation of the drop inlet in the drop inlet protection test channel. A second load of fill soil had to be trucked in to fill this need. Once the grade of the test channels was met, a back hoe was used to excavate each channel to a rough grade condition. A conveyance channel as well as a sampling area was excavated out at the lower end of each ditch check test channel connecting the testing areas to the sediment basin. A 12 in. PVC pipe connected the drop inlet to the sediment basin. Figure 4.16 shows the rough excavation of the test channels. Figure 4.17 shows the rough excavation of the conveyance channel and sampling area of the right test channel.

(a) backhoe used to excavate test channels

(b) rough excavation of the left test channel

(c) rough excavation of the middle drop inlet testing channel

(d) rough excavation of the right test channel

Figure 4.16: Rough Excavation of the AU-ESCTF Experimental Test Channels.

(a) conveyance section of ditch check test channel

(b) sampling area of ditch check test channel

Figure 4.17: Excavation of the Conveyance and Sampling Area of the Right Channel.

The rough excavation and grading of the test channels concluded the last heavy machinery construction effort. The remaining fine grading of the channels used manual labor and the use of two walk-behind mini skid steer track-loaders. The first track-loader was rented from Rental, Inc. in Opelika, AL. This was used to evaluate the overall functionality of the machinery to determine its usefulness in furthering the construction efforts. A large majority of the right test channel, herein referred to as the prototype channel, was fine graded using this equipment. Purchase of a used Mini Skid Steer Bobcat Track Loader from RSC Rental Company was made once the utility of the equipment was determined.

The conveyance and sampling areas of the prototype channel were temporarily lined with 0.5 in. untreated plywood to evaluate the overall design of the prototype without permanently altering the channels. This material was chosen because it was the cheapest alternative for temporarily stabilize these sections. Once these areas were lined, the rest of the construction efforts emphasized completing the installation of the prototype channel. A 2 ft wide trapezoidal shaped wooden mold was built to help with final grading of the prototype channel as shown in Figure 4.18. The mini skid steer loader was used to excavate and properly shape the testing area of the test channel. The trapezoidal mold was loaded down with soil and dragged up the channel with the mini skid steer loader to perform final grading of the prototype channel. Shoveling, raking, and compaction with hand tamps encompassed the manual labor of the final grading process.

(a) trapezoidal mold used for fine grading of the channel

(b) fine grading to achieve the desired channel shape

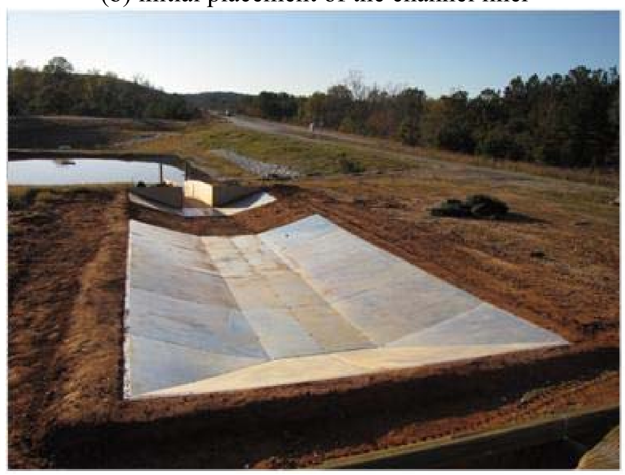
Figure 4.18: Fine Grading of the Prototype Test Channel.

Installing the sheet metal channel liner was the final step in constructing the prototype channel. With exception to the liner used for the head of the channel, each liner section lined one half the width of the channel and 5 ft of the channel length. Each section was seemed together with 2 in. wide angle iron attached to each adjacent section with self-tapping screws. Each seam was sealed with clear, weather proof silicone caulk. A berm was constructed around the perimeter of the ditch to divert runoff from undermining the channel liner. Nonwoven filter fabric was installed over the berm to protect it from weathering. ALDOT No. 57 crushed stone was placed around the berm to further protect it from undermining. Figure 4.19 displays the channel liner installation progression.

(a) sheet metal channel liner and angle iron

(b) initial placement of the channel liner

(c) earthen section of the channel after liner installation



(d) berm and drain around channel prototype

(e) view of the channel liner from the top of the channel

(f) final installation of the channel prototype

Figure 4.19: Installation Progression of the Prototype Sheet Metal Channel Liner.

Feasibility tests were performed on the prototype channel to evaluate the conveyance functionality of flows introduced into the channel and the functionality of the channel to install ditch checks to be evaluated for performance. This is shown in Figure 4.20.

(a) ditch check feasibility test

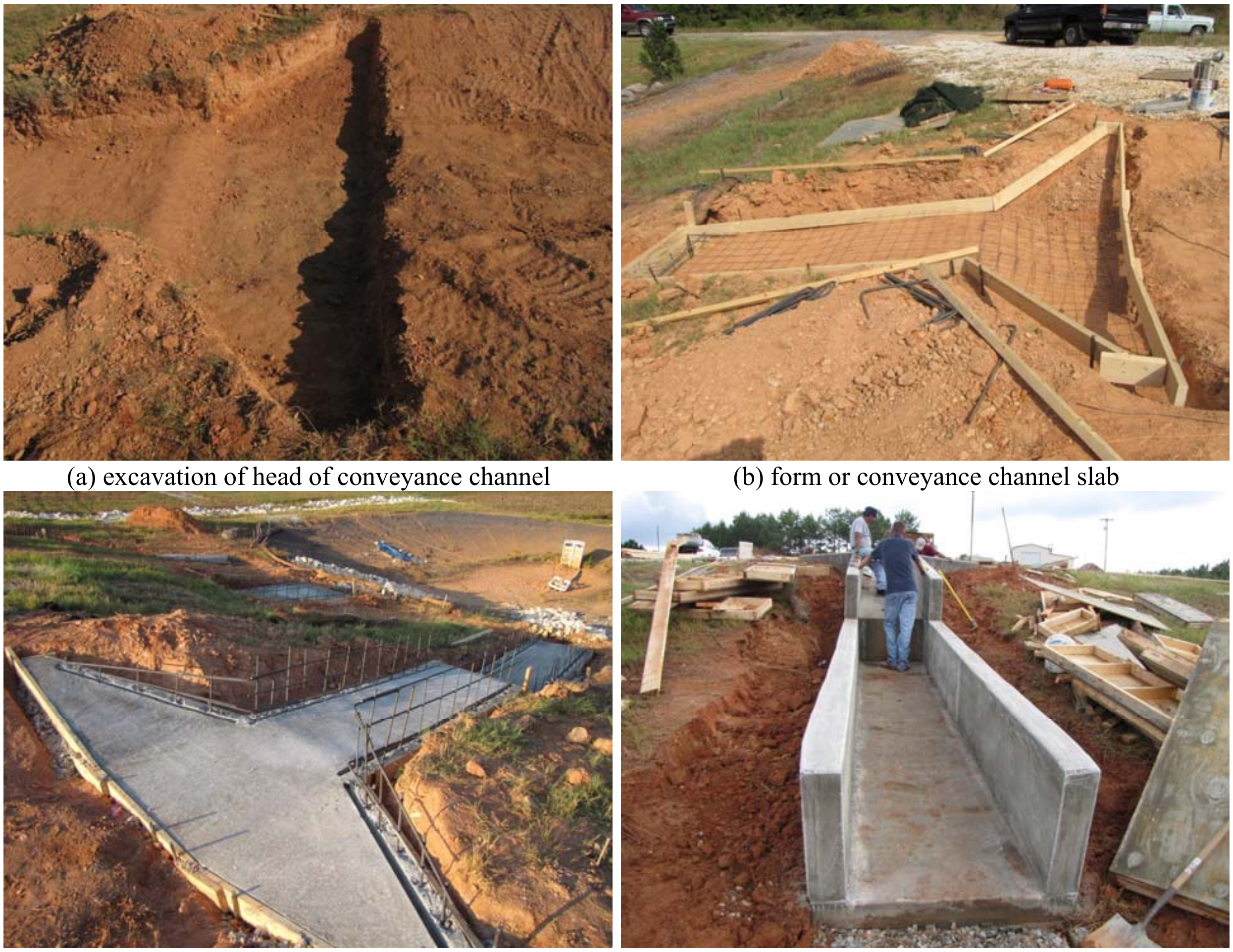
(b) conveyance ability of ditch check channel

(c) initial test setup

(d) initial flow introduction calibration

Figure 4.20: Initial Testing to Evaluate Ditch Check Channel Testing Feasibility.

Once it was determined that the testing capabilities of the ditch check channel was adequate, other upgrades were installed to enhance the usability and longevity of the ditch check testing channel. First the plywood lined conveyance channel was removed and all conveyance channels were further excavated so that a permanent concrete conveyance channel could be constructed for each test channel. Reinforced concrete was used to construct each of the conveyance channels. Figure 4.21 shows the construction process for the ditch check channel conveyance sections.



(a) excavation of head of conveyance channel (b) form or conveyance channel slab

(c) reinforcement of retaining walls (d) constructed conveyance channels

Figure 4.21: Construction of Permanent Ditch Check Conveyance Channels.

During this time period the inlet protection channel discharge area was also permanently stabilized with reinforced concrete and is shown in Figure 4.22.


(a) excavation of drop inlet discharge area (b) slab and reinforcement


(c) retaining walls forms (d) constructed discharge area

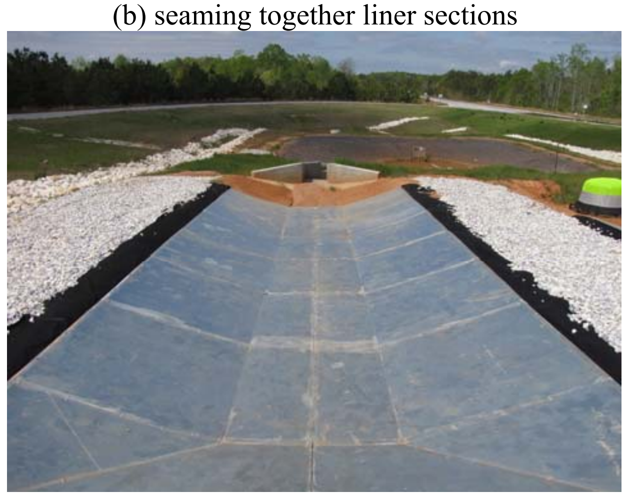
Figure 4.22: Construction of Permanent Inlet Protection Discharge Area.

After the conveyance and discharge areas were constructed, the inlet protection channel and the second ditch check testing channel were completed. These are shown in Figure 4.23 and Figure 4.24.

(a) placement of sheet metal liner

(b) seaming together liner sections

(c) grading of installation area



(d) construction final

Figure 4.23: Construction of Second Ditch Check Channel.

(a) grading and compaction

(b) fine grading using mold

(c) installation of sheet metal sections (d) construction final

Figure 4.24: Construction of Drop Inlet Channel.

The final attributes added to the facility were three sheet metal canopies which decrease the affect precipitation has on the testing effort. These are shown in Figure 4.25.

(a) addition of first canopy (b) post construction w/3 canopies (Google Earth)

Figure 4.25: Final View of Completed Construction Effort.

Once construction of the channels were completed, a testing methodology was developed properly test ditch check practices. This methodology is described in Chapter 5.

4.4 Summary

The construction of the AU-ESCTF proves a means for testing erosion and sediment control practices in channelized applications. Two ditch check testing channels and one inlet protection channel provide the means to evaluate different ditch check and inlet protection practices for ALDOT. The next step required for accomplishing this task is to develop a testing methodology which can create repeatable and representative conditions.

CHAPTER FIVE

METHODOLOGY

5.1 Introduction

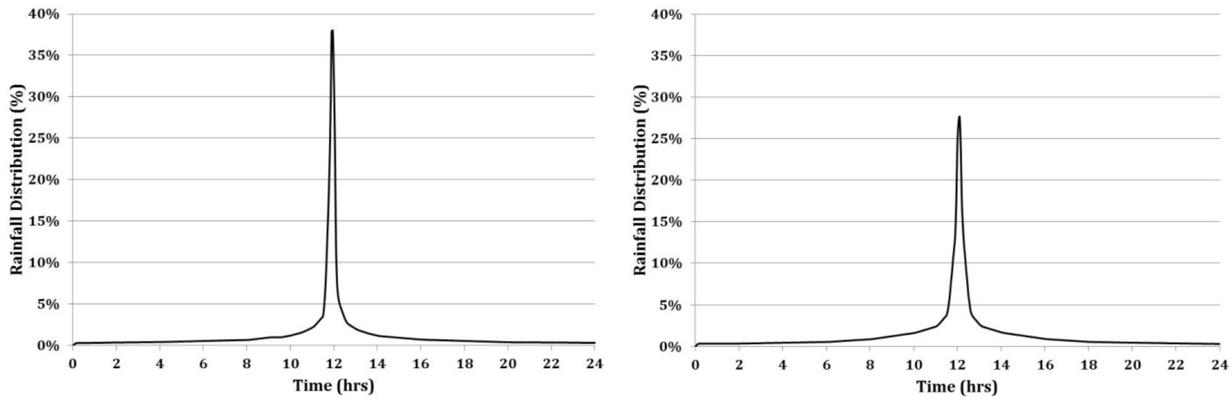
The purpose of this chapter is to outline the experimental methodology developed to evaluate the performance of ditch checks using large-scale testing techniques with the goal to create repeatable and reproducible results. The methodology includes procedures to achieve repeatable installations, consistent flow rates, and develop data collection and analysis methods. To accomplish this task, a testing methodology was developed and evaluated to determine if reproducible results could be created. The first step in this process was to attain and develop testing apparatuses for creating the desired testing conditions. This included acquiring water pumps and developing a water introduction system. Next, the flow rates had to be confirmed independently of the manufacturer specified flow rates. Once the water flow conditions were verified, a data collection methodology had to be developed to provide the researchers with accurate and reproducible data. And finally, installation methodologies had to be developed for each ditch check to attain consistent results between ditch check tests. The entire process for developing an experimental methodology is discussed herein.

5.2 Test Flow System

The first requirement for evaluating ditch checks was developing a flow system which could create conditions expected in the field. ASTM D 7208-06 specifies a flow rate of 3.0 cfs for ditch check performance testing. This flow rate was not justified in the test standard, nor was specific flow data for testing ditch checks identified in the literature, therefore further hydrologic investigation was required.

5.2.1 Design Storm and Drainage Area

A literature review produced no specific direct guidance for maximum flow rates for ditch checks beyond recommendations of drainage areas, and these areas ranged from 0.5 acres to 10 acres. A design storm and drainage area was developed to evaluate different possible flow conditions. Determining a representative theoretical watershed for roadside ditches relies on many different variables which include: geometric road design, soil conditions, and cover conditions. Technical Release 55 (TR-55) is a widely used method for determining peak flow rate at an outlet (Ponce and Hawkins 1996, Mishrah and Singh 1999). TR-55 uses SCS rainfall distributions to mimic theoretical rain events prevalent for different U.S. regions. A graphical method and a tabular method are both available for use and both methods result in very similar peak flows. This method takes into consideration flow type (i.e., sheet flow, shallow concentrated flow and channelized flow), ground cover and soil conditions, geographical storm type (i.e., Type I, Ia, II, and III), flow path, and conveyance geometry. Figure 5.1 shows the hyetographs based on the SCS storm Type II and Type III typical for the southeast and Alabama. These show the percent rainfall over a 24 hour period for a 2-year storm event. It should be noted that Type II distributions have higher peak rainfall events than Type III, but Type III distributions have longer duration peaks than Type II.



(a) SCS Type II Hyetograph (b) Type III Hyetograph

Figure 5.1: Southeast U.S. SCS Rainfall Distributions.

For testing erosion and sediment control products, it may be appropriate to design for the worst case scenario for ground cover for a construction site. Assuming there is no vegetation, this case is described by TR-55 as *Newly Graded Areas (pervious areas only, no vegetation)*. This results in four possible curve numbers for the four hydrologic soil groups A thru D which are: A–77, B–86, C–91, and D–94. These curve numbers describe conditions that result in low infiltration and high runoff as the curve numbers increase towards 100 which describes the condition of maximum runoff with no infiltration.

Developing a representative drainage area can be problematic due to the varying topography typical to linear projects in the state of Alabama. An interstate highway can present a consistent drainage area when the topographical constraints allow. Therefore a field survey was conducted to evaluate a local four lane interstate highway in Auburn, AL. Interstate 85 (I-85) near mile marker 56 was identified as a possible section to model. Inlet placement was used to identify the drainage spacing along the roadway median. One of these drainage sections is shown in Figure 5.2.

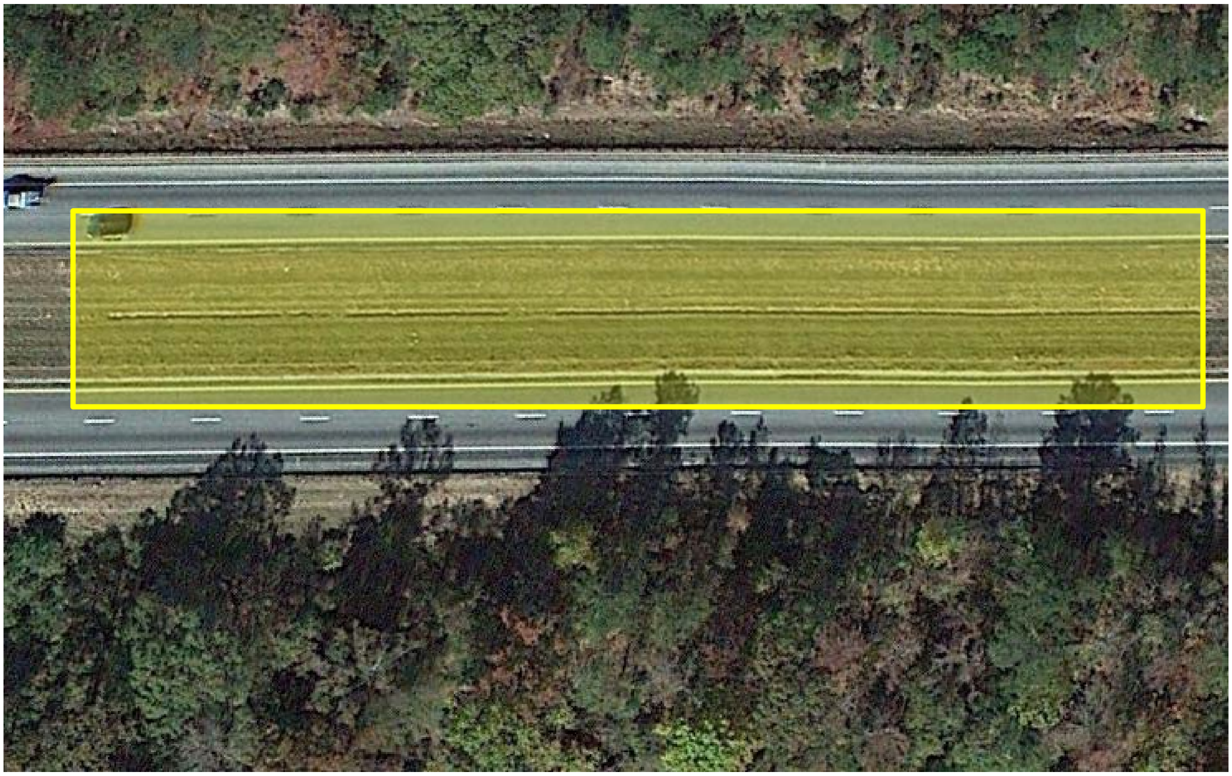


Figure 5.2: I-85 Field Survey Single Drainage Area Aerial.

The drainage basins ranged between 0.58- to 1.17-acres and the average area being 0.73-acres. Slopes along the corridor ranged between 0.7 and 4.4%.

Using this information and typical design details a theoretical drainage area was developed to mimic a highway median. This consisted of two, 12 ft lanes each with 10 ft shoulders drain towards a 44 ft wide median. The basin is sloped longitudinally at 5% which mimics the test channel slope. The median slopes are 6H:1V which drain into the center channel which runs longitudinally with the basin. The drainage basin length is 495 ft which equates to a 1 acre drainage basin. This drainage area is similar to the measured I-85 basins. This drainage area also coincides with the riprap ditch check recommendations of the NCDOT. It should be noted that that riprap ditch checks are considered one of the most structural stable ditch check practices and are typically reserved for higher flow rate conditions. Therefore, it is important to determine the other flow rate limitations of other ditch check practices.

Designers often use the local 2-year, 24-hour storm event to design erosion and sediment control practices on construction sites (Yoder et al. 2007). This storm frequency has been used by the EPA to dictate compliance on construction sites. The 2009 EGL published by the EPA required all discharges up to a 2-year, 24-hour storm frequency be adhere to the 280 NTU limitation. Though the effluent limit was stayed, the EPA does require sediment basins to provide enough storage for the 2-year, 24-hour storm event. This leads designers to often use the 2-year, 24-hour storm event in other design aspects of the SWPPP.

Technical Paper No. 40, Rainfall Frequency Atlas of the United States (USDA 1961) shows the precipitation depth (P) for the 2-year, 24-hour storm frequency for the state of Alabama ranges between 3.7 and 6.0 in. across the state. A mean value of 4.85 in. was determined using this range. TR-55 was referenced for the 2-year, 24-hour storm event for Montgomery County, AL which is 4.6 in. This falls close to the mean value of 4.85 in. for the state. Montgomery County, AL was therefore chosen due to being the central location within the state and for being close to the mean rainfall rate of the state. Montgomery County also falls within the Type III rainfall distribution pattern which makes up the majority of the state.

PondPack was used to determine the runoff hydrograph for the 2-year, 24-hour storm event. The previously described theoretical drainage area was used to develop this hydrograph. The time of concentration for the drainage area was 0.1 hours and the weighted curve number (CN) used was 88.4 which was based upon the state average curve number for newly graded urban areas based on the average hydrologic soil group (USDA 1986). Figure 5.3 shows the resulting runoff hydrograph.

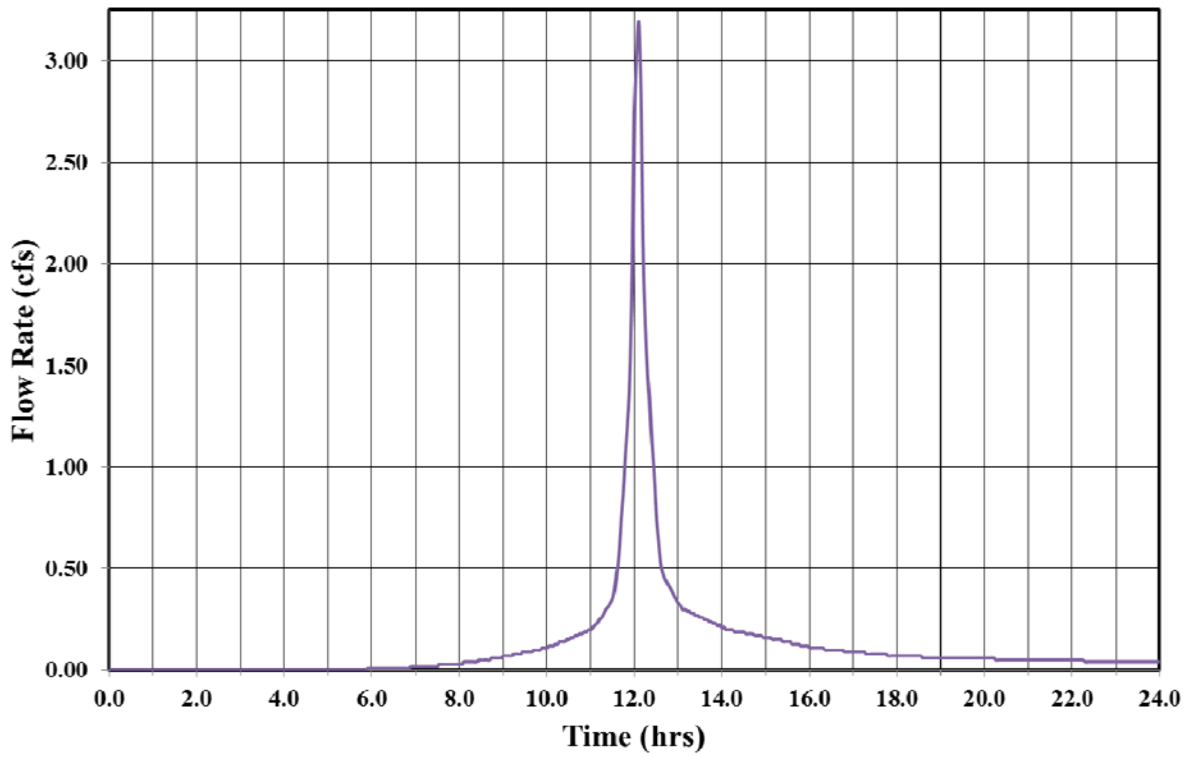


Figure 5.3: Runoff Hydrograph for Montgomery County, AL 2-year, 24-hour Event.

Table 5.1 shows the average flow rate (Q) and total volume for the entire storm event as well as other time spans which centered on the peak of the storm. It was a desire of this research effort for a testing regime which could mimic the increasing characteristic of a hydrograph which was representative of field runoff conditions.

Table 5.1: Volume and Runoff for Montgomery County, AL 2-year, 24-hour Event

Peak Time Period (hrs.)	Drainage Volume (ft^3)	% Volume	Average Q (cfs)
24	11,734.2	100%	0.136
6	9,361.8	80%	0.433
3	7,583.4	65%	0.702
2	6,742.8	57%	0.624
1.5	6,193.8	53%	0.574
1	5,454.0	46%	1.515
0.5	4,138.2	35%	2.299

5.2.2 Pump System

Once a theoretical hydrograph was developed, it became necessary to identify a water introduction system which would be representative of the 2-year, 24-hour storm event. Several pump systems were evaluated for flow capacity including the Northstar® 3" semi-trash pumps. This pump is specified by the manufacturer to pump at a maximum rate of 15,850 gal/hr (0.59 cfs). This flow rate is close to the 1.5 hour average peak flow rate shown in Table 5.1. Using multiple pumps would allow for higher flow rates and volumes. Two pumps would create a flow rate of 1.18 cfs and three pumps would create a flow rate of 1.77 cfs based upon manufacturer's specifications. Due to the limited water source, it was deemed necessary to limit the testing time for each ditch check. Therefore the 0.5 hour test time specified by ASTM D 7208-06 was adopted for this testing effort. Table 5.2 shows the flow rate and volumes created by three Northstar® 3 in. port sized semi-trash pumps if the testing flow rate was divided into three 10 minute flow rates which would mimic the rising limb of a hydrograph.

Table 5.2: Manufacturer Specified Flow Rates and Vol. for Northstar® 3 in. Trash Pump

# of Pumps	Time Period (hrs)	Q (cfs)	Volume (ft^3)	% Volume of 2 year, 24hour
1	0.17	0.59	354	3%
2	0.17	1.18	708	6%
3	0.17	1.77	1,062	9%
Total	0.50	N/A	2,124	18%

The flow rate created by using three pumps resulted in a flow rate greater than the average 1 hour peak flow rate shown in Table 5.1. Only 15 minutes of the storm event will create a flow rate greater than this highest flow rate of 1.77 cfs. Therefore, this flow condition was deemed adequate and allowed the ditch checks to be tested under different flow regimes by varying the number of pumps used for each test.

5.2.2.1 Water Introduction System

Once the pumps were procured, a water introduction system had to be developed for these pumping conditions. During testing these pumps are set up in series and pump from the upper supply pond to a 40.1 ft^3 (300 gal.) polypropylene trough. This trough has been modified with three openings specifically designed to accommodate flow from each pump, (i.e. when only one pump is pumping water, the lowest opening is being fully used, when two pumps are pumping water, two openings are being fully used, and when three pumps are pumping water, all the openings are fully discharging). This is helpful during testing to visually ensure the pumps are pumping at the required rate. Figure 5.4(a) shows the outlet configuration and the respective water levels when the pumps are operating in series. Figure 5.4(b) illustrates the condition when all three pumps are engaged, and all three openings on the trough are being used to discharge the flow.

(a) corresponding flow rates per opening (b) max. flow when all 3 pumps are operating

Figure 5.4: Trough Openings and Flow Levels at Various Flow Conditions.

5.2.2.2 Flow Verification

Once the water introduction system was developed, the pumps flow rates were evaluated to determine actual flow conditions developed by the pumping system. The pump flow rates were determined by first calculating the volume of the trough from the bottom up to the bottom of the first trough opening. This volume was estimated to be 20.4 ft^3. The time required to fill the trough to this volume by each pump was recorded and reported in Table 5.3. This task was repeated five times per pump to collect data and establish an average time to fill the trough, which was 36.5 seconds. This corresponds to an average flow rate of 0.56 cfs. This flow rate of 0.56 cfs is very close to the manufacturer specified flow rate of 0.59 cfs and differences can be attributed to friction and head losses within the system. Therefore, the measured flow rate of 0.56 cfs will be used as the true pumping rate in all subsequent analyses.

Table 5.3: Flow Rate Determination for Each Pump (Trough Volume = 20.4 ft^3)

Pump	Time (sec.)	Q (cfs)	Avg. Q (cfs)
Left	36.82	0.55	**0.57**
	34.87	0.59	
	35.84	0.57	
	35.38	0.58	
	35.47	0.58	
Middle	37.81	0.54	**0.55**
	36.96	0.55	
	36.32	0.56	
	36.35	0.56	
	37.00	0.55	
Right	36.13	0.56	**0.55**
	37.22	0.55	
	36.62	0.56	
	37.25	0.55	
	37.19	0.55	
AVERAGE	**36.48**	**0.56**	

The pumping rates evaluated from the data collected in Table 1 were used to evaluate the performance of a 90° sharp crested v-notch weir which was installed in the concrete channel, as shown in Figure 5.5(a). The purpose of the 90° sharp crested v-notch weir was two-fold: (1) to evaluate the performance of the weir equation and (2) as a secondary means of determining the flow rates of the three pumps. Figure 5.5(b) illustrates the heights on the 90° sharp crested v-notch weir that correspond with the three flow rates of 0.56, 1.1, and 1.7 cfs. The v-notch weir was chosen due to ease of construction and suitable application for discharges up to 5 cfs (Eli 1986).

(a) installation of weir in concrete channel (b) water level heights for various flow rates

Figure 5.5: 90° Sharp Crested V-notch Weir.

The equation for flow over a 90° sharp crested v-notch weir is given in Equation 5.1 (Brater et al, 1995):

$$Q = 2.44\ H^{2.5} \qquad \text{(Equation 5.1)}$$

where,

H = height (ft)

Q = flow rate (ft^3/s)

A plot of the 90° sharp crested v-notch weir equation used to determine flow rates is shown in Figure 5.6. The average Q (i.e. 0.56 cfs) from Table 5.3 along with the respective flow rates with two pumps (i.e., 1.1 cfs) and three pumps (i.e., 1.7 cfs) discharging have been plotted with respect to the measured heights of flow going over the weir are shown in Figure 5.5(b). The figure shows that the weir equation was able to predict the known flow rates.

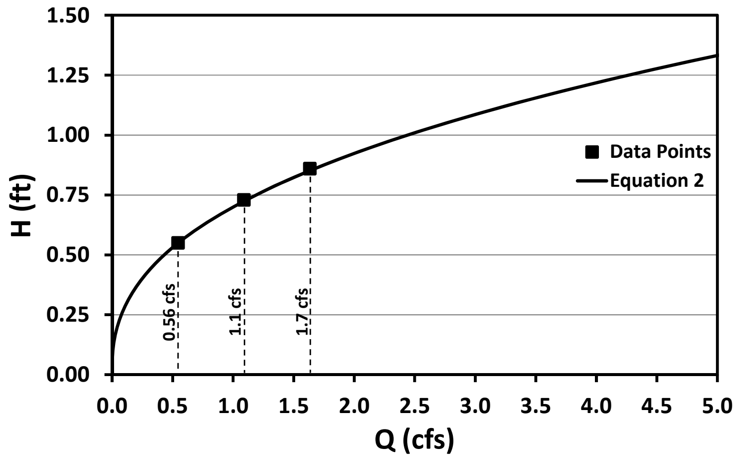


Figure 5.6: Test Data vs. Weir Equation.

5.3 Test Channel Preparation

The soil used in the earthen section of the test channel was classified using the Unified Soil Classification System (USCS). The particle size distribution of the soil was plotted against an inverse semi-log graph as shown in Figure 5.7.

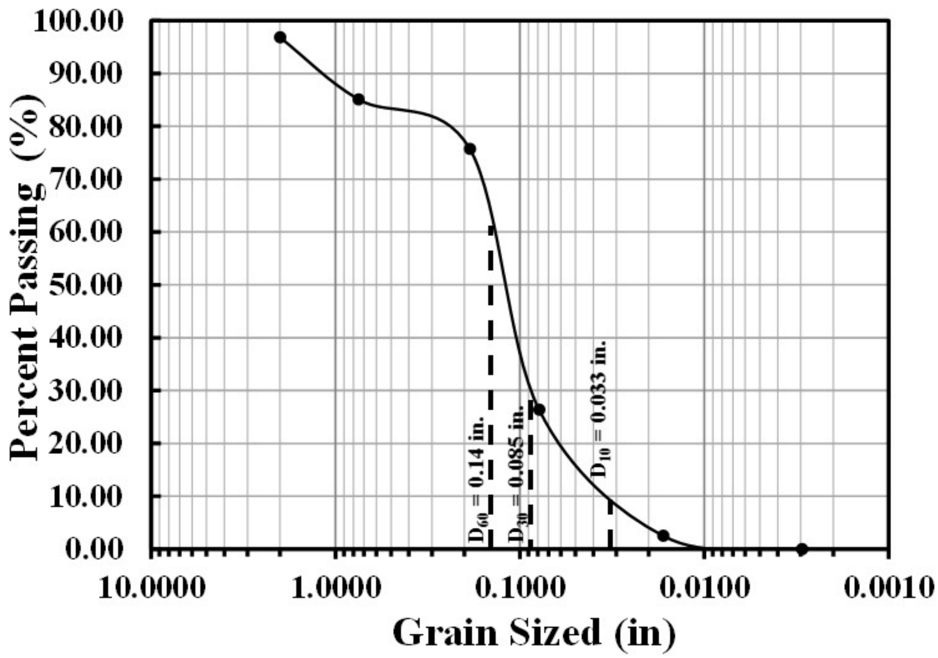


Figure 5.7: Grain Size Distribution for Earthen Section.

Table 5.4 displays the sieve analysis of the soil used for the earthen section. Using the data illustrated in Figure 5.7, Table 5.4, and Table 5.5; the soil used for testing within the earthen section of the channel has been classified as a poorly graded sand (SP).

Table 5.4: Sieve Analysis of Earthen Section Soil

Sieve	Apparent Opening Sizes (in)	Mass Retained (lbs)	Percent Retained (%)	Percent Passing (%)
2"	2.0000	0.00	0.00	100.00
0.75"	0.7500	0.17	3.17	96.83
#4	0.1870	0.64	11.75	85.09
#10	0.0787	0.51	9.36	75.72
#40	0.0167	2.68	49.34	26.38
#200	0.0030	1.29	23.86	2.52
Pan	0.0000	0.14	2.52	0.00

Table 5.5: Properties of Earthen Section Soil

SP (Poorly Graded Sand)	
D_{60} = 0.14 in	C_u= 4.24 %
D_{30}= 0.085 in	C_c= 1.56 %
D_{10}= 0.033 in	% Gravel= 14.91 %
LL= 31.03 %	PL= 15.97 %

The moisture-unit weight relationships were also determined for this soil. The Standard Proctor Test was performed to determine the maximum dry density (ρ_{dmax}) and the optimum moisture content (OMC) for the soil. Figure 5.8 and Table 5.6 illustrate the results of this test. The ρ_{dmax} determined was 123.8 lbs/ft^3 and the OMC was determined to be 10.9%.

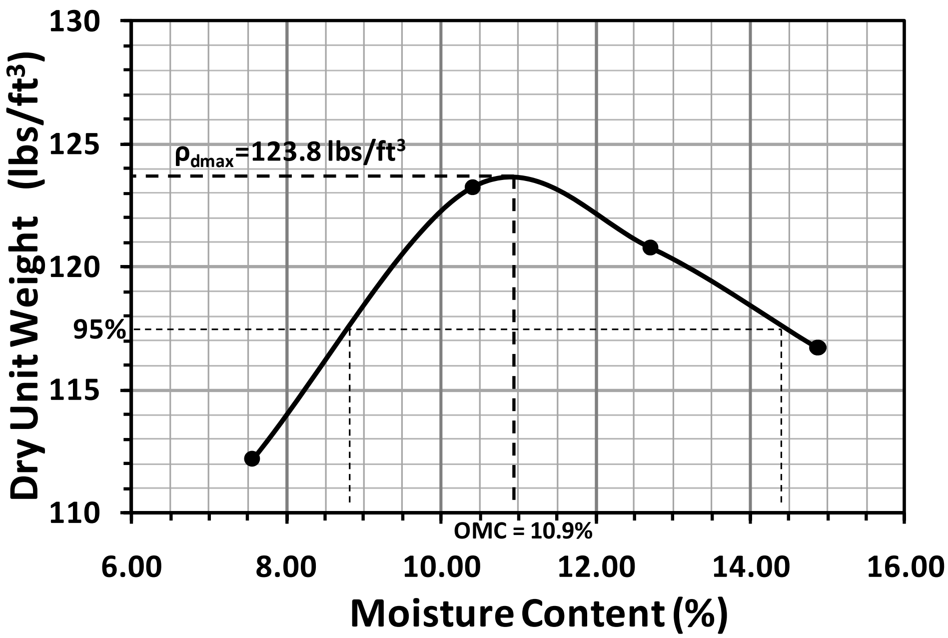


Figure 5.8: Proctor Curve for Earthen Section Soil.

Table 5.6: Proctor Test Data for Earthen Section Soil

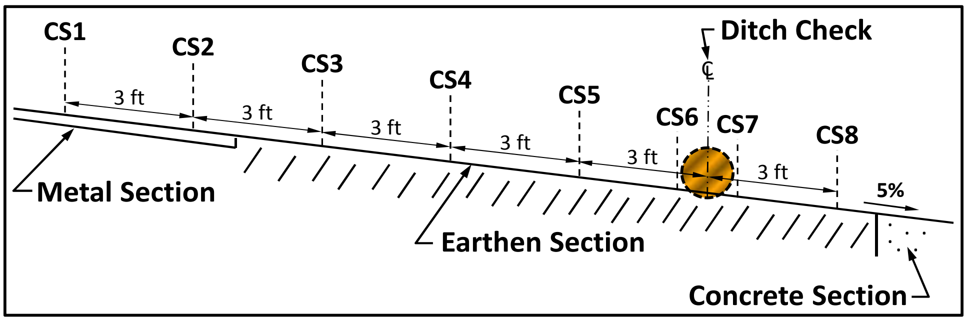
Sample	Total Soil Mass (lbs)	Dry Soil Mass (lbs)	Water Content (%)	Bulk Density (lb/ft^3)	Dry Density (lbs/ft^3)
1	4.02	3.74	7.56	120.68	112.20
2	4.54	4.11	10.41	136.09	123.25
3	4.54	4.03	12.71	136.13	120.77
4	4.47	3.89	14.87	134.07	116.71

The dry unit weight for 95% compaction was calculated to be 117.5 lbs/ft^3 with a moisture content ranging from 8.8% to 14.4%. During the regrading process of the earthen section, an upright rammer hammer with a compaction plate of 14 in. x 11.5 in., a blow count of 600 plows per minute and a compaction force of 2,700 lbs is used to compact the earthen section between tests. In place density is taken using a density drive hammer and thin walled Shelby tubes. Density samples are taken periodically during the testing effort. The samples are taken at varying locations to minimize bias based upon sampling from the same location. The average in-place density taken was 117.5 lbs/ft^3 at an average moisture content of 10.01% which falls within the specified range.

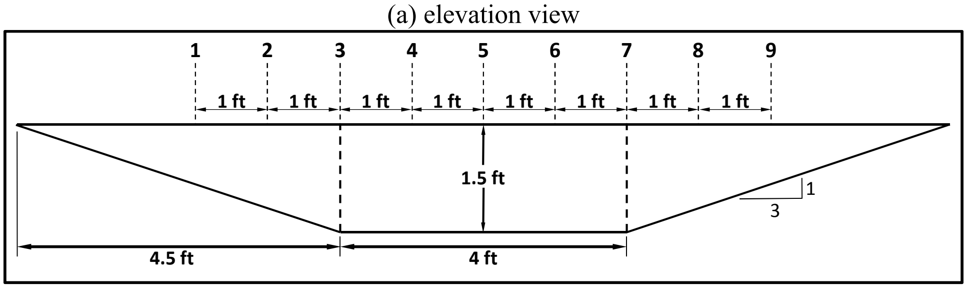
5.4 Data Collection

Pre and post channel surveys are performed for each test using manual surveying techniques. The data is collected at predetermined cross-sections as shown in Figure 5.9(a). These cross sections are spaced 3 ft apart upstream and downstream of the ditch checks being tested. For each cross section, the height to the cross section string line from the channel bottom is measured using an engineer’s ruler to determine pre versus post erosion and deposition patterns. These measurements are taken at all points shown in Figure 5.9(b). Also, at these cross-sections, once steady-state flow conditions are achieved, water depth and velocity measurements are taken at cross sectional measurement points 4, 5 and 6 for every cross section

(CS1-CS8) shown in Figure 5.9(a) and Figure 5.9(b). These points are averaged to determine the average water depth and average velocity for each cross section. The distance from the upstream face of the ditch check to the hydraulic jump is also recorded once steady state conditions are achieved to determine subcritical flow length (i.e. impoundment) created by a ditch check based upon its ability to impound water.



(a) elevation view



(b) cross-sectional view

Figure 5.9: Ditch Check Test Channel Dimensions and Configuration.

Velocity measurements are performed using a stagnation tube as shown in Figure 5.10. The stagnation tube measures the velocity head which is the difference between the hydraulic grade line (HGL) which is also the water surface for open channels and the energy grade line (EGL). This is measured as the height of the water in the tube above the water surface and is represented as *l* in Figure 5.10(a) and *h* in Equation 5.2. Using this difference, the velocity can be calculated using Equation 5.3.

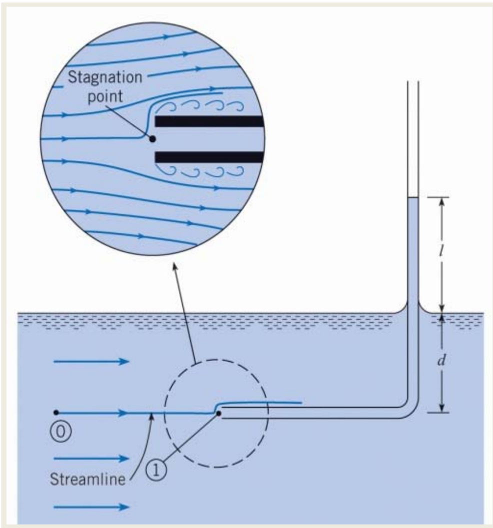
$$v = \sqrt{2gh} \qquad \text{(Equation 5.2)}$$

where,

v = velocity (ft/sec)

g = acceleration due to gravity (32.2 ft/sec^2)

h = measured velocity head (ft)



(a) stagnation tube principle (b) depth and velocity measurements during testing

Figure 5.10: Ditch Check Data Collection Process.

Using the collected average data, the slope of the energy grade line (EGL) for the water profile was plotted as specified by ASTM D 7208-06. The EGL is defined by Equation 5.3 (ASTM 2006).

$$EGL = WSE + \frac{v^2}{2g} \qquad \text{(Equation 5.3)}$$

where,

EGL = energy grade line (ft)

WSE = water surface elevation (ft)

v = average water velocity (ft/sec)

g = gravitational constant (32.2 ft/sec2)

The slope of the EGL for long, unimpeded, continuous flow channels should closely mimic the channel slope. When the channel is impeded (e.g., by a ditch check), the slope of the EGL within the impoundment area becomes smaller than the channel slope as ponding depths increase towards the flow impedance. The potential energy built up by the subcritical flow is returned to kinetic energy as the impounded water goes under, through, and/or over the ditch check. In addition to impounding water and reducing erosion causing shear stresses, the ditch check must also withstand the change from potential energy back to kinetic while simultaneously maintaining the structural integrity of the installation. This makes the installation of the ditch check critical when implemented in the field.

5.5 Ditch Check Installation Improvement Testing Methodology

This research focused on the evaluation of four different ditch check practices for the purpose of improving installation and determining maximum performance. These practices were: (1) wattles, (2) riprap, (3) silt fence, and (4) sand bags. A testing methodology for determining the optimum ditch check installation for each practice was developed.

5.5.1 Bare Soil Control

A bare soil control test was performed that consisted of the channel being graded and compacted to experimental specifications without a ditch check installed. This test establishes a baseline for flow velocities and water depths under supercritical flow conditions (i.e., no impedance of flow) at each cross-section (CS1-CS8).

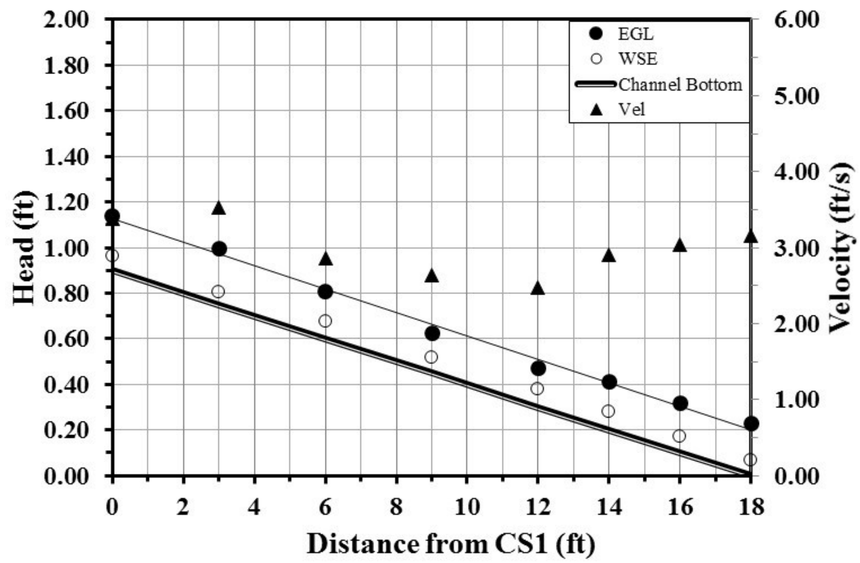


Figure 5.11: Bare Soil Control Average EGL, WSE, and Velocity.

Note that the drop in velocity is due to the transition from metal liner to the earthen section at 5 ft from CS1. This bare soil control average is used to compare hydraulic performance to other ditch check practices throughout the testing process. Once the Bare Soil Control was tested, the wattle installation improvement testing commenced.

5.5.2 Installation Improvement Testing Regime

After performing the bare soil control test, the first step is to test each practice using the standard ALDOT installation per ALDOT standard drawings and specifications. This installation is tested using three replications at a prescribed flow rate (i.e., 0.56, 1.12, or 1.67 cfs) for 30 minutes to determine the average performance of each ditch check practice. The standard installation is then evaluated, based upon performance, to determine overall strengths and deficiencies. If this installation is determined to be inadequate, a new installation is developed based upon the previous tests performance, or by adapting other practices used by other state highway agencies (SHAs). Once the new test installation is developed, this new installation is tested using the same testing parameters as the previous installation so direct comparison can be made. This process is continued until the most feasible and effective installation (MFE-I) is determined based upon ease of installation, overall practice improvement, inspection and maintenance requirements, and cost. This experimental testing regime is outlined in Figure 5.12.

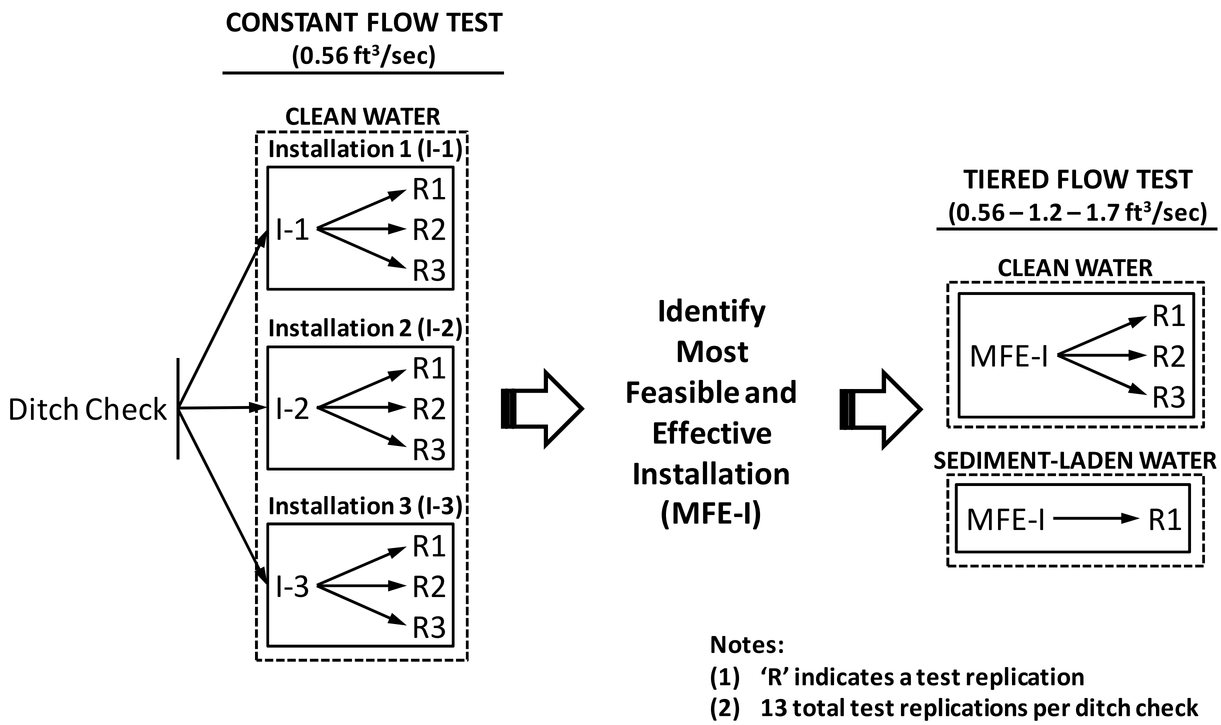


Figure 5.12: Ditch Check Testing Regime for Determining the MFE-I.

Once testing was been completed and the MFE-I has been identified, this installation may be tested using a tier flow test regime of 0.56 cfs, 1.12 cfs and 1.68 cfs for 10 minutes for each flow rate for a total test time of 30 minutes. The purpose of this test regime is to evaluate the ditch check as well as the installation to determine performance and effectiveness at each flow rate. Each ditch check can then be categorized based upon flow rate.

5.6 Installation Methodologies

The installation of each ditch check is important for attaining reproducible results. This is necessary for improving installations and for evaluating different products using the same installation. To properly evaluate each ditch check practice, the installations should be easily repeated for each replication. Each installation varies based upon previous installations performance or by previously prescribed improvements which are developed from existing installation options which were developed by SHAs such as ALDOT.

5.6.1 Wattle Installations

A series of constant low flow (i.e., 0.56 cfs), large-scale ditch check experiments were performed to evaluate each wattle installation configuration. These were done to comparatively analyze the seven different wattle ditch check installation configurations. For each installation configuration, including the control with no wattle installed, three replicate tests were performed, totaling 24 large-scale experiments.

5.6.1.1 Wattle Materials for Installations:

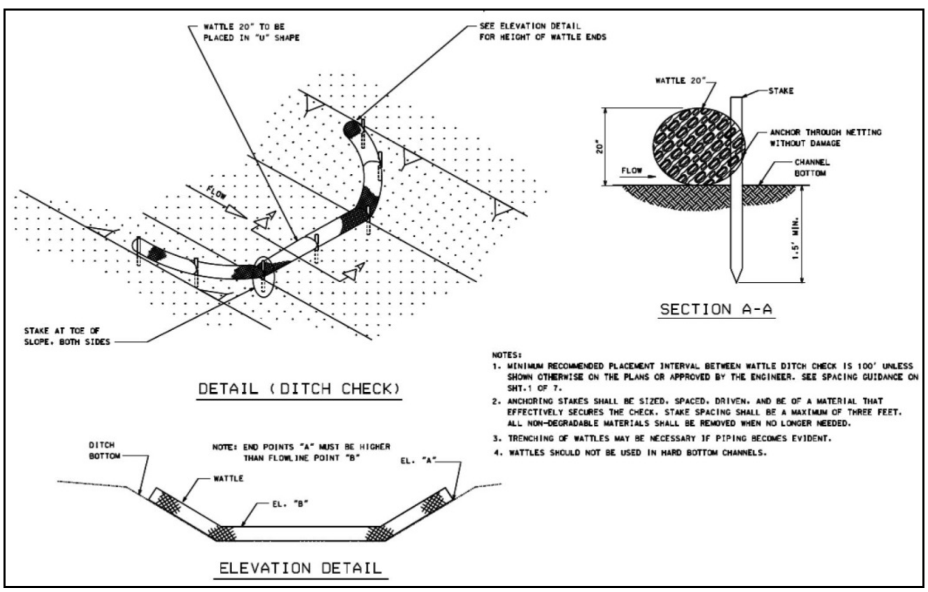
The following is a list of materials used for the various wattle installation configurations:

- *wattle:* 20 in. diameter, 20 ft long wheat straw wattle with synthetic netting,
- *wooden stakes:* 1 in. x 2 in. x 3 ft, used to secure the wattle in place,
- *sod staples:* 11 gauge metal, 6 in. long x 1 in. wide U-shape staples, used to secure the filter fabric underlay and the wattle, and

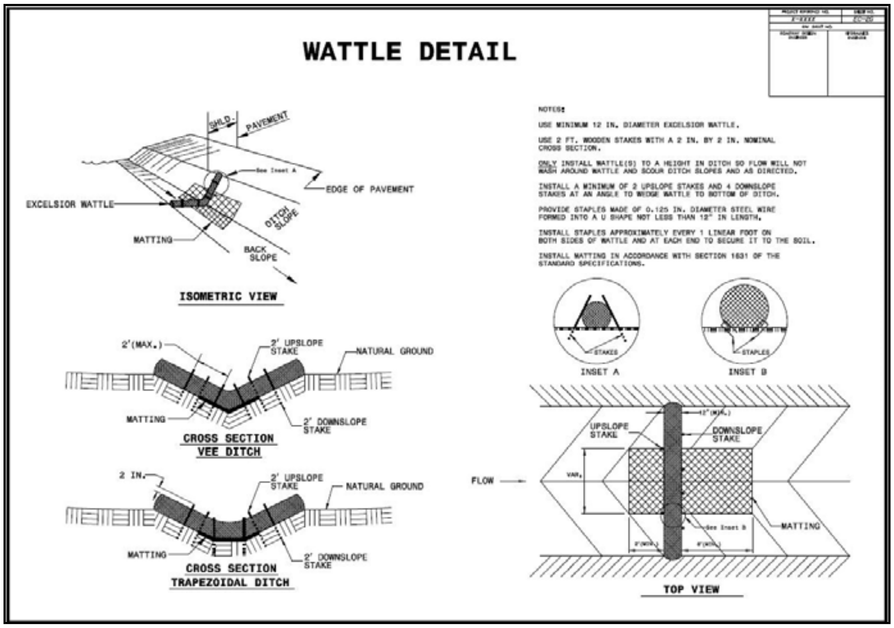
- *filter fabric (FF) underlay:* 8 oz., nonwoven FF, 7.5 ft long, 15 ft wide. Extends 3 ft upstream from the upstream face of the wattle. The fabric underlay extends 3 ft downstream beyond the wattle. The upstream and downstream edges of FF were secured with two rows of sod staples spaced 10 in. apart and one row longitudinally along each side and the centerline of the fabric spaced 1.5 ft.

5.6.1.2 Wattle Installation Variations

The standard ALDOT installation was comprised of simply laying out the 20 in. wattle across the channel, perpendicular to flow. The wattle was staked in place on the downstream side, piercing the netting to secure the wattle in place. This installation is shown in Figure 5.13(a). Variations of the wattle installation that were also based directly off of the NCDOT wattle detail shown in Figure 5.13(b).



(a) ALDOT standard wattle detail.



(b) NCDOT standard wattle detail.

Figure 5.13: Comparison of ALDOT (ALDOT 2012) and NCDOT Wattle Installation Practices (NCDOT 2012).

The NCDOT installation uses an underlay to protect the channel bottom near the installation to prevent scour and undermining. This installation also uses a nondestructive staking technique where the stakes are driven into the ground on the upstream and downstream side of the wattle, angled towards the wattle in an A-frame or teepee form without piercing the wattle or netting. This nondestructive method is meant to increase the longevity of the wattle by not tearing the outer encasement. The NCDOT installation also uses U-shaped sod staples to secure the wattle in place and to increase ground contact by pushing the netting downward, flattening out the wattle bottom and thereby reducing undermining.

Many manufacturers recommend digging a trench in the channel bottom and placing the wattle into the trench to help minimize undermining. The American Excelsior Company recommends this for their Premier Straw Wattle ™. This installation is shown in Figure 5.14.

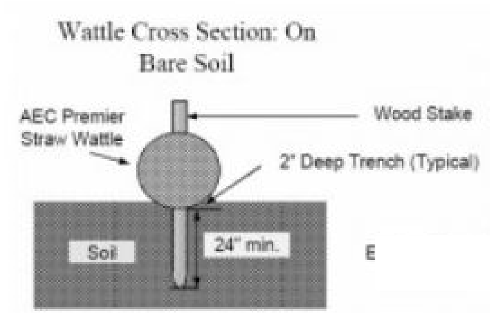


Figure 5.14: American Excelsior Premier Straw Wattle™ Install Guide (AEC 2013).

From these installation guides, seven different installations were developed for testing, including the ALDOT, NCDOT, and manufacturer recommended installations. The channel was prepared to experimental specifications for all tests performed on the seven different wattle installation configurations so direct comparisons could be made between the various configurations. The following seven wattle installation configurations were developed and tested:

(1) *Downstream Staking*: mimicked ALDOT installation, wattle is placed across the channel in a U-shape, concave upstream, and secured with wooden stakes driven into the ground a minimum of 1.5 ft and positioned every 2 ft on the downstream side of the wattle piercing the netting.

(2) *Teepee Staking*: mimicked NCDOT staking practices creating a “teepee” or A-frame over the wattle by driving the stakes into the ground a minimum of 1.5 ft next to the wattle without piercing the wattle or wattle netting. These stakes were driven in at an angle towards the wattle securing the wattle in place. Two stakes were installed upstream and five stakes installed downstream with a stake spacing of 2 ft.

(3) *Downstream Staking w/8 oz. FF*: wattle was installed with an 8 oz. FF underlay and secured in place using ALDOT staking practices.

(4) *Teepee Staking w/8 oz. FF*: wattle was installed with an 8 oz. FF underlay and secured in place following NCDOT staking practices.

(5) *Downstream Staking w/Trenching*: mimicked manufacturer recommended installation where the entire width of the wattle was trenched into channel 2 in. deep, perpendicular to the flow of water and anchored using ALDOT staking practices.

(6) *Teepee Staking w/8 oz. FF and Trenching*: a 2 in. deep trench extending the entire width of the wattle was excavated and covered with the 8 oz. FF underlay. The wattle was installed and secured using NCDOT staking practices.

(7) *Teepee Staking w/8 oz. FF + Staples*: wattle was installed exactly as described in configuration (4), also securing the bottom of the upstream and downstream face of the wattle to channel using sod staples along each side, spaced 12 in. apart to improve contact with the channel bottom.

5.6.2 Riprap Installation

A typical standard riprap installation, such as the ALDOT riprap installation shown in Figure 5.15 consists of placing riprap across a drainage channel perpendicular to flow, a minimum height of 1 ft and a maximum height of 3 ft. The standard detail also allows for choking by placing either smaller rock or filter fabric on the upstream side of the riprap ditch check. The purpose of the choker is to minimize pore spaces for water to flow through, thereby reducing flow-through rates and maximizing impoundment. This results in 3 installation variations that were tested to quantify performance of riprap ditch checks.

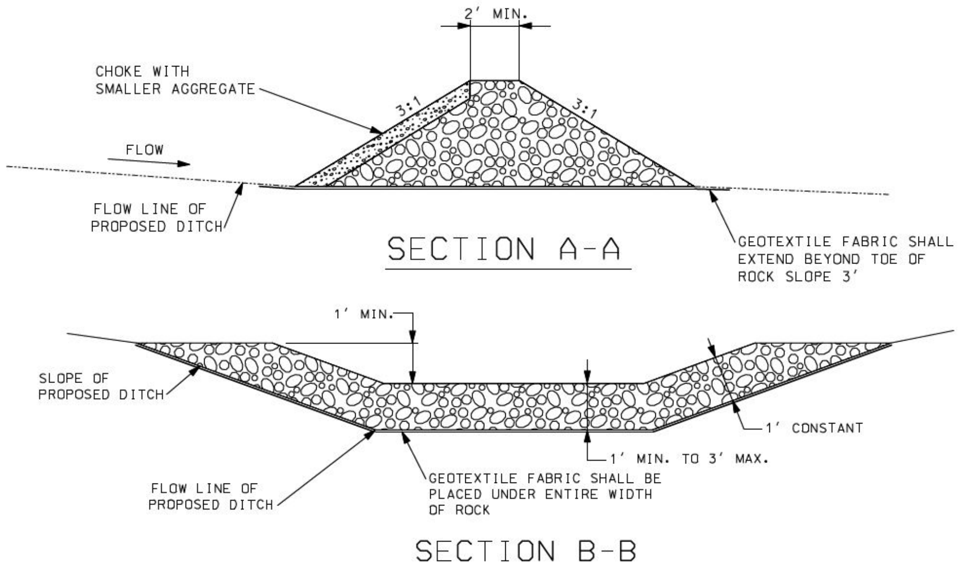


Figure 5.15: ALDOT Riprap Ditch Check Installation Detail (ALDOT 2012).

5.6.2.1 Riprap Materials for Installation

The following is a list of materials used for the various riprap installation configurations:

- *ALDOT Class I riprap:* consist of graded stones ranging from 10 to 100 pounds with not more than 10% having a weight over 100 pounds and at least 50% having a weight over 50 pounds and not over 10% having a weight under 10 pounds (ALDOT 2012),
- *modified no. 4 coarse aggregate:* consist of graded aggregate ranging from 4 in. to ¾ in. with not more than 10 % greater than 4 in., at least 20% to 55% 1 in. and no more than 15% smaller than ¾ in.,
- *sod staples*: 11 gauge metal, 6 in. long x 1 in. wide U-shape staples or 11 gauge metal, 6 in. long x $1^3/_8$ in. round-top sod pin , used to secure the filter fabric underlay, and
- *filter fabric (FF) underlay:* 8 oz., nonwoven FF, 12-20 ft long depending on installation, 15 ft wide. Extends 3 ft upstream from the upstream face of the riprap and pinned by two rows of sod staples spaced every 10 inches staggered on center. The filter fabric underlay

extends 3 ft downstream beyond the riprap. The downstream edge of FF was secured with sod staples spaced 10 in. apart. The FF is also pinned longitudinally along each side and the centerline of the fabric spaced 1.5 ft.

5.6.2.2 Riprap Installation Variations

A series of constant high flow (1.68 cfs), large scale ditch check experiments were performed to evaluate each installation configuration. Since riprap ditch checks are typically used for conditions where high flows or velocities are expected, a constant flow of 1.68 cfs was used for 30 minutes for each installation test. These tests were performed to comparatively analyze the three different riprap ditch check installation configurations. For each installation configuration, three replicate tests were performed, totaling 9 large scale experiments.

(1) *No Choker*: Class I ALDOT riprap is constructed per Figure 5.15 w/FF underlay extending 3 ft from the upstream toe and downstream toe of the ditch check. The middle of the ditch check is 1.5 ft high with a base width of 6 ft. The 8 oz. nonwoven FF underlay is pinned in the same manner used in the wattle ditch check underlay pinning method.

(2) *Modified no. 4 Stone Choker*: installed in the same manner as the *No Choker* installation, however, a layer of modified no. 4 stone was layered across the upstream side of the ditch check to minimize flow-through.

(3) *Filter Fabric Choker*: installed in the same manner as the *No Choker* installation, however, an 8 oz. nonwoven filter fabric choker is placed across the front face of the ditch check and 3 ft upstream of the toe to further minimize flow-through and undermining.

5.6.3 Silt Fence Installation

The silt fence ditch check installation tested uses a V pattern typically used by many SHAs including ALDOT. The ALDOT silt fence installation specifies constructing a silt fence ditch check at a 45^{o} angle with the "V" aligned to the direction of flow, concave upstream as shown in Figure 5.16. The center post of the "V" is to be installed in the center line of the channel with posts at each side of the side slope toe. There is no specific guidance for installing the silt fence ditch check beyond referencing the standard ALDOT silt fence installation for perimeter control in ALDOT ESC 200. This guidance specifies a minimum of 10 ft post spacing which is not a practical spacing requirement for ditch checks since the practice is subjected to concentrated flow conditions. The silt fence is to be reinforced with wire backing and trenched in a 6 in. by 6 in. trench to help prevent undermining.

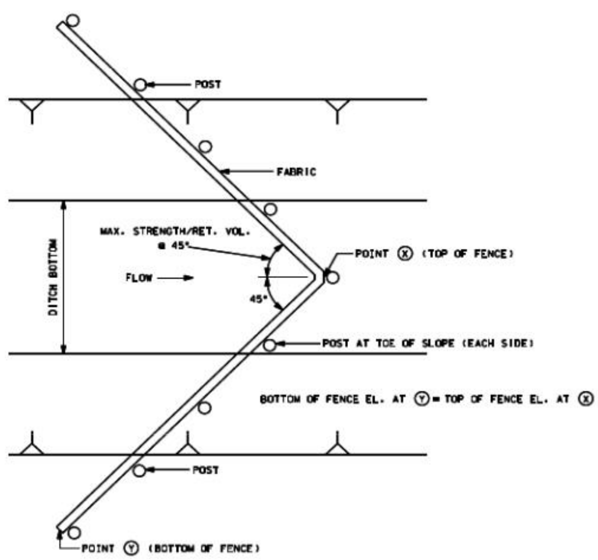
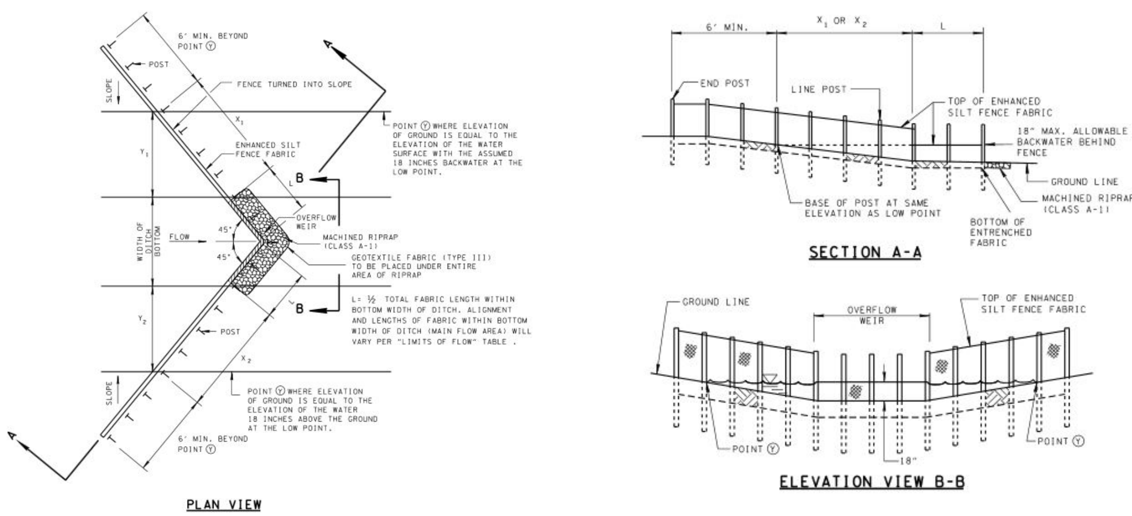


Figure 5.16: Standard ALDOT Silt Fence Ditch Check Detail (ALDOT 2012).

The minimum silt fence height is 32 in. and could create issues with impounding a large amount of water upstream. If the silt fence collapses with full impoundment of water upstream, all downstream erosion and sediment control practices could be subjected to a large hydraulic wave that could result in downstream practices failing. To minimize the potential of this situation, the Tennessee Department of Transportation (TDOT) developed an enhanced silt fence ditch check which uses a weir cut into the fabric to reduce the height of the ditch check and the resulting impoundment. This height can be adjusted based upon the depth of the channel. This installation also uses a downstream splash pad to help minimize downstream scour. This installation is shown in Figure 5.17.



(a) Plan view (b) Elevation and cross-sectional views

Figure 5.17: Tennessee Department of Transportation Enhance Silt Fence Installation (TDOT 2012).

Since the TDOT installation overflow weir allows a designer to adjust the maximum height for impoundment based upon channel geometry, the depth of impounded stormwater and the slope of the channel will directly affect the required ditch check spacing. In addition, since silt fences typically become clogged by sediment early in the life span of a ditch check, the ditch check could be assumed to be impervious if stormwater runoff is heavily sediment-laden. This clogging assumption needs to be considered when attempting to optimize the spacing of silt fence ditch checks.

5.6.3.1 Silt Fence Materials for Installation

The following is a list of materials used for the silt fence installations:

- *silt fence fabric*: 3.5 oz., nonwoven FF, 45 in. wide. The fabric is attached to wire backing with c-ring staples. The fabric is trenched in a 6in. by 6 in. trench or stapled to the channel bottom on top of a FF underlay using two rows of sod pins, staggered and spaced 10 in. apart,
- *FF underlay*: 8 oz., nonwoven FF, 12.5 ft long which extends the length of both of the V sides of the 45 degree ditch check installation. Each section is 3 ft wide and extends 1 ft upstream of the silt fence and 2 ft downstream of the silt fence when the pinned silt fence installation is used. The underlay is pinned to the channel bottom with sod pins spaced 10 in. on center,
- *sod pins*: 11 gauge metal, 6 in. long by 1 in. diameter circle top pins, used to secure the filter fence and the filter fabric underlay to the channel bottom,
- *wire mesh backing*: 14 gauge steel wire mesh with a minimum 6 in. by 6 in. vertical and horizontal spacing of the wire mesh,
- *c-ring staples*: 11/16 in., 16 gauge, galvanized steel. The c-ring staples are used to attach the filter fabric to the top wire of the wire backing,
- *studded t-post*: 5 ft studded steel t-post, driven into the ground 24 in. spaced 3 ft apart on center,
- *wire ties*: 6.5 in, 11 gauge, aluminum fence tie wires. These wire ties are used to secure the wire backing to the t-posts,
- *hay bale*: 3 ft long bound straw bale w/a weight of approximately 35 lbs,

- *modified no. 4 coarse aggregate:* consist of graded aggregate ranging from 4 in. to ¾ in. with not more than 10 % greater than 4 in., at least 20% to 55% 1 in. and no more than 15% smaller than ¾ in., and
- *ALDOT Class I riprap:* consist of graded stones ranging from 10 to 100 pounds with not more than 10% having a weight over 100 pounds and at least 50% having a weight over 50 pounds and not over 10% having a weight under 10 pounds (ALDOT 2012),

5.6.3.2 Silt Fence Installation Variations

Using the standard ALDOT installation and referencing the TDOT Enhanced Silt Fence Check installation various installation possibilities were developed and modified as testing took place. Concerns with downstream scour arose, and dissipation methods were investigated along with the standard V installation. Discussions began regarding the process of trenching the silt fence in and whether or not a silt fence could be installed without the need for trenching. Therefore a pinning method was also investigated. The following 6 silt fence installations were evaluated.

(1) *Standard V installation*: mimics ALDOT installation, center post placed in the channel centerline, posts are spaced 2.5 ft on center. Fabric and wire backing is trenched in a 6 in. by 6 in. trench. Overall fence height is 32 in.

(2) *V installation w/ Hay Bale Dissipater*: installed in the same manner as the ALDOT installation, however hay bales a placed downstream, abutted to the fence in an attempt to dissipate energy and reduce downstream scour next to the fence.

(3) *V Installation w/ Modified No. 4 Stone Dissipater*: installed in the same manner as the ALDOT installation, however modified no. 4 stone is placed downstream, in an attempt to dissipate energy and reduce downstream scour next to the fence.

(4) *TDOT Enhanced Silt Fence Check*: installed in the same manner as the ALDOT installation, however an 18 in. tall weir is cut into the fabric that extends across the width of the channel bottom. Directly downstream of the weir, a 2 ft wide 8 oz. FF splash apron the length of the weir is installed and covered with ALDOT Class I riprap to dissipate energy of the water overtopping the weir.

(5) *Enhanced ALDOT Pinned Installation*: the ALDOT Installation w/ Pinning is installed, however a weir is cut into the fabric in the same manner as the Enhanced TDOT Installation.

5.6.4 Sand Bag Ditch Check

Typically, sand bags are to be used as ditch checks for hard bottom or stone bottom channels. These ditch checks are comprised of three layers of sand bags stacked on top of each other, placed perpendicular to the flow. The standard ALDOT sand bag ditch check is shown in Figure 5.18.

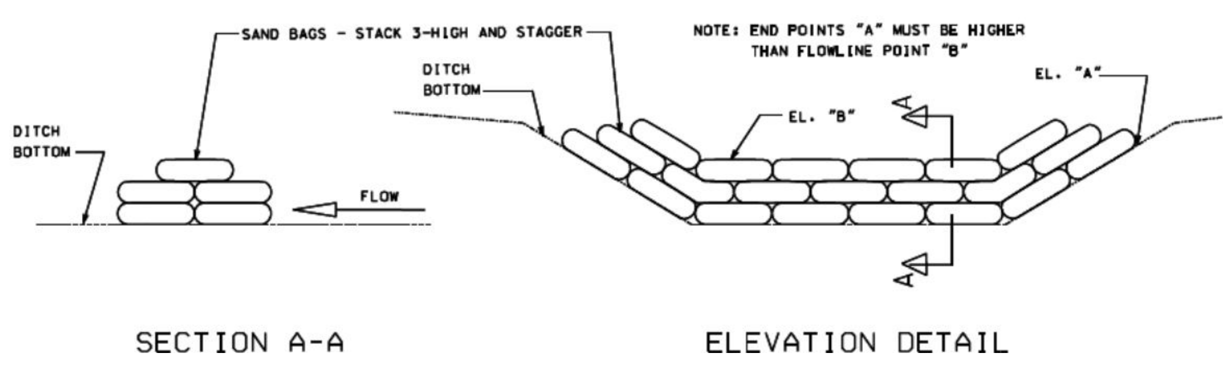


Figure 5.18: ALDOT Sand Bag Ditch Check Installation (ALDOT 2012).

5.6.4.1 Sand Bag Materials for Installation

The following list of materials is required for the sand bag ditch check installations:

- *sandbags*: 18 by 12 by 3 in. woven polypropylene bags, filled with a sandy soil, stacked in a three layer configuration, orientation dependent upon the particular installation, and
- *8 oz. filter fabric*: an 8 oz. filter fabric wrap is used to wrap the entire sand bag ditch check to create a more stable structure.

5.6.4.2 Sand Bag Installation Variations

A series of tier flow, large-scale ditch check experiments were performed to evaluate each sand bag installation configuration. Since sand bag ditch checks are typically used for conditions where low and high flows or velocities are expected, tier flow testing of 0.56 cfs, 1.12 cfs, and 1.68 cfs of flow for ten minutes each for a total duration of 30 minutes was used for testing. For each installation configuration, three replicate tests were to be performed, totaling 9 large-scale experiments. The three installation variations tested included:

(1) *Standard ALDOT*: current ALDOT installation, three layers of bags, the bottom two layers have 2 rows with the top layer having one row. All rows are placed perpendicular to flow with the length of the bag also oriented perpendicular to flow as shown in Figure 5.18.

(2) *ALDOT Sand Bag "Burrito"*: installed in the same manner as the *Standard ALDOT* installation, however filter fabric is placed on the channel bottom, the bags placed on top of the filter fabric, and the FF is wrapped around the bags, encasing the bags in the FF.

(3) *ALDOT Modified*: this installation also uses three layers of sand bags. However, the middle layer sand bags are oriented parallel to the flow direction and 18 extra bags are placed on the downstream side of the ditch check in the channel bottom to help increase ditch check stability.

5.7 Statistical Analysis

Statistical analysis was used to evaluate different wattle installations and products based upon performance and material properties. A multiple linear regression model was used to determine the significance of the installation variables used to enhance the wattle installations. The multiple linear regression model independently evaluates the effect each variable has on increasing the length of impounded water (i.e., length of subcritical flow). The model develops partial regression coefficients that report how strongly that dependent variable (i.e., trenching, stapling, staking, or the underlay) affects the independent variable (i.e., subcritical flow length). The multiple linear regression model used for these analyses is shown in Equation 5.4.

$$f(x) = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \ldots + \beta_n x_n \qquad \text{(Equation 5.4)}$$

where,

$f(x)$ = dependent variable (e.g., subcritical flow length or impoundment length)

x_i = independent variables (e.g., trenching, stapling, staking, or the underlay)

β_i = the ordinary least squares coefficients

Wattle materials and dimensional properties were evaluated to determine performance differences between material properties. The wattles were divided into three groups of excelsior, wheat straw, and synthetics. A one-way analysis of variance (ANOVA) test was used to determine if these different wattle groups performed similarly. This was done by comparing the physical properties of each wattle tested and the resultant hydraulic performance for each flow tier (i.e., impoundment depth measured for each flow tier at CS6). Cross section CS6 was chosen because it is the closest measurement point upstream of the wattle. For all tests performed, CS6 was in subcritical flow, and therefore all CS6 measurements were hydraulically comparable. The ANOVA test is defined by:

$$\mathbf{F = \frac{MST}{MSE}} \qquad \text{Equation 5.5}$$

where,

F = ANOVA coefficient

MST = mean sum of squares due to treatment

MSE = mean sum of squares due to error

$$\mathbf{MST = \frac{SST}{p-1}} \qquad \text{Equation 5.6}$$

$$\mathbf{SST = \sum n\,(x - \bar{x})^2} \qquad \text{Equation 5.7}$$

where,

SST = sum of squares due to treatment

p = total number of populations

n = total number of samples in population

$$\mathbf{MSE = \frac{SSE}{N-p}} \qquad \text{Equation 5.8}$$

$$\mathbf{SSE = \sum (n-1)S^2} \qquad \text{Equation 5.9}$$

where,

SSE = sum of squares due to error

S = standard deviation of the samples

N = total number of observations

Though the ANOVA test can determine if there are differences between material groups, it is not able to determine specifically which groups are different. Therefore a *post hoc* test is conducted in the case that the null hypothesis is rejected and at least one group is statistically significantly different than one or both of the other groups. IBM® SPSS Statistics 21 was used to determine statistical significance using the one-way ANOVA test and a *post hoc* test called least significant difference (LSD) was used to determine which groups were statistically significantly different.

5.8 Summary

This chapter detailed the testing methodology created for improving ditch check installation practices, evaluating ditch check performance, and developing product performance evaluation criteria. Testing the different ditch checks used during construction activities is important for understanding their applications while also developing a database of data to use when comparing manufactured products against standard/modified installations. A process for consistently collecting the velocity, water depth, erosion and deposition patterns have been outlined within this chapter for reference. The results of this testing effort are described in *Chapter Six: Installation Improvements* and *Chapter Seven: Product Evaluation* of this dissertation.

CHAPTER SIX

INSTALLATION IMPROVEMENTS

6.1 Introduction

Ditch checks are used in the construction industry as either permanent or temporary structures constructed across runoff conveyances, intended to slow and impound stormwater runoff, reduce shear stresses causing channel erosion, and create favorable conditions for sedimentation. It is important for the functionality of these devices that they be installed properly and installed in a way that maximizes their performance. This is especially prudent since maximizing performance may reduce channel erosion. Because ditch checks are typically spaced based on geometry (i.e., the height of the ditch check versus the slope of the channel) it is important for ditch checks to impound water efficiently. Figure 6.1 shows spacing requirements for different ditch check heights based upon geometry and channel slope. Often times, however, spacing is based upon policy, rather than geometry. ALDOT, for example, does not specify spacing criteria based upon geometry, and requires all ditch checks to be spaced a minimum of 100 ft for channel slopes up to 5%. This is represented by the dashed line in Figure 6.1.

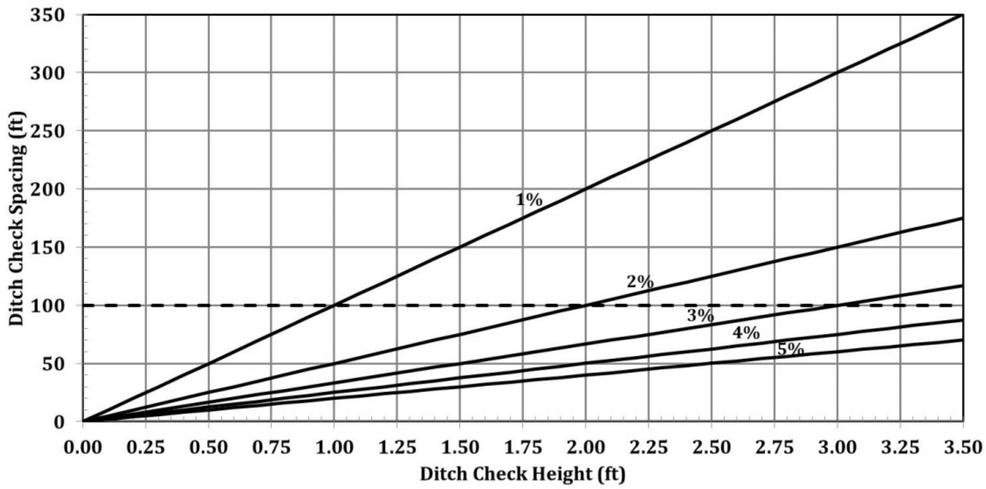
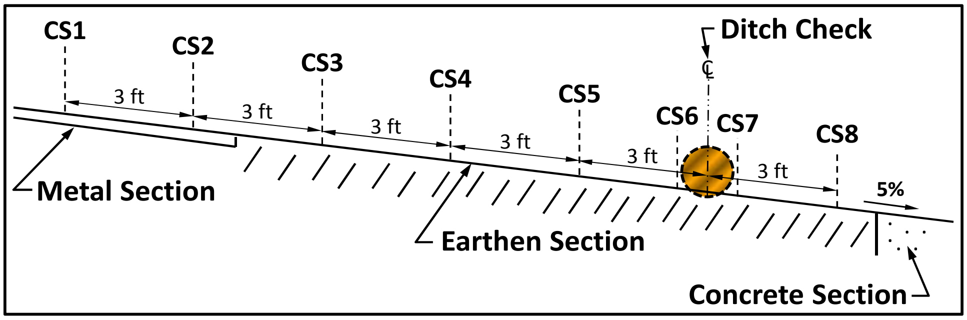


Figure 6.1: Spacing Guide Based upon Channel Slope and Ditch Check Height.

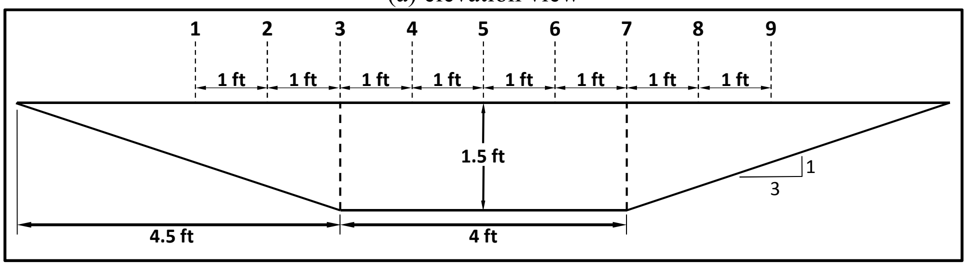
Determining the optimal ditch check installation includes testing the current installation practice, evaluating its performance and determining what improvements could be made, and enhancing the ditch check installations until the maximum performance is achieved. Maximum performance for all the ditch checks typically entails creating the maximum pool length upstream of the ditch check. However, structural stability and maintaining the integrity of the channel are also important aspects of a properly functioning ditch check.

6.2 Bare Soil Control Test

A bare soil control test was performed that consisted of the channel being graded and compacted to experimental specifications without a ditch check installed. This test establishes a baseline for flow velocities and water depths under supercritical flow conditions (i.e., no impedance of flow) at each cross-section (CS1-CS8) as shown in Figure 6.2(a).



(a) elevation view



(b) cross-sectional view

Figure 6.2: Ditch Check Test Channel Dimensions and Configuration.

6.3 Wattle Installation Improvements

Improving the wattle installation first meant identifying a wattle that was representative of the industry practice. Referencing ALDOT's List II-24 for approved erosion and sediment controls lists five of the eleven products on that list as wheat straw wattles. The ALDOT ESC-300 wattle detail calls for a 20 in. by 20 ft wattle. The only product marketed as a 20 in. by 20 ft. wattle was the Western Excelsior EXCEL Straw Logs. This products manufactured specifications is shown in Table 6.1. This product was chosen to be used for modifying the standard installation.

Table 6.1: Western Excelsior EXCEL Straw Log Manufacturer Material Specifications

Product	***Diameter (in.)***	***Length (ft)***	***Density (lbs/ft^3)***	***Fill Material***	***Netting***
Excel Straw Log	20	20	2.4	Straw	½ in. by ½ in. synthetic

6.3.1 Wattle Installation Descriptions

The seven installations tested improving wattle installations are as follows:

(1) *Downstream Staking*: current ALDOT installation, wattle is placed across the channel in a U-shape, concave upstream, and secured with wooden stakes driven into the ground a minimum of 1.5 ft and positioned every 2 ft on the downstream side of the wattle piercing the netting. [Figure 6.3(a)]

(2) *Teepee Staking*: mimicked NCDOT staking practices creating a “teepee” or A-frame over the wattle by driving the stakes into the ground a minimum of 1.5 ft next to the wattle without piercing the wattle or wattle netting. These stakes were driven in at an angle towards the wattle securing the wattle in place. Two stakes were installed upstream and five stakes installed downstream with a stake spacing of 2 ft. [Figure 6.3(b)]

(3) *Downstream Staking w/8 oz. FF*: wattle was installed with an 8 oz. FF underlay and secured in place using ALDOT staking practices. [Figure 6.3(c)]

(4) *Teepee Staking w/8 oz. FF*: wattle was installed with an 8 oz. FF underlay and secured in place following NCDOT staking practices. [Figure 6.3(d)]

(5) *Downstream Staking w/Trenching*: entire width of the wattle was trenched into channel 2 in. deep, perpendicular to the flow of water and anchored using ALDOT staking practices. [Figure 6.3(e)]

(6) *Teepee Staking w/8 oz. FF and Trenching*: a 2 in. deep trench extending the entire width of the wattle was excavated and covered with the 8 oz. FF underlay. The wattle was installed and secured using NCDOT staking practices. [Figure 6.3(f)]

(7) *Teepee Staking w/8 oz. FF + Staples*: wattle was installed exactly as described in configuration (4), also securing the bottom of the upstream and downstream face of the

wattle to channel using sod staples along each side, spaced 12 in. apart to improve contact with the channel bottom. [Figure 6.3(g)]

The downstream staking is the standard staking method used by ALDOT. This staking pattern is considered a destructive method because it breaks the netting by piercing it with the stake which can reduce the life expectancy of the wattle. The nondestructive, *teepee* staking used by the NCDOT. The filter fabric underlay that was used to minimize scour at the ditch check and channel interface. Note also that staples were used for the *Teepee Staking w/8 oz. FF + Staples* installation.


(a) control


(b) downstream staking

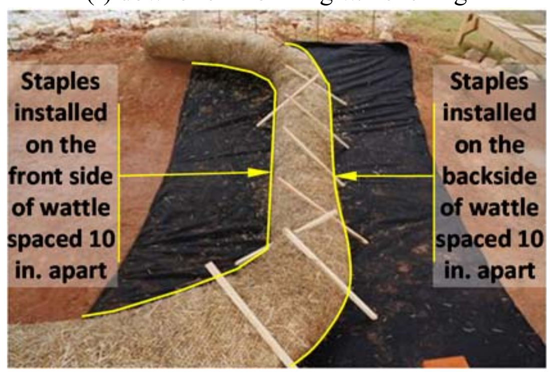

(c) teepee staking


(d) downstream staking w/8 oz. FF

(e) teepee staking w/8 oz. FF

(f) downstream staking w/trenching

(g) teepee staking w/8 oz. FF + trenching



(h) teepee staking w/8 oz. FF + staples

Figure 6.3: Control and Seven Wattle Installations Tested.

6.3.2 Wattle Installation Improvement Results and Discussion

The following section is a summary of the results and comparisons that were made from the experiments using a 0.56 cfs constants flow rate for all large-scale tests performed. Table 6.2 shows the comparative results of the various wattle installation configurations and the bare soil control.

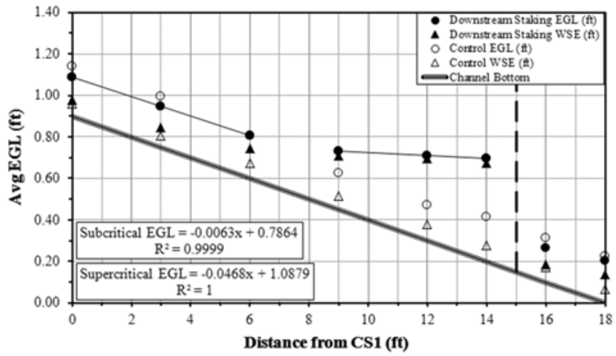
Table 6.2: Comparative Results of Each Wattle Installation Configuration and the Control

Treatment	Length of Subcritical Flow (Impoundment)		Energy Grade Line Slopes (ft/ft)	
	Length (ft)	Percent Difference(%)[a]	Based on ASTM D7208[b]	Subcritical Flows Only
Teepee Staking w/8 oz. FF + Staples	20.5	99.0	-0.0166	-0.0166
Teepee Staking w/8 oz. FF	16.5	60.2	-0.0250	-0.0250
Downstream Staking w/8 oz. FF	15.0	45.6	-0.0302	-0.0210
Teepee Staking	10.7	3.9	-0.0197	-0.0060
Downstream Staking	10.3	--	-0.0277	-0.0063
Downstream Staking w/Trenching	9.0	-12.6	-0.0457	-0.0250
Teepee Staking w/8 oz. FF + Trenching	8.0	-22.3	-0.0275	-0.0077
Bare Soil Control	N/A	N/A	-0.0514	-0.0514

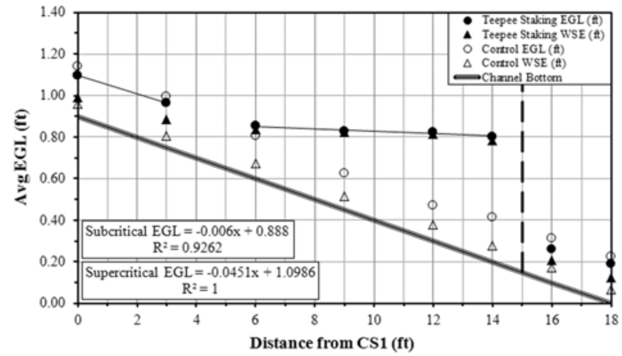
Notes:
[a] Percent increase/decrease in comparison to the *Downstream Staking* installation.
[b] ASTM D7208-06 EGL slope was a single linear trend line through all EGL points upstream the wattle (including both supercritical and subcritical flow).

The current ALDOT installation practice, referred to as *Downstream Staking*, was tested and compared to the nondestructive *Teepee Staking* pattern used by NCDOT. Data analysis determined that staking pattern had little effect on the average subcritical flow length when comparing the *Downstream Staking* pattern of 10.3 ft to the *Teepee Staking* pattern of 10.7 ft. Visual documentation noted that during testing, for both staking patterns, a maximum impoundment length was achieved early, then receded to a shorter steady-state subcritical flow length as the test continued due to excessive undercutting and piping occurring at the interface of the wattle and channel bottom. To prevent the piping effect, the teepee and downstream staking were tested using an 8 oz. filter fabric underlay that was intended to protect the channel bottom at the wattle installation. The data collected shows that the *Teepee Staking w/8 oz. FF* installation increased subcritical flow length to 16.5 ft in comparison to the previously discussed *Teepee Staking* installation. The *Downstream Staking w/8 oz. FF* installation also increased subcritical flow length to 15 ft when compared to the *Downstream Staking* installation. Note however, that though the FF increased the subcritical flow length for both installations, both subcritical flows lengths were once again similar (i.e. 16.5 ft for *Teepee Staking w/8 oz. FF* and

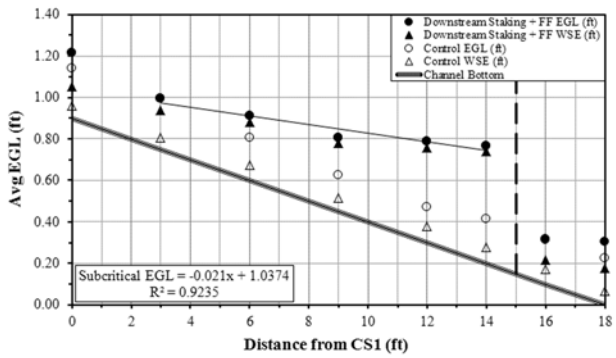
15 ft for *Downstream Staking w/8 oz. FF*). These installations are each compared to the control (i.e., no wattle installation) in Figure 6.4. The EGL and water surface elevation (WSE) are plotted for each. In Figure 6.4(a) and Figure 6.4(b), there are two EGLs plotted for each installation. These two EGLs are a result of two different flow conditions (i.e. supercritical or subcritical) that fell within the measurement cross-sections. The upstream EGLs represent supercritical flow. These supercritical EGL points are above the WSE points and indicate higher kinetic energy from greater flow velocity. However, the downstream subcritical flow EGLs show less kinetic energy since the EGL points fall almost directly on top of the WSE points indicating impoundment of flow. This decrease in kinetic energy is the ideal circumstance for channel protection. This impoundment length of subcritical flow increases with the inclusion of the filter fabric underlay.



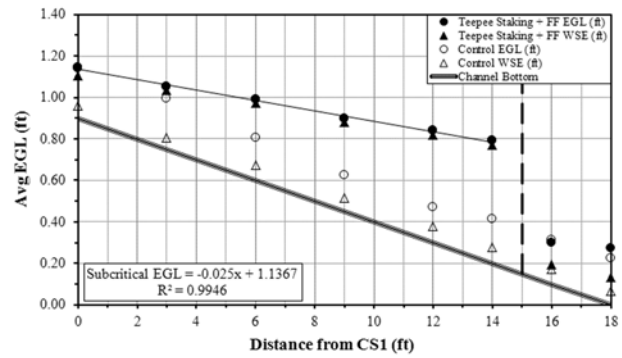
(a) downstream staking vs. control



(b) teepee staking vs. control



(c) downstream staking w/ FF vs. control



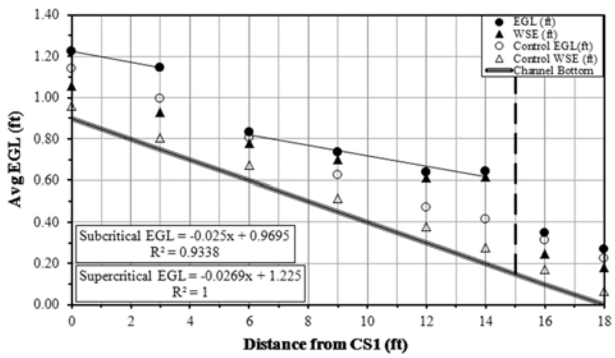
(d) teepee staking w/ FF vs. control

Figure 6.4: Comparison of Installations Energy Grade Line (EGL) and Water Surface Elevation (WSE) to the Control Test with No Wattle Installed.

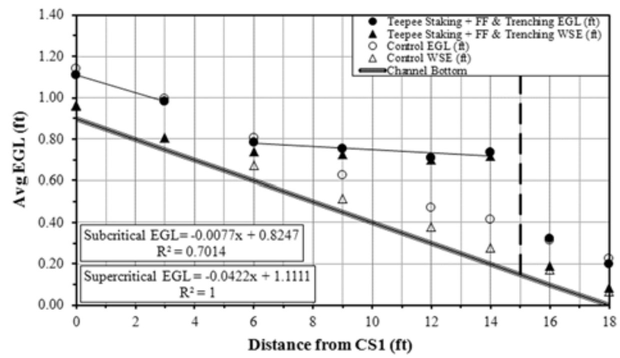
ALDOT and many manufacturers recommend trenching the wattle if piping is a concern. This installation was tested using the *Downstream Staking w/Trenching* installation and resulted in a decrease in impoundment length with an average subcritical flow length of 9 ft which was 1.3 ft shorter than the *Downstream Staking* installation. Visual documentation also observed piping and scour under the wattle, along with higher amounts of erosion occurring on the downstream side of the wattle due to the trench being washed out as shown in Figure 6.5(a). Anticipating better performance by once again using the FF underlay, the *Teepee Staking w/8 oz. FF and Trenching* was also tested and shown in Figure 6.5(b). However, trenching with FF did not increase performance; rather the average subcritical flow was reduced to 8 ft long compared to the *Teepee Staking w/8 oz. FF* impoundment of 16.5 ft long. This seems to suggest that trenching reduces the wattle's ground contact with the channel bottom, allowing an easier path for water to pass under the wattle.

(a) downstream staking + trenching

(b) teepee staking + trenching and FF



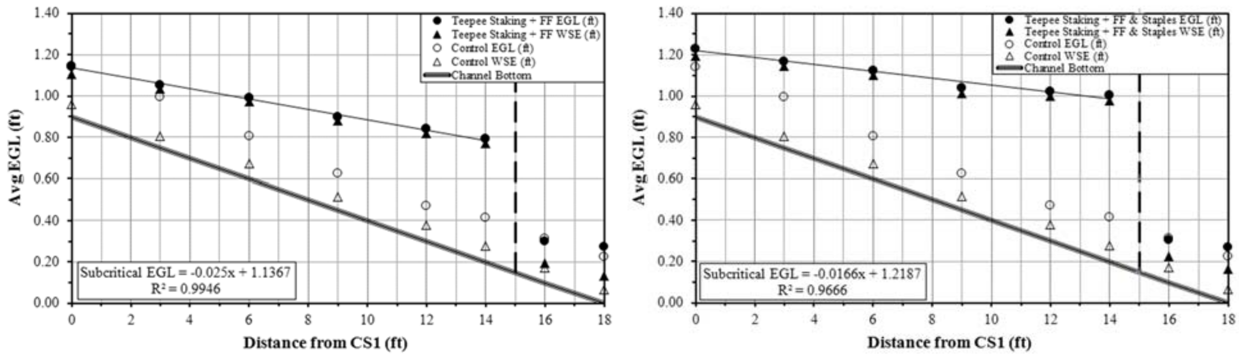
(c) downstream staking + trenching vs. control



(d) teepee staking + trenching and FF vs. control

Figure 6.5: Test Comparison of Trenched Wattle Configurations.

The final installation tested, *Teepee Staking w/8 oz. FF + Staples,* mimics the NCDOT's wattle detail described in the Methodology chapter. This installation uses a filter fabric underlay, teepee staking, and 12 in. sod staples anchoring the wattle to the channel which may improve ground contact and minimize undercutting. This installation resulted in an average subcritical flow length of 20.5 ft. Figure 6.6 shows the hydraulic results of this test compared to the *Teepee Staking w/8 oz. FF* installation. The inclusion of staples to increase ground contact appears to successfully improve wattle performance as evident by the increase of subcritical flow length and visual observations. Because the sod staples increased ground contact, undercutting was reduced and increased flow was visually noted as flowing through the wattle instead of under. This assumption was further verified by statistical analyses.



(a) teepee staking w/ FF vs. control (b) teepee staking w/ FF + staples vs. control

Figure 6.6: Installation Comparison with and without Staples.

The impoundment lengths as well as the EGL slopes are tabulated in Table 6.2. ASTM D7208-06 says to determine the EGL by fitting a regression line through EGL elevation points determined at each cross-section (ASTM 2006). No further guidance for interpreting or analyzing the data is given. This could be problematic if a hydraulic jump is within the measurement cross-sections since steady-state supercritical EGL slopes typically closely match the channel slope while the subcritical EGL slope is flattened out by the impoundment caused by the wattle. Using a single trend-line to mimic the EGL slope across the hydraulic jump would make it inaccurate since the supercritical flow EGL is more affected by the water velocity while the subcritical flow EGL is most affected by WSE or water depth. The only installations that resulted in the hydraulic jump extending beyond the measurement threshold was *Teepee Staking w/8 oz. FF* and *Teepee Staking w/8 oz. FF +Staples* (Figure 6.6). The bare soil control is all supercritical flow. However, evaluating installations based on subcritical flows only can also be problematic because the shorter pool lengths typically have EGL slopes approaching zero (Figure 6.4(a) and (b), Figure 6.5 (d)). Longer impounding EGL slopes tend to be steeper sloped which is evident when comparing longer subcritical flows to shorter subcritical flows as shown in Table 6.2. The EGL and WSE should mimic impoundments such as dammed reservoirs or sluices and should have small slopes along the flow direction; instead of the steeper impoundment slopes shown in Figure 6.6. This anomaly may be caused by the complex flows (e.g., three dimensional flow circulations observed during testing) created by undercutting and the wattles porous material. This could be further investigated in a future study.

6.3.3 Statistical Analysis Results

A multiple linear regression model was used to determine the effect of the different installation configurations on overall wattle performance. Evaluating each installation component was particularly important since the overarching goal was to increase the impoundment length in order to achieve maximum ditch check performance. Each of the installations was classified by different combinations of the independent variables considered in the analysis: (1) trenching, (2) underlay, (3) downstream staking, (4) teepee staking, and (5) stapling. This multiple linear regression model independently evaluates the effect each variable has on increasing the length of impounded water (i.e., length of subcritical flow). The model develops partial regression coefficients that report how strongly that dependent variable (i.e., trenching, stapling, staking, or the underlay) affects the independent variable (i.e., subcritical flow length). The multiple linear regression model used for these analyses is shown in Equation 6.1.

$$f(x) = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + ... + \beta_n x_n \qquad \text{(Equation 6.1)}$$

where,

$f(x)$ = dependent variable (e.g., subcritical flow length or impoundment length)

x_i = independent variables (e.g., trenching, stapling, staking, or the underlay)

β_i = the ordinary least squares coefficients

Using this model, the most effective means of increasing the subcritical flow length can be determined. The downstream staking pattern was used as the analysis base, from which all other installation components are compared. This was selected because a popular industry practice is simply staking the wattle with downstream staking only. The results of this analysis along with corresponding p-values are shown in Table 6.3.

Table 6.3: Statistical Relationships of Installation Components

Installation Component	Statistical Significance	
	Coefficients	**p-value[a]**
Downstream Staking	Base	N/A
Teepee Staking	-0.833	0.389
Filter Fabric Underlay	3.500	0.002
Trenching	-4.667	< 0.001
Stapling	5.583	0.001

Notes: [a] comparison to effects of *Downstream Staking* at 99% confidence interval and p-values < 0.01.

Compiling all this information results in the following conclusions based on statistical significance of the model: (1) because the coefficient for staking is not statistically significant, we can conclude that the staking pattern does not significantly affect the performance of the installation for increasing subcritical flow length, (2) trenching the wattle has a significantly detrimental effect on performance, as evidenced by the negative coefficient, and (3) the underlay and stapling significantly improve performance by increasing the subcritical flow length.

6.3.4 Wattle Installation Improvements Conclusions

Evaluating the installations requires determining the greatest mitigating factor that defines the wattles performance as a ditch check. The slope of the EGL is plotted to evaluate the energy reduction of the experimental flow, as kinetic energy (i.e., $v^2/2g$ of the supercritical flow) changes into potential energy (i.e., WSE of the subcritical flow) by the ditch check. However, recognizing that increased impoundment length means increased subcritical flow is also relevant for determining performance. For channelized flow, reducing the erosive forces caused by super critical flows in an earthen channel while also prompting sediment deposition in the subcritical flow area is the ideal scenario. This can be accomplished by maximizing the subcritical flow length, therefore minimizing highly erosive supercritical flows.

The multi linear regression model showed that trenching, stapling, and underlay did significantly affect wattle performance with trenching being detrimental to performance and stapling and underlay improving performance. It should also be noted that trenching causes greater erosion downstream and may actually increase the effects of undercutting. Therefore taking the statistical significance into consideration while also looking at the largest increase in subcritical flow length, it is the recommendation of this study that the *Teepee w/ 8 oz. FF + Staples* installation be used to install 20 in. diameter wheat straw wattles as ditch checks for maximum stormwater control performance. This recommendation was presented to ALDOT as a means for maximizing wattle performance as ditch checks, however, ALDOT determined that using staples as a means to secure the wattle in place would be difficult to enforce through inspections. Therefore, ALDOT adopted the *Teepee w/ 8 oz. FF Underlay* as the new wattle installation. This has been added to the ESC 300 Temporary Erosion and Sediment Controls detailed drawings and is shown if Figure 6.7.

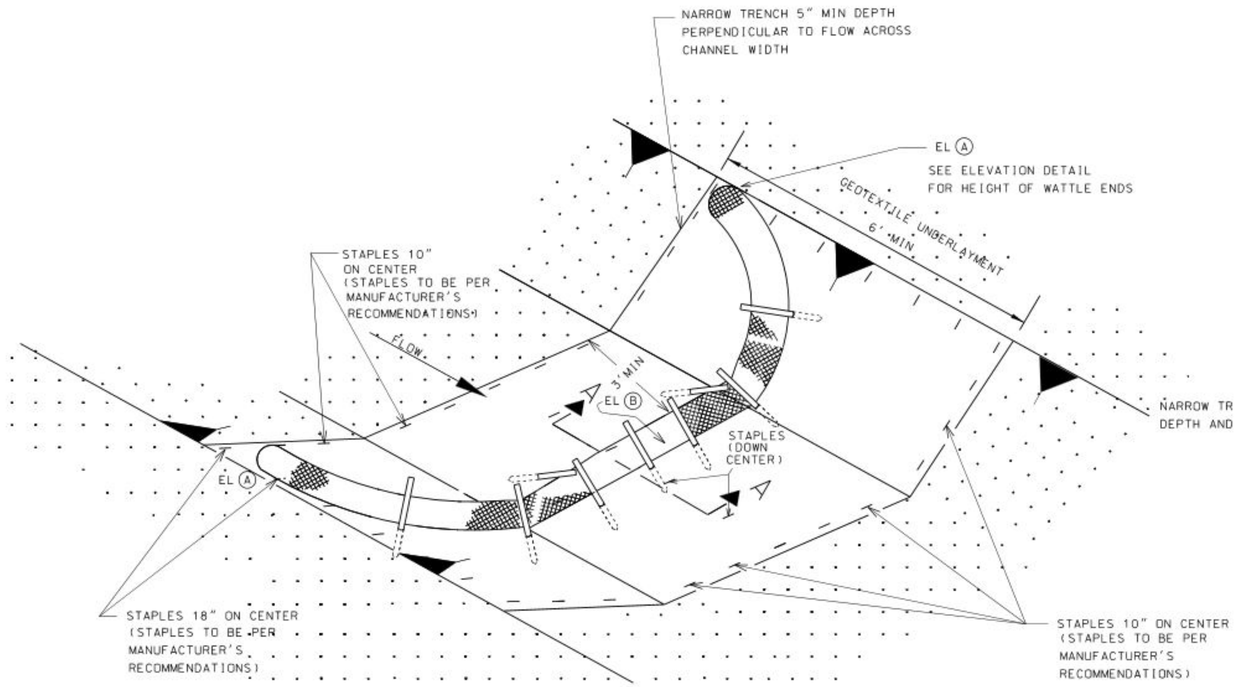


Figure 6.7: Adopted ALDOT Wattle Installation Detail (ALDOT 2012).

6.4 Riprap Installation Improvements

Rock ditch checks are typically comprised of one or more classifications of aggregate which must be large enough to withstand velocities of concentrated, channelized stormwater runoff while also impounding water. The Alabama Handbook for Erosion Control, Sediment Control and Stormwater Management on Construction Sites and Urban Areas Vol. 1 requires the use of ALDOT Class I Riprap with a geotextile underlay to protect the channel from undercutting and piping (ASWCC 2009). The spacing of ditch checks if they are to be used to maximize performance and minimize erosion, such as riprap ditch checks, are to be no greater than the length of the ditch check's maximum pool length which protects the greatest amount of channel as shown in Figure 6.8.

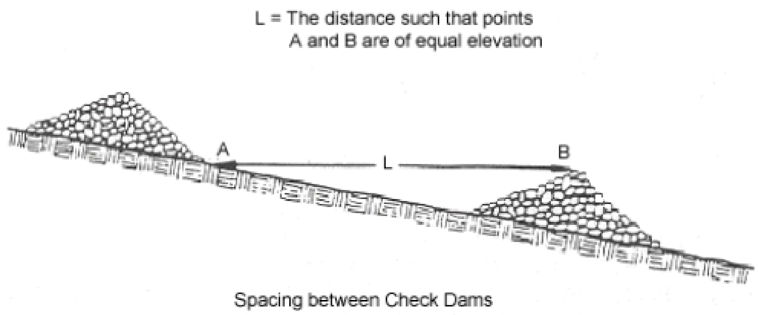


Figure 6.8: Alabama Handbook Recommended Ditch Check Spacing.

This spacing requirement is based upon ditch check height and channel slope. Shallower channels will allow greater ditch check spacing while steeper channels require shorter spacing for ditch checks of the same height. Also, taller ditch checks will require longer spacing when compared to shorter ditch checks used for the same longitudinal slope. This concept is shown in Figure 6.1. ALDOT places a minimum ditch check spacing of 100 ft and is referenced in Figure 6.1 (ALDOT 2012). The following installations were tested to determine

(1) *No Choker*: Class I ALDOT riprap is constructed per ALDOT riprap ditch check detail w/FF underlay extending 3 ft from the upstream toe and downstream toe of the ditch check. The middle of the ditch check is 1.5 ft high with a base width of 6 ft. The 8 oz. nonwoven FF underlay is pinned in the same manner used in the wattle ditch check underlay pinning method.

(2) *Modified no. 4 Stone Choker*: installed in the same manner as the *No Choker* installation, however, a layer of modified no. 4 stone was layered across the upstream side of the ditch check to minimize flow-through.

(3) *Filter Fabric Choker*: installed in the same manner as the *No Choker* installation, however, an 8 oz. nonwoven filter fabric choker is placed across the front face of the ditch check and 3 ft upstream of the toe to further minimize flow-through and undermining.

6.4.1 Riprap Ditch Check Practice and Modifications

Riprap ditch checks are typically reserved for flows with high flow rates or high velocity. This is because ditch checks are typically viewed as temporary devices and riprap ditch checks normally are not removed upon project completion, leaving large rock piles in the right of way that becomes hazardous to out-of-control vehicles and maintenance crews that mow the right of ways. However, when riprap ditch checks are needed, it is important to understand how they will perform based upon the installation utilized. Choking is a method used to decrease flow-through and increase impoundment lengths by means of blocking the flow passages between the rocks. This can be done by adding a smaller gradation stone or a fabric cover to the upstream face of the riprap ditch check. ALDOT specifies that smaller aggregate or filter fabric may be used for choking. In order to understand how these two choking practices affect the ditch check installation, a riprap installation without a choker must first be evaluated. These three installations are shown in Figure 6.9.

(a) Riprap ditch check w/no choker

(b) Riprap ditch check w/no. 4 coarse aggregate

(c) Riprap ditch check w/8 oz. FF choker (d) 8 oz. FF choker secured w/riprap

Figure 6.9: Riprap Ditch Check Installation Configurations.

6.4.2 Riprap Installation Improvement Results and Discussion

The following section is a summary of the results and comparisons that were made from the experiments using a 1.7 cfs constant flow rate for all large-scale tests performed. Table 6.4 shows the comparative results of the various riprap installation.

Table 6.4: Comparative Results of Each Riprap Installation Configuration

Installation Type	Avg. Pool Length (ft)	% Difference[(1)]	% Efficiency[(2)]
Riprap w/No Choker	14.5	N/A	48.3%
Riprap w/no. 4 Coarse Aggregate	20.5	41.4%	68.3%
Riprap w/8 oz. FF Choker	29.1	100%	97%

(1) % difference w/respect to riprap w/no choker installation
(2) % efficiency refers % of spacing required for ditch check based on 1.5 ft high ditch check for 5% slope in Figure 6.1

The Riprap w/no Choker installation shown in Figure 6.9(a) allows flow to easily pass through the ditch check decreasing possible impoundment length when compared to the two ditch check installations with chokers. Adding modified no. 4 coarse aggregate to the installation as shown in Figure 6.9(b) decreased the flow-through capabilities of the ditch check which therefore increased the impoundment length from 14.5 to 20.5 ft, an increase of 41.4%. Using the 8 oz. FF choker instead of the modified no. 4 coarse aggregate choker, as shown in Figure 6.9(c), further increased the pool length from 14.5 to 29.1 ft, an increase of 100%. A secondary benefit of using the filter fabric choker is that the fabric restricts flow through the ditch check and causes it to pass over and down the downstream slope of the ditch check as shown in Figure 6.9(d). This decreases the velocity of the water as it resumes down the ditch whereas water that flows through the ditch check finds the path of least resistance which are typically small passages which increase water velocity, creating a nozzle affect. Another secondary benefit of the filter fabric choker is when maintenance action is required due to an accumulation of sediment in front of the upstream face of the ditch check, the filter fabric choker can be cut away from the underlay and replaced and re-pinned increasing the longevity of the ditch check.

Referring back to Figure 6.1 and Table 6.4, it should be noted that the Riprap w/8 oz. FF Choker installation nearly reaches the expected ditch check impoundment length of 30 ft for an

18 in. tall ditch check in a 5% sloped channel. This means the ditch check was performing at nearly 100 % efficiency for creating the greatest impoundment length.

(a) riprap ditch check w/no choker

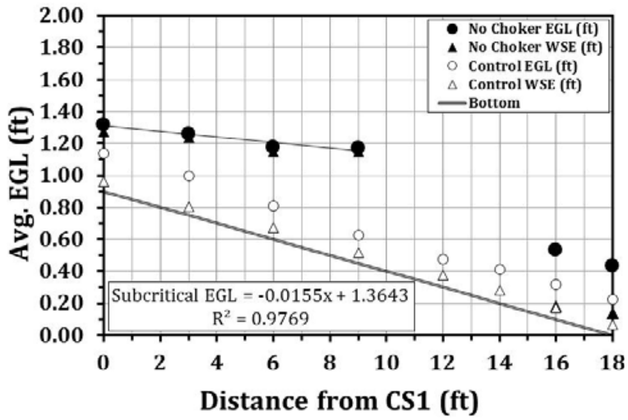
(b) riprap ditch check w/no. 4 coarse aggregate choker

(c) riprap ditch check w/8 oz. FF choker

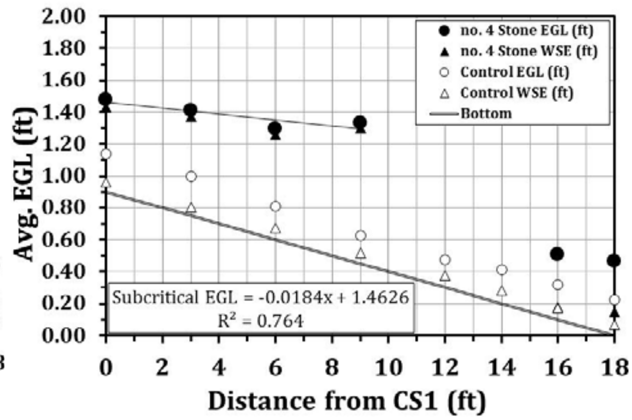
(d) energy dissipation w/ 8 oz. FF choker

Figure 6.10: Average Performance of Each Riprap Ditch Check Installation.

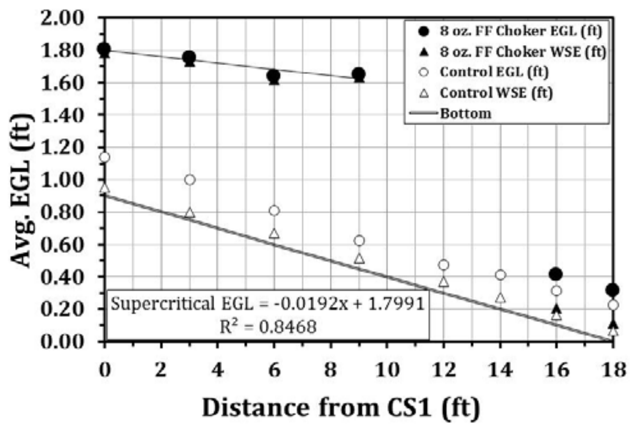
The slope of the EGLs for each installation is shown in Figure 6.11. The graphs depict each average EGL and WSE for all the installation replications. The WSE elevation as well as the EGL increase as the choker restricts more flow which therefore causes increased impoundment lengths.



(a) riprap ditch check w/no choker EGL and HGL



(b) riprap ditch check w/no. 4 coarse aggregated choker EGL and HGL



(c) riprap ditch check w/8 oz. FF choker EGL and HGL

Figure 6.11: Riprap Ditch Check Choker Comparisons.

6.4.3 Riprap Installation Improvements Conclusions

Riprap ditch checks are typically used in high flow, high velocity conditions due to their structural stability to withstand high velocity and flow forces. However, due to the large pores created by the aggregate shape, water has greater passages available for water to pass through the ditch check rather than over which results in decreased impoundment lengths. Choking these pore passages is a means to mitigate this issue. ALDOT ESC 300 recommends choking with aggregate to decrease flow through. Both the modified no. 4 coarse aggregate and 8 oz. FF to were tested to determine which choker creates the longest impoundment length. This study has determined that choking the ditch check with 8 oz. FF is a better means of flow impoundment and resulted in an impoundment length of 29.1 ft which was a 100% increase in flow length and a 97% impoundment efficiency when compared to the riprap ditch check with no choker which impounded water14.5 ft at 48.3% impoundment efficiency. Modified no. 4 coarse aggregate impounded flow 20.5 ft which is a 41% increase in impoundment length at a 68.3% impoundment efficiency when also compared to the riprap ditch check with no choker installation.

6.5 Silt Fence Installation Improvement

Silt fences are typically used as perimeter controls for construction sites. These are barriers installed down gradient of disturbed areas and are typically meant to intercept sheet flow. A properly functioning silt fence when used as a sheet flow interceptor is installed at the same grade across the gradient so that flow is evenly distributed across the width of the silt fence. The silt fence is trenched-in by digging a 6 in. by 6 in. trench and compacting soil on top of the trenched-in fabric or by using a silt fence installation machine that slices the ground open, pushes the fabric into the ground, and pushes the slice back together. If undercutting is prevented by properly installing the silt fence, then flow will impound up gradient. As sediment deposits on the fence, flow-through is restricted and the water level increases. This scenario could become problematic if a portion of the silt fence is installed down gradient of the rest of the fence, creating a low point and therefore a point of concentration. Often when this occurs, the flow overtops at the low point once the fence reaches volumetric capacity for that low point, creating a scour area on the down gradient side. This can also cause the fence to push over and release the impounded flow.

Even though silt fence tends to have structural integrity problems when flows are concentrated to one point, silt fence is used in drainage channels as ditch checks which exposes the barrier to the greatest flow concentrations created on the construction site. Silt fence ditch check can also be pushed over in the same manner as silt fence used as flow interceptors if improperly installed and the flow is concentrated to one point along the fence as shown in Figure 6.12.

(a) perimeter control silt fence (b) silt fence ditch check

Figure 6.12: Silt Fence Structural Failures.

Due to this failure concern, the structural integrity of silt fence ditch checks should be evaluated. Improving structural integrity and minimizing failure possibilities should reduce the maintenance requirements and increase the longevity of the practice.

6.5.1 Silt Fence Installation Improvements Results and Discussion

The following section is a summary of the results and comparisons that were made from the experiments using the constant flow of 0.56 cfs for 30 minutes or tier flow test regime of 0.56 cfs, 1.12 cfs, and 1.68 cfs for ten minutes each for a total test duration of 30 minutes for all large-scale tests performed. Each ditch check installation was required to pass three replicate tests.

The following installations were tested to determine overall installation performance improvement.

(1) *Standard V-Installation*: mimics the ALDOT and other SHAs installation, center post placed in the channel centerline, posts are spaced 2.5 ft on center. Fabric and wire backing is trenched in a 6 in. by 6 in. trench. Overall fence height is 32 in.

(2) *V-Installation w/ Hay Bale Dissipater*: installed in the same manner as the ALDOT installation, however hay bales a placed downstream, abutted to the fence in an attempt to dissipate energy and reduce downstream scour next to the fence.

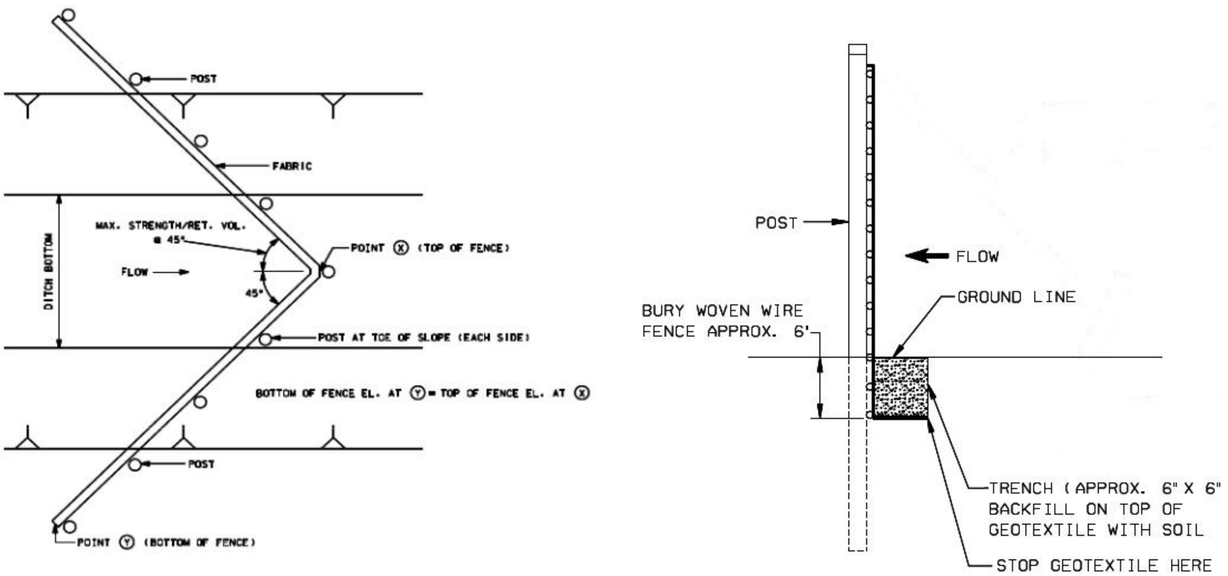
(3) *V-Installation w/ Modified No. 4 Stone Dissipater*: installed in the same manner as the ALDOT installation, however modified no. 4 stone is placed downstream, in an attempt to dissipate energy and reduce downstream scour next to the fence.

(4) *TDOT Enhanced Silt Fence Check*: installed in the same manner as the ALDOT installation, however an 18 in. tall weir is cut into the fabric that extends across the width of the channel bottom. Directly downstream of the weir, a 2 ft wide 8 oz. FF splash apron the length of the weir is installed and covered with ALDOT Class I riprap to dissipate energy of the water overtopping the weir.

(5) *Enhanced ALDOT Pinned Installation*: the ALDOT Installation w/ Pinning is installed, however a weir is cut into the fabric in the same manner as the Enhanced TDOT Installation.

6.5.1.1 ALDOT Silt Fence Installation Evaluation

The standard ALDOT silt fence is classified as 3.5 oz. per yd^2 nonwoven filter fabric, supported by wire backing and attached to 5 ft. tall metal t-posts. This installation detail is shown in Figure 6.13.



(a) standard ALDOT ditch check installation (b) standard ALDOT silt fence installation

Figure 6.13: Standard ALDOT Silt Fence Ditch Check Detail (ALDOT 2012).

The height of the installed silt fence is 32 in. which is at least a foot taller than the 20 in. nominal height of wattle ditch checks and over a foot taller than the rip rap and sand bag ditch checks. Therefore, the height of the silt fence will result in impounding more water and creating longer impoundment lengths. However this extra height will also increase the hydrostatic pressure that the installation must endure under higher flow conditions.

If the height of the silt fence is greater than the depth of the channel, and if the silt fence is expected to impound water the entire depth of the fence, then the fence must be extended out of the channel and back up gradient, parallel with the channel as shown in Figure 6.14(a).

(a) standard ALDOT installation w/ extended sides (b) full pool w/ flow overtopping channel banks

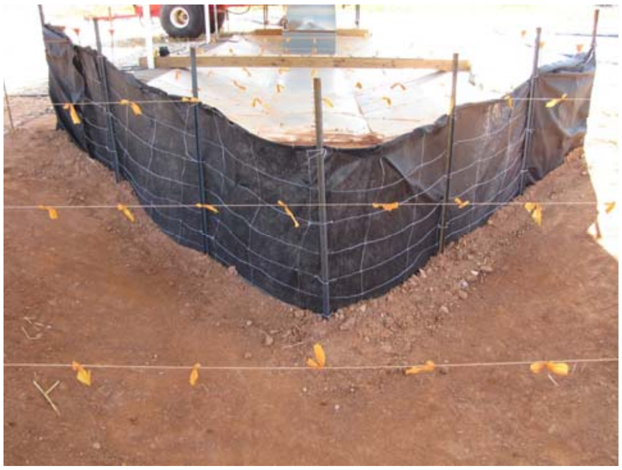
(c) silt fencefull pool length (d) silt fence full pool depth

Figure 6.14: Standard V-Installation Silt Fence Ditch Check Tier Flow Test.

The impoundment condition shown in Figure 6.14 is not an ideal impoundment condition due to the risk of catastrophic consequences if the structure fails at full pool. The impounded water would be released downstream creating a moving bore which could possibly cause a chain reaction where other downstream ditch checks could also fail.

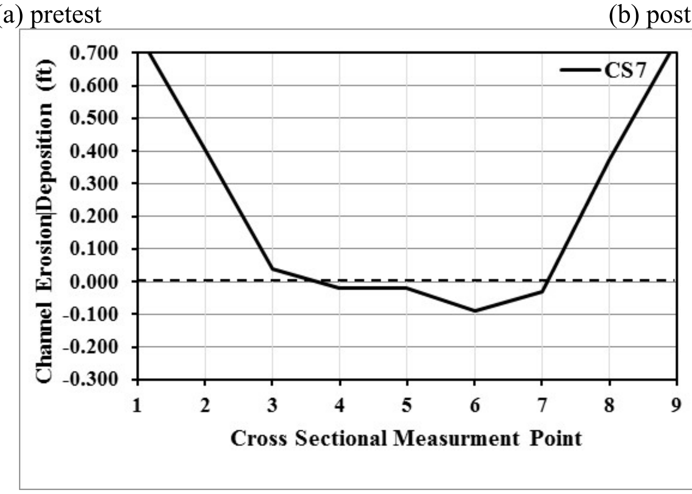
6.5.2 Downstream Dissipation Installation Evaluation

Since structural failure is the major concern for the silt fence ditch check, evaluation of the V-installation required investigation. This installation was tested to determine possible structural failure modes. Silt fence installations typically degrade over time as water passes through or over the filter fabric and creates scour points directly downstream. These scour points can erode around the middle t-post reducing the structural integrity of the installation and cause the fence to eventually fall over. These scour patterns became evident after testing the standard ALDOT installation under low flow conditions of 0.56 cfs. This is shown in Figure 6.15.



(a) pretest

(b) post test

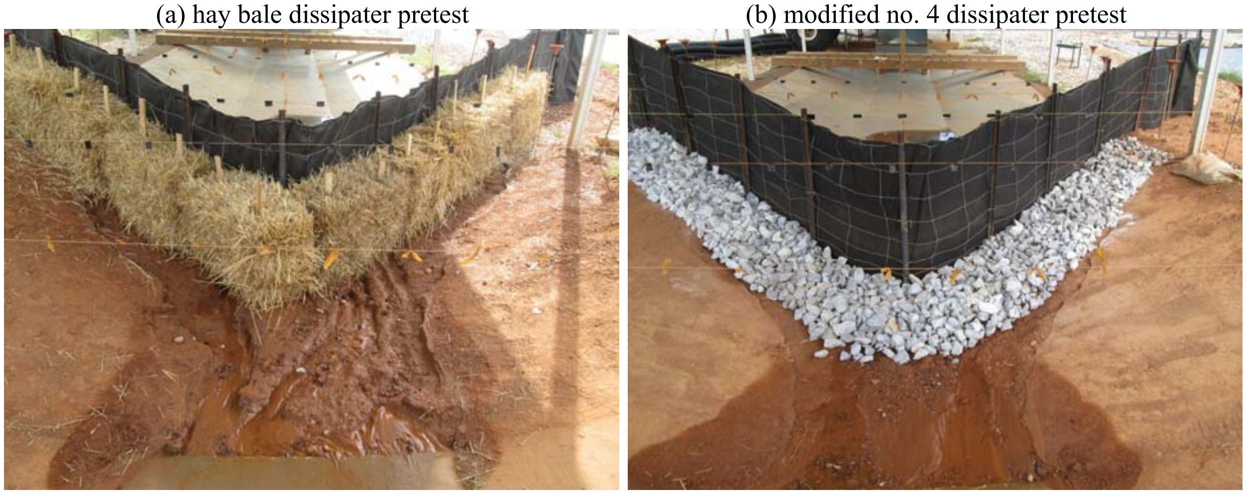


(c) downstream erosion pattern

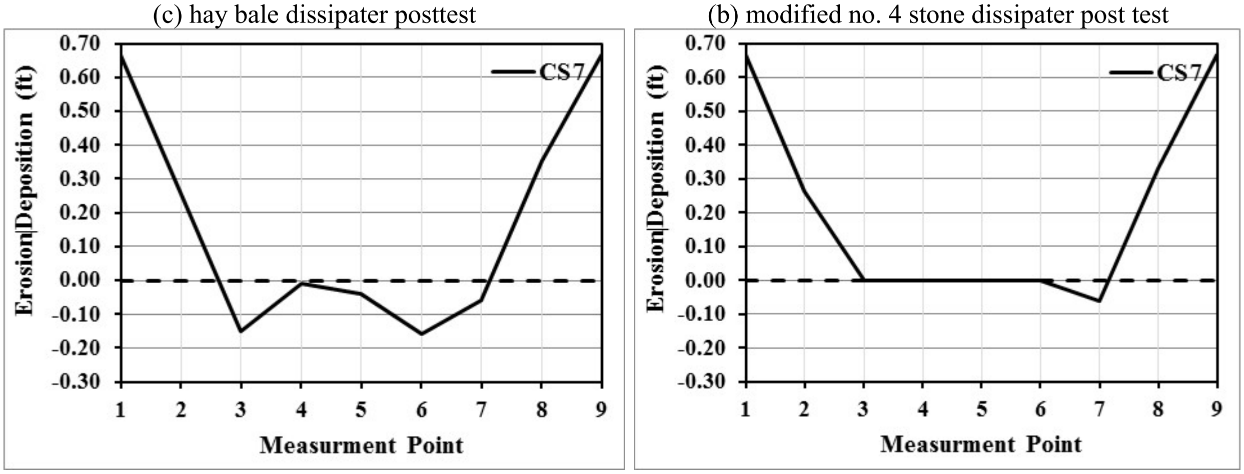
Figure 6.15: Standard ALDOT Silt Fence Installation.

Figure 6.15(c) shows that up to 0.1 ft of erosion occurred directly downstream of the ditch check after one 30 minute test at 0.56 cfs. Due to this, an installation modification directed towards reducing this downstream scour was tested. The proposed modifications included using rock or hay bales positioned directly downstream of the silt fence, to be used as flow dissipation and channel protection. These modifications are shown in Figure 6.16.

(a) hay bale dissipater pretest (b) modified no. 4 dissipater pretest



(c) hay bale dissipater posttest (b) modified no. 4 stone dissipater post test

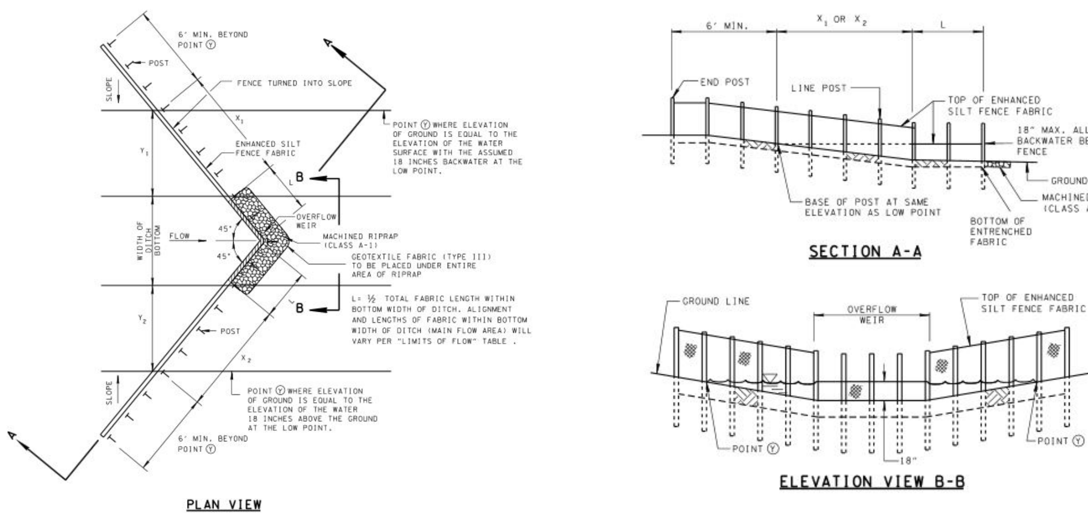


(e) hay bale dissipater erosion pattern (f) modified no. 4 erosion pattern

Figure 6.16: Downstream Dissipation Modifications for V-Installation Silt Fence.

The inclusion of downstream dissipation measures did not improve the erosion patterns for the channel. However, the modified no. 4 stone dissipater did manage to move the erosion away from the downstream side of the fence as shown in Figure 6.16. Once beyond the dissipaters, the flow is able to resume supercritical flow velocities causing downstream erosion. Erosion occurred beneath the hay bales, but the rock reduced the erosion directly downstream of the fence. It should be noted that using a rock dissipation method may not be cost effective and would create a need to remove the rock which increases the overall cost of the practice.

Tennessee Department of Transportation (TDOT) enhanced silt check installation was also evaluated. This installation uses a weir cut across the fabric the width of the channel bottom. This weir is 18 in. high and uses a splash pad downstream of the weir. This splash pad is comprised of a geotextile fabric and riprap. This installation detail is shown in Figure 6.17.



(a) plan view (b) elevation and cross-sectional views

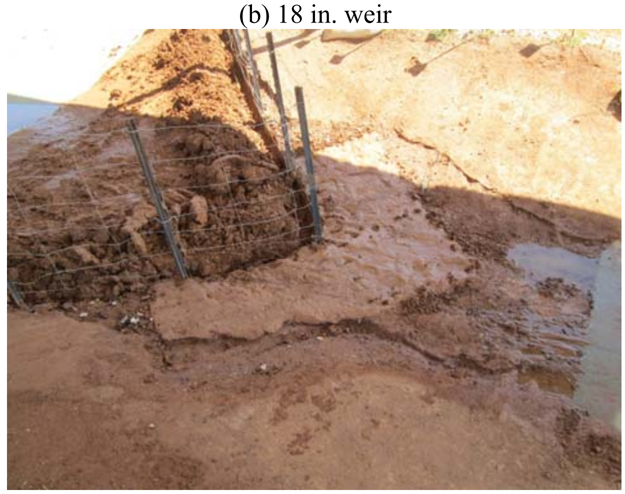
Figure 6.17: TDOT Enhanced Silt Fence Installation (TDOT 2012).

The purpose of this installation is to minimize the amount of impoundment upstream of the silt fence in an attempt to reduce the hydrostatic pressure placed on the device. This method reduces the amount of silt fence that needs to be installed outside of the channel, if the depth of the ditch is less than the height of the fence. This installation is shown in Figure 6.18.

(a) pretest

(b) 18 in. weir

(c) posttest



(d) effect of underlay

Figure 6.18: TnDOT Enhanced Silt Fence Ditch Check Test.

The weir allowed flow to impound the entire length of the channel, creating 30 ft of subcritical flow. This is the optimum condition for earthen channels whereas flow is impounded the maximum impoundment length and the ditch check is not subjected to adverse conditions created by a much larger impoundment. Testing this method showed that a splash pad was an effective means of controlling scour directly downstream of the ditch check. This is evident in Figure 6.18(d).

Integration of a downstream splash pad appeared to be the optimum choice. However, proper implementation of this enhancement was crucial to the success of the practice. Installing the splash pad after the silt fence is installed could create gaps between the splash pad and the fence which could allow water to undercut the splash pad. The ideal scenario is to install the underlay first, then the silt fence on top, however, this creates issues with trenching the fence into the channel. Therefore, a new installation was developed which uses an underlay that extends 2 ft downstream and 1 ft upstream and does not involve trenching the fence into the channel. Instead, the extra fabric is pinned to the channel bottom as shown in Figure 6.19(a).

(a) underlay and pinning of fence

(b) resulting impoundment pool

(c) resulting downstream erosion

(d) demonstration of downstream subcritical pool

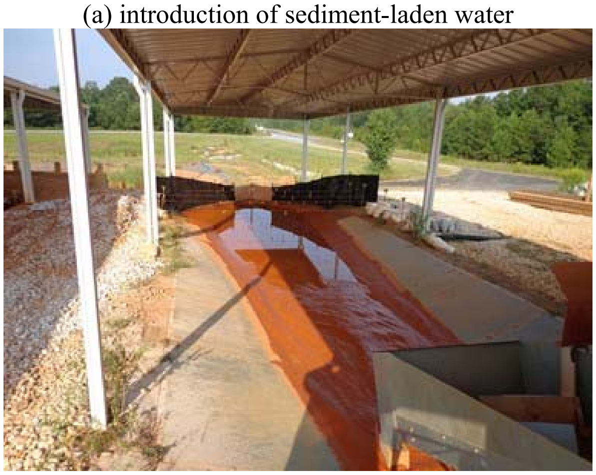
Figure 6.19: Pinned Silt Fence Installation Test.

The downstream splash pad provided protection immediately downstream of the ditch check however, downstream erosion is still a concern for this and all ditch check practices. Figure 6.19(d) demonstrates how properly spacing the ditch check would create a subcritical pool directly downstream of each ditch check which would result in less channel scour from supercritical flow.

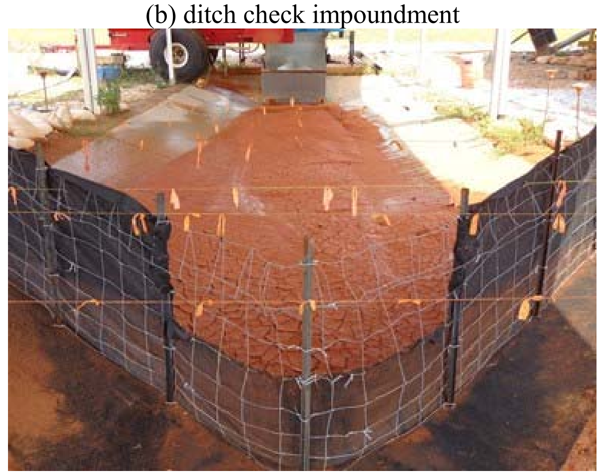
The overall longevity of this practice was a concern. If repeated storm events eventually caused undercutting, the ditch check could be rendered useless over time. Therefore a longevity test was performed to determine the effectiveness of the ditch check over time. Six tests were performed two month period on the same pinned silt fence installation. During the entirety of this longevity test, 41.9 ft^3 of sediment was added to the test flow to further mimic field like conditions. This test is shown in Figure 6.20.

(a) introduction of sediment-laden water

(b) ditch check impoundment



(c) sediment retention after first three tests



(d) sediment retention after one week w/out testing

(e) large particle sediment deposition near channel head

(f) sediment deposition post longevity test

Figure 6.20: Sediment-Laden Silt Fence Longevity Test.

A robotic total station was used to survey the pretest and posttest elevations of the channel to determine sediment retention and downstream scour after the longevity test was completed. The survey concluded that approximately 38.2 ft^3 of sediment was retained upstream of the ditch check which results in 91.2% sediment retention. However, 6.2 ft^3 of sediment loss due to erosion was measured downstream as shown in Figure 6.21.

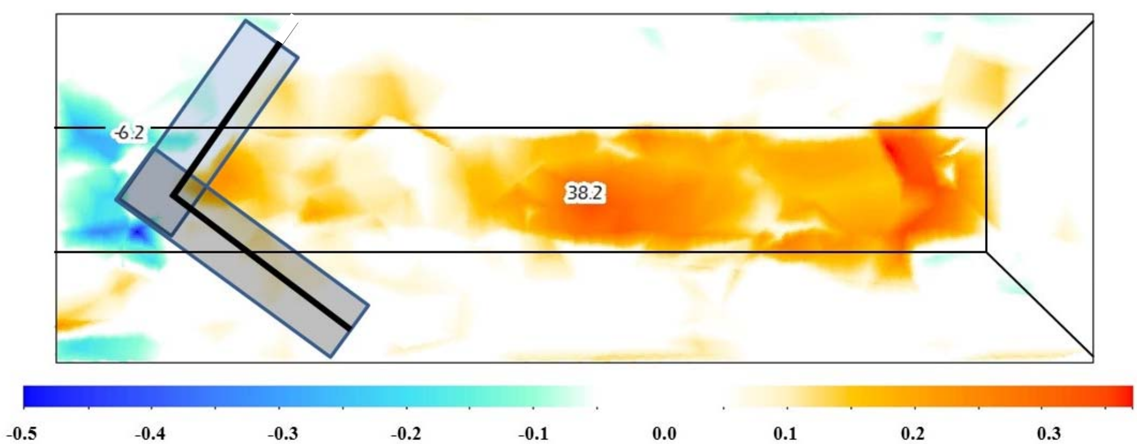


Figure 6.21: Erosion and Deposition Patterns of Silt Fence Longevity Test.

Figure 6.21 shows the deposition pattern upstream of the silt fence installation and erosion patterns caused by the downstream flow. The greatest concentration of sediment deposition occurred just downstream of the sediment and water introduction trough as denoted by the dark orange area. Approximately 0.4 ft thick layer of sediment was deposited. This sediment consisted of the larger sandy particles that fell out quickly due to the low velocity caused by the impoundment. Sediment deposition decreased closer to the ditch check. This is due to the larger sediment depositing upstream and the smaller particles depositing further downstream as more time is required for the smaller particles to settle out. The sediment not retained by the silt fence most likely require longer impoundment times or flocculation to settle out.

6.5.3 Silt Fence Installation Improvements Summary

When silt fence is used to create ditch checks, massive impoundments can be created due to the height of the silt fence when compared to other, shorter ditch check practices. The strain created by the large impoundment accompanied with downstream scour can cause structural failure of the ditch check. If structural failure occurs while at full impoundment, the resulting mass release of impounded water would most likely cause additional failures downstream. Therefore, creating a silt fence ditch check installation that minimizes these conditions are important. The inclusion of a weir creates a more manageable impoundment area for the silt fence installation to endure. Including a splash pad downstream also helps with scour issues which could also cause structural failure. Finally, pinning the silt fence to the channel rather than trenching it allows an underlay to be installed which stretches upstream and downstream of the ditch check, further armoring the ditch check and channel interface. It should be noted that the channel should have a smooth area for the underlay and silt fence to be installed. Rocky and bumpy channels could create issues for the underlay to maintain full ground contact. If full ground contact does not occur, undermining could occur decreasing the ditch checks performance.

6.6 Sand Bag Installation Improvements

Sand bag ditch checks are typically used in hard bottom channels. However, sand bag ditch checks may be used in earthen channels in ditch check applications for higher flow situations where rock may not be desired and wattles would not be adequate.

6.6.1 Sand Bag Installation Improvements Results and Discussion

The following section is a summary of the results and comparisons that were made from the experiments using the tier flow test regime of 0.56 cfs, 1.12 cfs, and 1.68 cfs for ten minutes each for a total test duration of 30 minutes for all large-scale tests performed. Each ditch check installation was required to pass three replicate tests. Table 6.5 shows the comparative results of the various sand bag installations.

Table 6.5: Comparative Results of Each Sand Bag Installation Configuration

Installation Type	Avg. Pool Length (ft)			Avg. % Efficiency(1)
	0.56 cfs	1.12 cfs	1.68 cfs	
Standard ALDOT	29.0	*failed*	*failed*	N/A
Burrito Method	32.7	33.2	33.5	109%
Modified ALDOT	29.0	29.5	30.8	99%

(1) Efficiency compared based upon spacing requirement for a 18 in. tall ditch check in 5% slope

Sand bag ditch checks are comprised of two rows of sand bags, two layers high with a third layer of single row sand bags on top as shown in Figure 5.18 and Figure 6.22. Because the bags and sand restrict flow-through, maximizing the impoundment is not necessarily the main concern for optimizing the installation due to the sandbags ability to block flow and create impoundment. However, structural stability is a concern because the bags are reliant upon gravity and friction to maintain the ditch checks structural integrity. This was evident when testing the Standard ALDOT installation. This installation was able to impound water at the low flow condition of 0.56 cfs, however, once flow increased, the structural integrity was compromised and sand bag dislodgement became a major issue as shown in Figure 6.22(b).

(a) pretest (b) failure at 1.12 cfs

Figure 6.22: Standard ALDOT Sand bag Installation.

ALDOT had used a modified installation to try to inhibit this structural failure by wrapping the sand bags in filter fabric as shown in Figure 6.23. The sandbags were installed in the same manner as the ALDOT standard installation on top of a piece of filter fabric, and the extra fabric was wrapped over the sandbags and tucked under the bags on the downstream side.

(a) downstream view (b) post storm even

Figure 6.23: Sand Bag Burrito Installation on ALDOT Project in Franklin County, AL.

This method garnered some success in the field, so it warranted further investigation to compare to the original installation. The installation in Figure 6.23 was mimicked and tested under the same flow conditions as the standard ALDOT installation. This configuration is shown in Figure 6.24.

(a) standard installation before wrapping (b) installation after wrapping w/ FF

Figure 6.24: Sand Bag Burrito Ditch Check Installation.

This new installation was tested three times using the three tier flow regime. However, during the high flow condition of 1.68 cfs during the third test, the sandbags dislodged at the center, within the filter fabric wrapping causing partial dewatering of the impoundment. These tests are shown in Figure 6.25.

(a) successful test (b) failure at 1.68 cfs

Figure 6.25: Sand Bag Burrito Ditch Check Test.

A fourth replication was tested without failure occurring. This installation proved to increase the overall structural integrity when compared to the standard ALDOT installation, however there was still concern about the stability of the sand bags. Failure of both the standard installation and the burrito installation occurred when the sand bags in the middle layer on the downstream side of the ditch check were finally pushed off the back by the force of the flowing water. The friction holding the sand bags in place was in the direction across the width of the sand bags due to the orientation of the sand bags with regards to the direction of flow.

Therefore, a third installation was developed by changing the orientation of the sand bags. The idea behind changing the orientation was that by orienting the bags to where the length of the bags were parallel to the flow rather than perpendicular, the resulting frictional forces that are resisting dislodgement would be across the length of the sand bag and not the width which is shorter. This reorientation would also keep the bags from rolling off the back of the installation. The first new installation was created by reorienting the bottom and middle layer to be parallel to the direction of flow. This installation held up better than the standard ditch check installation , however in the third flow tier, the middle sand bags were pushed downstream and spread apart, eventually causing structural failure. From this, it was determined that having the bottom layer oriented perpendicular to flow would help deter the bags from being pushed down stream by the flow. Therefore the second orientation iteration used sand bags that were oriented perpendicular to the flow in the bottom layer and parallel to the flow in the second layer. The single row third layer of sand bags was also oriented perpendicular to flow to minimize the number of sandbags required while still increasing the overall height of the ditch check. This installation is shown in Figure 6.26.

(a) bottom sand bag layer perpendicular to flow (b) middle sand bag layer parallel to flow

(c) top sand bag layer perpendicular to flow (d) test during third flow tier of 1.68 cfs

Figure 6.26: Second Iteration of the Modified ALDOT Sand Bag Installation.

The test shown in Figure 6.26 was successful in that the installation was structurally stable throughout the entire tier flow test. However, two subsequent tests were unable to meet this performance stability and structural failure occurred for both tests during the highest flow tier phase. Though this would be an improved installation when compared to the standard ALDOT installation, it did not perform as well as the sand bag burrito installation.

A final installation modification was performed to try to address this structural failure issue. Additional sand bags were placed on the downstream side of the ditch check to help reinforce the sand bags in the middle of the channel. This installation modification is shown in Figure 6.27.

(a) downstream reinforcement sand bags (b) 1.68 cfs flow tier

Figure 6.27: Modified ALDOT Sandbag Installation.

This new modification reinforced the portion of the ditch check most susceptible to failure since the greatest amount of hydrostatic pressure and force due to the flow is located in the middle of the channel. By adding this reinforcement and by reorienting the middle layer of the sand bags, the middle of the ditch check has been stabilized, and the reinforcement bags actually act as a spillway and dissipate energy from the water flowing over the ditch check.

6.6.2 Sand Bag Installation Improvement Conclusion

Sand bag ditch checks rely upon their weight and friction to hold the sand bags in place and resist dislodgment. Because of this, structural integrity was compromised for higher flow rates when all the sand bags were oriented perpendicular to the flow. Wrapping the ditch check in filter fabric provided extra stability, however the issue of sandbags being pushed off the downstream side from the middle layer was still an issue in one of the sand bag burrito installations. Due to this, tests were performed on modified installations that reoriented bags in the middle layer parallel with the flow. This modification improved stability, but the center of the ditch check was unstable in high flow conditions of 1.68 cfs. Finally additional bags were added to the downstream side to reinforce the stability of the middle sand bags.

These installations were all capable of efficiently impounding water, as evident from Table 6.5. However, the sand bag burrito installation did outperform the installations that did not use a filter fabric wrapping. This is most likely due to the filter fabric reducing the flow-through rate of the installation as flow was unable to pass between the sand bags. Therefore, the recommended installation for sand bag ditch checks is to reorient the middle layer to parallel with the flow, wrap the sand bags with filter fabric to increase the stability of the ditch check while also minimizing flow-through, and add additional sand bags to the downstream side of the ditch check to further reinforce the stability of the middle bags.

6.7 Summary

Determining the most effective ditch check installation is difficult because opinions can vary based on manufacturer recommendations or SHA's standard practices. Reevaluating installation procedures in the field can be risky because installation failure often results in increased erosion along with greater sediment transport. Therefore evaluating ditch check installations in a controlled environment helps alleviate risk while providing a more controlled and scientific platform to test various installation configurations.

In order to maximize the installation for wattle ditch checks, seven different installations were evaluated and were: (1) *Downstream Staking*, (2) *Teepee Staking*, (3) *Downstream Staking w/Trenching*, (4) *Teepee Staking w/8 oz. FF and Trenching*, (5) *Downstream Staking w/FF*, (6) *Teepee Staking w/8 oz. FF* and (7) *Teepee Staking w/8 oz. FF + Staples*. Hydraulic evaluation of the test results showed that evaluating performance based solely on EGL slope reduction may lead to improper conclusions, especially if the EGL crosses the hydraulic jump. This is a concern because the standard test method for evaluating ditch checks, ASTM D 7208-06, requires that an EGL be developed from the data with no specific guidance as to interpreting the data. Therefore, it is recommended that the ASTM standard be modified to include more specific guidance for interpreting the EGL data.

A multiple linear regression model was used to evaluate the most significant installation component for increasing subcritical flow length. Five independent variables were identified and compared: (1) trenching, (2) downstream staking, (3) teepee staking, (4) underlay, and (5) stapling. The model showed that the staking pattern did not significantly affect the performance of the wattles. The model did show that trenching, stapling, and underlay did significantly affect performance with trenching being detrimental and stapling and underlay improving. It should also be noted that trenching causes greater erosion downstream and may actually increase the effects of undercutting. Therefore taking the statistical significance into consideration while also looking at the largest increase in subcritical flow length, it is the recommendation of this study that the '*Teepee w/ 8 oz. FF + Staples*' installation be used to install 20 in. diameter wheat straw wattles as ditch checks for maximum stormwater control performance.

Riprap ditch checks are primarily used in high flow situations. Choking is typically recommended so flow-through can be minimized and impoundment length can be maximized. Evaluating the two choking methods recommended by ALDOT required first evaluating the ditch check with no choking mechanism. The two chokers tested were an 8 oz. filter fabric overlay and modified no. 4 stone placed on the upstream face of the ditch check. From this, direct comparison with the installation with no choker was made so that performance increase could be quantified. Based upon this study, it is recommended that a filter fabric choker be used to decrease flow-through rate in order to maximize impoundment capabilities.

Sand bag ditch checks are typically used in hard bottom channels, but can be used in earthen channels if riprap is undesired and wattles are incapable of withstanding the flow.

Because sandbags are not secured in place and rely upon the weight of the sand bags and friction to resist dislodgement, determining the installation with the greatest structural stability was the greatest requirement. Three installations were evaluated for this study, the standard ALDOT installation, a sand bag burrito method that uses filter fabric wrap to encase the sandbags, and an enhanced ALDOT installation that reorients some of the ditch check sand bags while also adding reinforcing bags on the downstream face. This study showed that the standard ALDOT installation was incapable of maintaining the ditch checks structural integrity in flows greater than 0.56 cfs. The sand bag burrito method improved structural stability for all flows, however, one replication failed at high flow due to dislodgement of one of the middle sand bags. Because of this, an new sand bag orientation in which the middle layer of sand bags were reoriented to parallel with the flow in order to decrease dislodgement. This orientation increased stability, however, adding reinforcing bags to the downstream side of the ditch check was required to meet the stability needs for the high flow conditions. Therefore it is the recommendation of this study that when sand bags are used for ditch checks, the middle layer should be reoriented to parallel to the flow direction, the sandbags should be encased in a filter fabric wrap to increase the installations stability and decrease the ditch checks flow-through, and add reinforcing bags on the downstream side, the width of the channel bottom to further increase stability.

Though silt fences are typically reserved for perimeter control for intercepting sheet flow, silt fence is sometimes used as a ditch check device. The height of the ditch check can create a condition where the installation is subjected to very high hydrostatic pressure when compared to shorter ditch check practices. Therefore, an enhanced silt fence installation was developed which mimics the TnDOT installation which uses a weir cut 18 in. in height in the middle of the ditch check to minimize hydrostatic forces on the installation which could lead to increased structural longevity resulting in decreased maintenance requirements. Since trenching the silt fence in a channel can be a laborious task or require operating equipment in an awkward environment, a pinning installation method was developed to minimize the equipment requirements for a silt fence installation. This method showed comparative success to the typical, trenched installation. Therefore it is the recommendation of this study that when a silt fence is used as a ditch check, a weir should be cut in the fence across the width of the channel bottom at a height appropriate for the channel depth. A downstream splash pad should be utilized to minimize downstream scour, and pinning the silt fence should be considered as an alternative to trenching when labor or equipment requirements are a concern

CHAPTER SEVEN

PRODUCT EVALUATION TESTING

7.1 Introduction

Wattles are manufactured products that may be used as ditch checks and come in different dimensions and materials for various applications. These products are tubular, incased in netting (i.e., cotton or polypropylene) and filled with natural or synthetic materials (i.e., wheat straw, excelsior [wood shaving], coir, compost material, recycled carpet fiber, or recycled tires). The advantages of using wattles, over other types of ditch checks (i.e., rock, hay bales, silt fence, etc.) include: (1) overall biodegradability, (2) typically lightweight, (3) easily installed using minimum resources, (4) economical, and (5) are available in various dimensions making them adaptable to site specific constraints. Some limitations of wattle ditch checks include: (1) their elliptic shape may reduce the surface area available for ground contact with the channel bottom resulting in undermining and scour, and (2) the potential for lightweight wattles becoming buoyant under concentrated flow conditions, further reducing adequate ground contact.

Wattles not only vary by materials, but also by physical characteristics (i.e., weight, density, diameter, and length). Wattles are typically classified by diameter and length; however these dimensions are nominal. Materials and dimensions may affect the ability of a wattle to impound runoff and reduce shear stresses on the channel bottom. Understanding how materials and dimensions affect the performance of a wattle will assist practitioners (i.e., designers, inspectors, and contractors) in meeting specific stormwater, erosion, and sediment control needs.

Due to the increasingly stringent regulations being placed upon the C&D industry, an influx of new products geared towards solving erosion and sediment control issues in channelized stormwater runoff are being marketed for use on construction sites. Therefore, an apparent need by the industry to develop a method for evaluating new, manufactured erosion and sediment control products was needed.

Often, product evaluation is performed in the field and is based off of qualitative evaluations and not on quantitative performance. Determining overall effectiveness is inherently difficult when monitoring field installations at construction sites. McLaughlin et al. (2001) states,

> *"Field testing of existing and new sediment and erosion control products or systems has been problematic when conducted on active construction sites. Uncertainty about runoff quantity and quality due to weather patterns and construction activities makes objective, replicated experiments very difficult."*

Therefore, products should be tested using a consistent, reproducible methodology so that direct comparisons can be made between the various products and practices. To accomplish this, the optimum approach is to use large-scale, controlled testing to properly evaluate performance regardless of environmental conditions.

7.2 Current ALDOT Approved Materials

ALDOT maintains an approved material list for erosion and sediment control products called List II-24. This list includes wattles that can be used on ALDOT construction projects as ditch check devices. Figure 7.1 shows this list and the highlighted products were tested by the AU-ESCTF as part of this research. Each ones product specification can be found in Appendix B.

	PEB #	Product Name	Approved Manufacturer	Approval Date
WATTLES	1397	Curlex Sediment Log	American Excelsior - Arlington, TX	05/03/04
	1597	Aspen Excelsior Logs	Western Excelsior - Mancos, CO	12/06/04
	1758	EXCEL Straw Logs	Western Excelsior - Mancos, CO	06/06/06
	1770	Natural Straw Wattle 2010	Winters Excelsior - McWilliams, AL	07/24/06
	1851	ECWattles 100% Agricultural Straw	East Coast Erosion - Bernville, PA	03/05/07
	1866	Wheat Straw Sediment Logs	Erosion Tech - Juliette, GA	06/05/07
	1849	Erosion Eel	Friendly Environment - Shelbyville, TN	08/13/07
	1649	Filtrexx Filter Soxx	Filtrexx International - Grafton, OH	11/05/07
	2114	AEC Premier Straw Wattles	American Excelsior - Arlington, TX	09/14/09
	1905	GeoBale	GeoHay - Spartanburg, SC	11/02/09
	2008	GeoWattle	GeoHay - Spartanburg, SC	11/02/09

(Note: only products highlighted in yellow were tested)

Figure 7.1: ALDOT Materials List II-24.

List II-24 contains 8 different product manufacturers and 11 different products. Seven of the manufactured products are filled with natural material, five are filled with wheat straw, and two are filled with excelsior wood fiber. The GeoHay products are made of recycled carpet fibers, the Erosion Eel is filled with recycled tires, and the Filtrexx Filter Soxx are filled with compost. The Erosion Eel and Filtrexx Filter Soxx were not tested during this study because they are smaller diameter products and currently do not meet the nominal 20 in. diameter wattle ditch check specification. The GeoBale is the same diameter as the GeoWattle, but is a shorter length typically reserved for inlet protection where longer lengths are not required. Table 7.1 lists the manufacturer's specifications for each product tested. Note beside each manufacturer is an abbreviation that will be used herein when referring to the various wattle types. Figure 7.2 to Figure 7.4 is categorized by fill material and shows each evaluated product installed in the channel. Each wattle was tested using the *NCDOT staking pattern w/ FF underlay*. All tests performed used the tier flow testing regime of 0.56 cfs, 1.12 cfs, and 1.68 cfs for a duration of 10 minutes each resulting in a total test duration of 30 minutes.

Table 7.1: Manufacturer Specified Dimensions of Wattles Tested

Wattle	Nominal Dimensions	Length (ft)	Diameter (in.)	Mass (lbs)	Density (lbs/ft^3)	Filler Material
American Excelsior (AE): *Curlex Sediment Log*	20 in. x 10 ft	10.0	16.9	29.1	1.9	excelsior wood fiber
Western Excelsior (West): *Aspen Excelsior Logs*	20 in. x 10 ft	10.6	12.5	29.2	2.6	excelsior wood fiber
Erosion Tech (ET): *Wheat Straw Sediment Logs*	20 in. x 10 ft	10.4	15.9	40.2	2.8	wheat straw
Western Excelsior (West): *EXCEL Straw Wattles*	20 in. x 20 ft	20.0	20.0	100.0	2.3	wheat straw
Winters Excelsior (Wint): *Natural Straw Wattle 2010*	20 in. x 10 ft	10.0	20.0	83.0	3.80	wheat straw
American Excelsior (AE): *AEC Premier Straw Wattles*	20 in. x 10 ft	11.0	16.0	62.7	4.1	wheat straw
East Coast Erosion (EC): *ECWattles 100% Agricultural Straw*	20 in. x 10 ft	10.8	17.7	68.9	3.7	wheat straw
GeoHay (Geo): *GeoWattle*	18 in. x 10 ft	10.0	20.0	29.5	1.4	synthetic fibers

(a) American Excelsior: Curlex Sediment Log (b) Western Excelsior: Aspen Excelsior Logs

Figure 7.2: Excelsior Fiber Wattles Tested.

(a) Erosion Tech: Wheat Straw Sediment Logs

(b) Western Excelsior: EXCEL Straw Wattles

(c) Winters Excelsior: Natural Straw Wattle 2010

(d) American Excelsior: AEC Premier Straw Wattles

(e) East Coast Erosion: EC Wattles 100% Agricultural Straw

Figure 7.3: Wheat Straw Wattles Tested.

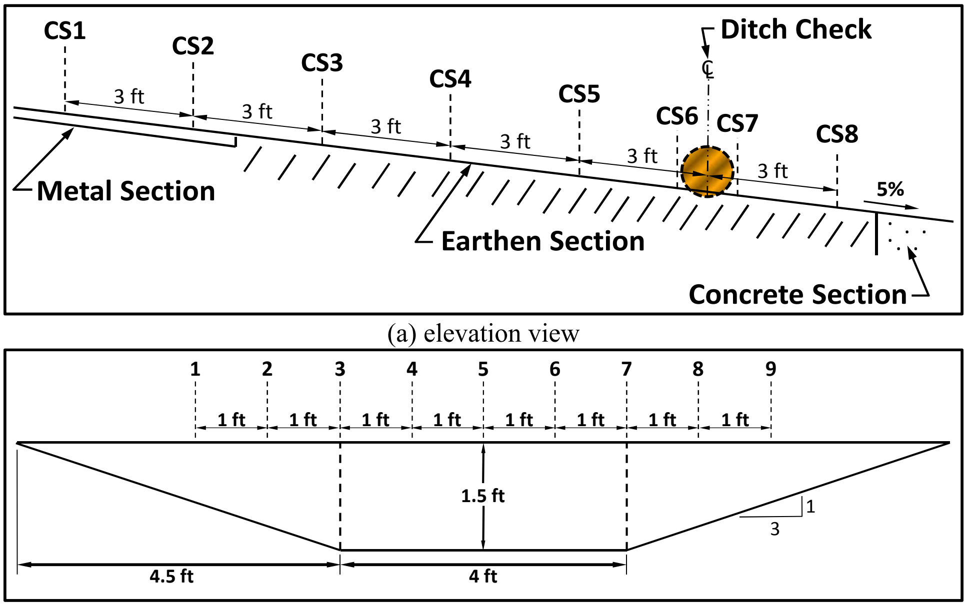
Figure 7.4: GeoHay: GeoWattle.

7.3 Product Approval Results and Discussion

Two excelsior wattle products from two different manufacturers, five wheat straw wattle products from five different manufacturers, and one synthetic wattle product were all tested under the same flow conditions for this research. Three replications using three different wattles from the same manufacturer were tested to represent the overall performance of each product. The data collected for each wattle evaluation test has been averaged to determine overall product performance. This data analysis for each wattle can be found in Appendix C.

7.3.1 Wattle Material Comparisons

Wattles are manufactured products that are comprised of different materials and result in different dimensional properties. Three independent groups used for comparison were separated into fill material groups and were: (1) excelsior wattle (EW), (2) wheat straw wattle (WW), and (3) synthetic wattle (SW). The resulting impoundment depths measured at CS6 from Figure 7.5(a) for each flow tier are shown in Table 7.2. This is the criteria used to help evaluate performance based upon material properties of wattles. Table 7.2 shows resulting average impoundment depths from three tests conducted and actual product densities measured pretest for each wattle.



(b) cross-sectional view

Figure 7.5: Ditch Check Test Channel Dimensions and Configuration.

Table 7.2: Manufactured Wattles with Product Density and Resultant Flow Depths

Wattle Category	**Wattle**	**Density (lbs/ft^3)**	**Low Flow[1] Pool Depth (ft)**	**Medium Flow[2] Pool Depth (ft)**	**High Flow[3] Pool Depth (ft)**
Excelsior (EW)	**American Excelsior (AE):** *Curlex Sediment Log*	1.38	0.34	0.50	0.65
	Western Excelsior (West): *Aspen Excelsior Logs*	2.40	0.42	0.64	0.78
Wheat (WW)	**Erosion Tech (ET):** *Wheat Straw Sediment Logs*	2.75	0.61	0.87	0.93
	Western Excelsior (West): *EXCEL Straw Wattles*	2.40	0.52	0.73	0.83
	Winters Excelsior (Wint): *Natural Straw Wattle 2010*	3.80	0.74	1.02	1.05
	American Excelsior (AE): *AEC Premier Straw Wattles*	2.75	0.78	0.98	1.16
	East Coast Erosion (EC): *ECWattles 100% Agricultural Straw*	4.22	0.84	1.02	1.11
Synthetic (SW)	**GeoHay (Geo)**: *GeoWattle*	1.67	0.86	1.21	1.34

Note: 1. Low Flow = 0.56 cfs, Medium Flow = 1.12 cfs, High Flow = 1.68 cfs

The velocity, WSE profile, and resulting EGL were determined for each flow tier for each test. For open channel flow applications, the water depth profile is equivalent to the HGL. The difference between the WSE and EGL is the velocity head defined by Equation 7.1:

$$h = \frac{v^2}{2g} \qquad \text{(Equation 7.1)}$$

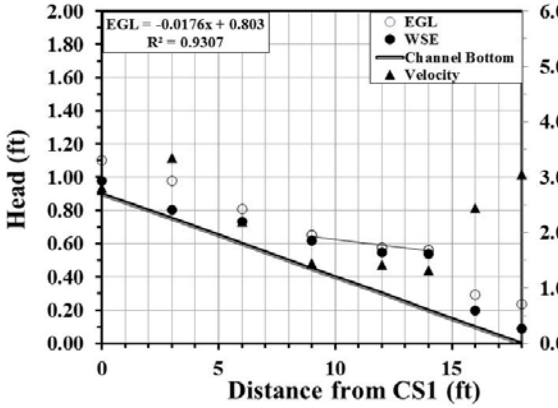
where,

h = velocity head (ft)

v = average water velocity (ft/sec)

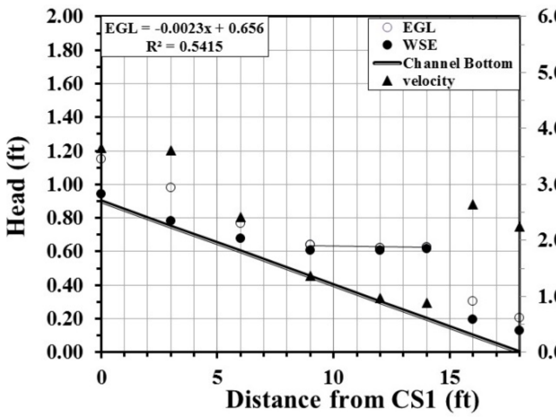
g = acceleration due to gravity (32.2 ft/sec^2)

The reduction of this velocity head and the increase in water depth shows the ability of a ditch check to impound water. It can be seen in Figure 7.6 to Figure 7.8, as water is impounded, the WSE approaches the EGL and therefore the velocity head decreases, thereby minimizing erosive forces and creating favorable conditions for sedimentation. The trend lines shown in Figure 7.6 to Figure 7.8 show the areas of highest WSE to EGL ratio which correspond to the areas with lowest velocity. As the impoundment area increases, a longer portion of the channel is protected from erosion by low velocity, subcritical flows. This is less apparent in the excelsior wattles shown in Figure 7.6 and more apparent in the Figure 7.7(e) to Figure 7.7(i) which are the largest of the wheat straw wattles, and in Figure 7.8 for the GeoHay wattle. A graphical representation for all wattles tested is shown for low flow conditions for all tested products in Figure 7.6 to Figure 7.8 along with corresponding pictures of impoundments produced by the wattles.



(a) HGL and EGL of AE EW

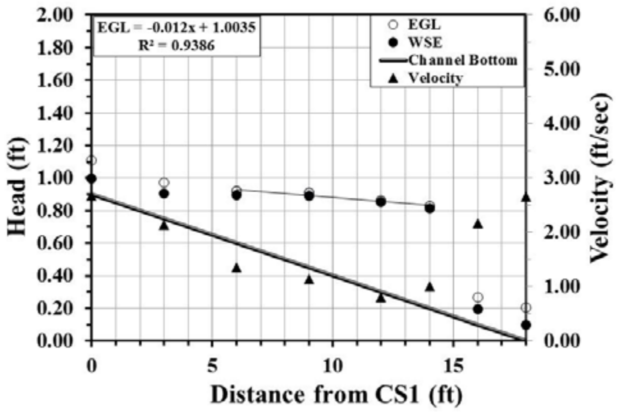
(b) impoundment length for AE EW (1.5 ft)



(c) HGL and EGL of EW-1

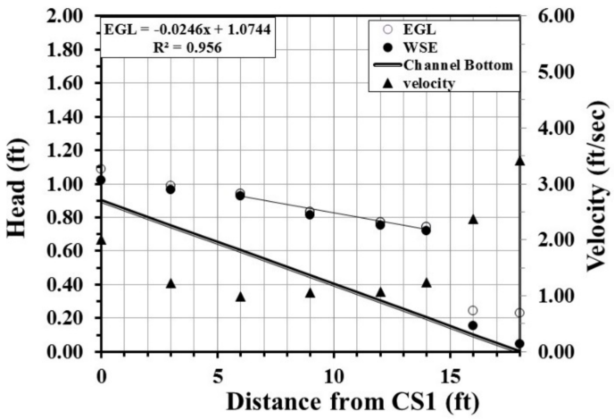
(d) impoundment length for West EW (8 ft)

Figure 7.6: Excelsior Wattles Results for Low Flow Conditions.



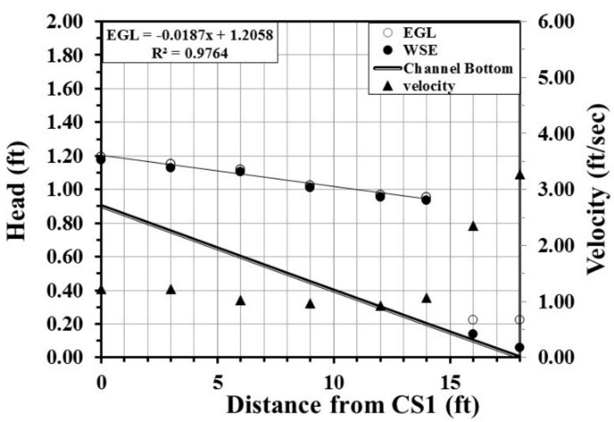
(a) HGL and EGL of ET WW

(b) impoundment length for ET WW (11.8 ft)



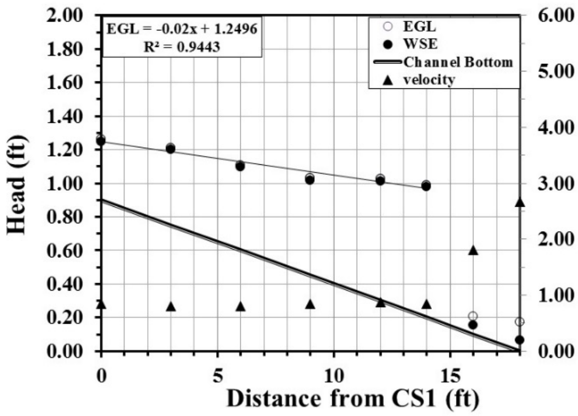
(c) HGL and EGL of West WW

(d) impoundment length for West WW (16.5 ft)



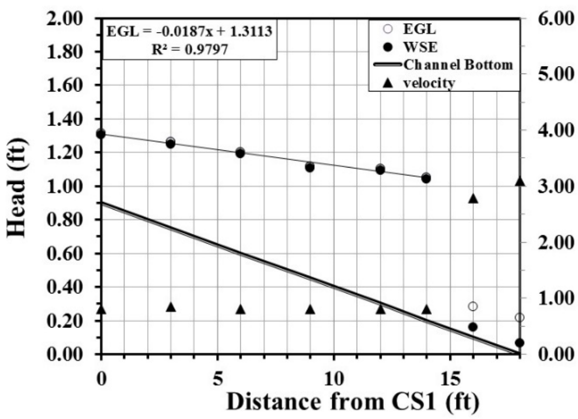
(e) HGL and EGL of Wint WW

(f) impoundment length for Wint WW (20.9 ft)



(g) HGL and EGL of AE WW

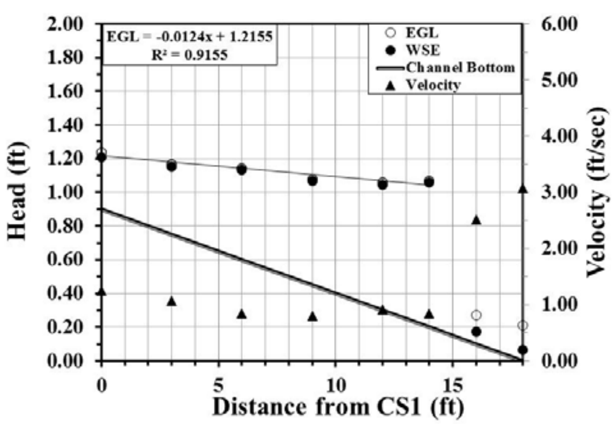
(h) impoundment length for AE WW (24.7 ft)



(i) HGL and EGL of EC WW

(j) impoundment length for EC WW (22.7)

Figure 7.7: Wheat Straw Wattle Results for Low Flow Conditions.



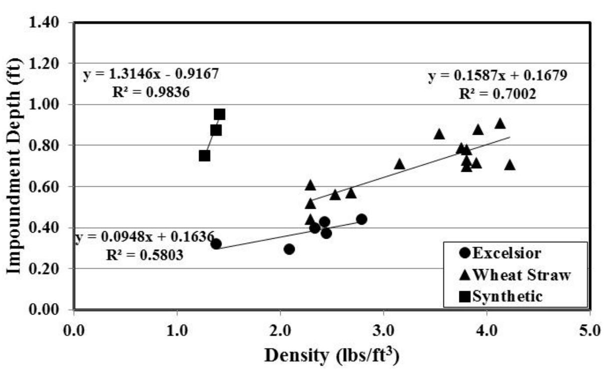
(a) HGL and EGL of Geo SW

(b) impoundment length for Geo SW (21.6 ft)

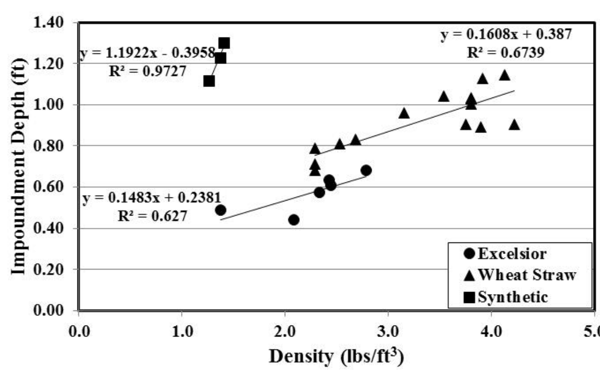
Figure 7.8: GeoHay GeoWattle Results for Low Flow Conditions.

The density of each wattle takes into consideration the cross-sectional area, length, and weight. Though each wattle tested falls into the 18 to 20 in. diameter wattle classification, these diameters are considered nominal and variance in actual diameters is to be expected. Practitioners may be able to determine wattle performance based upon manufacturer specified product density if a correlation can be determined through actual in-place testing of various wattle products.

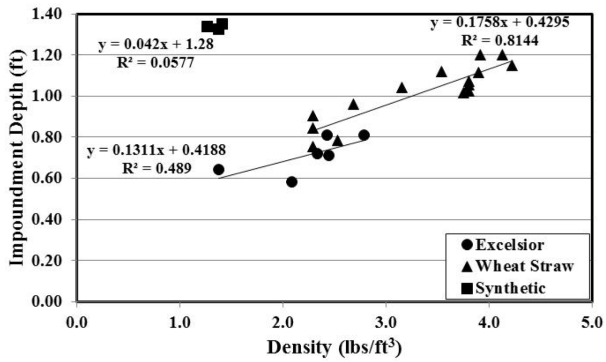
The data collected from this research was categorized into material groups of excelsior wattles (EW), wheat straw wattles (WW) and synthetic wattles (SW). The impoundment depths to density relationships for each wattle were developed and plotted in Figure 7.9. This relationship helps compare the performance of each wattle to determine if performance is affected by material properties. Since some wattles are denser, comparing this relationship of water depth to density illustrates the effect various wattle properties (i.e., weight, diameter, and length) may have on overall performance.



(a) low flow (0.56 cfs)



(b) medium flow (1.12 cfs)



(c) high flow (1.68 cfs)

Figure 7.9: Impoundment Depth vs. Density Relationships.

The data shown in Figure 7.9 shows that there appears to be similar trends within each material group for various flow conditions. It also shows that there may be performance similarities (i.e., depth to density relationship) between the excelsior and wheat straw wattle groups. The synthetic wattle creates a much greater depth, even though it has a lower density in comparison to the other material groups. This greater depth with less density relationship is due to the ability of the recycled carpet fibers to absorb water when exposed to concentrated flow, causing the wattle to become heavier over the course of the test than pretest measurements of density would suggest. A statistical analysis of this data was performed to determine if there is a statistically significant difference between material groups for impoundment depth versus density relationship.

7.3.1.1 Density Relationships Statistical Analysis

A one-way analysis of variance (ANOVA) test was performed on the impoundment depths to density relationships for each set of flow tiers. To perform this evaluation, the depth to density relationship for each wattle was developed into a ratio of depth (y) at cross section 6 divided by the product density. Though this ratio is not dimensionless, it will help normalize the depth versus density relationship and allow comparisons to be made between material groups.

The null hypothesis is that the material does not affect the hydraulic performance of a wattle. A significance level (p-value) of 0.05 was used to determine significance. If analysis results in a p-value of less than 0.05, the null hypothesis is rejected, and the fill material is deemed to significantly affect performance. The three test groups compared were excelsior, wheat straw, and synthetic fiber wattles.

Each flow tier was tested for significance between each group. For all flow tiers, it was determined from the ANOVA test that there were significant differences between the performance of each material group with regard to water impoundment with respect to product density (p-value = 0.00 for all tests). However, because the ANOVA test cannot determine which groups are different, a least significant difference (LSD) test was also performed for each flow tier to determine which groups were statistically significantly different from one another. Table 7.3 shows the significance between groups for each flow tier.

Table 7.3: Determination of Significance between Wattle Material Groups

Wattle Material Group Statistical Comparison	Low Flow (0.56 cfs)		Medium Flow (1.12 cfs)		High Flow (1.68 cfs)	
	Comparisons Ratio Avg.	Level of Significance	Comparisons Ratio Avg.	Level of Significance	Comparisons Ratio Avg.	Level of Significance
EW:WW	0.17:0.21	0.01	0.26:0.28	0.25	0.33:0.31	0.49
EW:SW	0.17:0.64	0.00	0.26:0.90	0.00	0.33:0.99	0.00
WW:SW	0.21:0.64	0.00	0.28:0.90	0.00	0.31:0.99	0.00

For the low flow tier, it was determined that all three groups were significantly different as designated by a level of significance for each comparison being less than 0.05. The excelsior fibers tend to allow flow to pass through the wattle and the lighter weight also allow water to easily pass under the wattle during the low flow tests. Therefore, for low flow conditions, the excelsior wattle material has a higher flow-through rate than wheat and synthetic wattles with the material properties controlling the hydraulic performance more than the density for impounding water. However, as the flow was increased, the excelsior wattles flow-through capacity was exceeded resulting in impoundments comparable to the wheat straw wattles when comparing impoundment depth to density. This is evident by the level of significance being greater than 0.05. Therefore, the depth to density ratios of excelsior and wheat straw wattles were not significantly different for the medium and high flow tiers. For these two flows, the flow through capacity of the excelsior wattle is exceeded by the flow and water must impound higher so new flow passages can become available for the flow to pass through as the wattle attempts to balance flow-through with the flow rate. The resulting impoundment is consistent to the resulting impoundment of the wheat straw wattles when the products density is taken into consideration, and therefore the medium and high flows are more affected by the density of the excelsior wattle than was the case for the low flow condition. The synthetic wattle however was significantly different from the other two groups for all three flow tiers due to its much higher absorption capabilities, resulting in larger impoundments of subcritical flow, thereby protecting the channel from erosion and creating conditions favorable for sediment deposition.

7.3.2 Wattle Material Properties Evaluation Conclusions

The material properties of wattles appear to play a significant role in the products ability to control stormwater runoff by impoundment. For synthetic material wattles, pre-installation density does not appear to play a critical role in the overall performance of the product. This is most likely due to the nonporous, yet light weight recycled carpet fibers that absorb water and increase the weight, therefore impound a greater amount of water than expected. For the natural materials, however, density does seem to play a role because of the loose, fibrous nature of the wheat straw and excelsior material. The wheat straw wattles are denser than the excelsior fiber wattles, and therefore better restrict flow through which results in greater impoundment.

7.4 Wattle Product Performance Criteria

One of the main focuses of this research was to develop a means for evaluating products and practices using a criteria that could affectively compare the performance of ditch checks. Determining an acceptable wattle performance criteria is especially important when determining product approval. This criteria can be used to determine which manufactured products affectively protect the channel from the erosive forces created by channelized flow.

7.4.1 Failure Modes

Throughout the testing process, it was noted that there are two possible conditions for failure: structural or performance. Structural failure typically is a result of improper installation or inability to withstand the hydrostatic pressure created by higher flows. Performance failure was the result of the product not impounding water adequately and therefore leaving the channel exposed to high velocity flow. Most designers rely on a 2-year, 24-hour storm event for many stormwater best management practices to design a SWPPP which usually details the type of ditch checks that are to be installed. This may lead practitioners to think that structural performance failure due to the products inability to withstand high flow rates would be the most crucial mitigating factor for determine product performance criteria. However, it should be noted that the likelihood of a 2-year storm event to occur is 50% during a 1-year period. During that time span, many smaller storm events occur producing lower peak flow rates. Therefore, if the product is unable to impound water during smaller storm events, then the product is not performing properly since a large portion of the channel is left susceptible to erosion. Therefore, the criteria should be developed to determine the performance point at which a product should be considered an inadequate ditch check device and determine the maximum flow rate these products should be implemented for use as ditch checks.

7.4.2 Performance Criteria Data Analysis

Each wattle was tested using the tier flow test regime of 0.56 cfs, 1.12 cfs, and 1.68 cfs for ten minutes each for a total test duration of 30 minutes. It was important to evaluate these products at these different flow rates to determine a level of performance (both structural and performance) for each product at these prescribed rates. Through testing, it was determined that wattle density did have a significant effect on performance for the excelsior and wheat straw wattles. It should also be noted that because of the absorption capabilities of the GeoHay: GeoWattle, it performed in the same manner as the higher density wheat straw wattles, while having a manufactured density less than the excelsior wattles. Testing showed that during the high flow rate test of 1.68 cfs, the GeoHay, Winters Excelsior wheat straw, American Excelsior wheat straw, and the East Coast Erosion wheat straw wattles either partially or completely dislodged the stakes used to secure the wattles in place. The East Coast Erosion and GeoHay wattle completely broke the downstream stakes during some tests and was completely dislodged resulting in total structural failure. This complete wattle failure not occurring in every test is most likely the result of manufacturer irregularities which could produce wattles that are denser than others. Therefore, the recommended peak flow rate for wattles being used as ditch checks for a 4 ft bottom, 3:1 side slope channel at a gradient of 5% is 1.12 cfs. From this, the products will be evaluated based upon the low and medium flow rates of 0.56 and 1.12 cfs respectively.

The average water depth (y) was used to calculate the cross sectional area of flow for each cross section. Calculating the cross section is performed by using Equation 7.2.

$$\mathbf{A} = (\mathbf{B} + \mathbf{sy})\mathbf{y} \qquad \text{(Equation 7.2)}$$

where,

A = cross sectional area (ft^2)

B = channel bottom width (ft)

s = ratio of horizontal to vertical slope of the channel side

y = flow depth for a specific cross section (ft)

The specific energy (E) was calculated using Equation 7.3.

$$\mathbf{E = y + \frac{v^2}{2g}} \qquad \text{(Equation 7.3)}$$

where,

E = specific energy (ft)

y = flow depth for a specific cross section (ft)

v = average velocity measured for each cross section (ft/sec)

g = acceleration due to gravity (32.2 ft/sec^2)

The Froude number is used in open channel flow to express the ratio of inertial forces (kinetic energy) to gravity forces (Crowe et al., 2001). For open channel flow, the Froude number is defined in Equation 7.4.

$$\mathbf{Fr = \frac{v}{\sqrt{gD}}} \qquad \text{(Equation 7.4)}$$

where,

Fr = Froude number

v = average velocity measured for each cross section (ft/sec)

g = acceleration due to gravity (ft/sec^2)

D = hydraulic depth (ft)

The hydraulic depth (D) is the cross sectional area (A) divided by the top width (b) of the flow at each cross section. The top width of flow for a trapezoidal channel is defined in Equation 7.5.

$$\mathbf{b = B + 2sy} \qquad \text{(Equation 7.5)}$$

where,

b = top width of flow (ft)

B = bottom width of channel (ft)

s = ratio of horizontal to vertical slope of the channel side

y = average flow depth for the cross section (ft)

The EGL is defined by Equation 7.6 (ASTM 2006).

$$EGL = HGL + \frac{v^2}{2g} \qquad \text{(Equation 7.6)}$$

where,

EGL = energy grade line (ft)

HGL = hydraulic grade line which equals the water surface elevation (ft)

v = average water velocity (ft/sec)

g = gravitational constant (32.2 ft/sec2)

When the HGL:EGL ratio is calculated, a relationship which shows how flow depth and flow velocity interacts is created. As the HGL:EGL relationship approaches one, velocity is approaching zero and the ratio is reliant upon the water depth, and as the ratio decreases, the velocity becomes more dominant. Table 7.4 shows the average test results for the three replicates tested for the bare soil control and each product for the low flow rate. Table 7.5 shows this same data for the medium flow rate tests. During testing, water depth and velocity measurements were taken at each cross section once steady state flow occurred. These tables only show the upstream cross sections (CS1 to CS6) where CS1 is the cross section furthest away from wattle and CS6 is directly upstream of the wattle.

Table 7.4: Low Flow Rate Wattle Ditch Check Results

Low Flow (0.56 cfs)												
Wattle	**CS**	**Depth, ft**	**v, ft/sec**	**E, ft**	**A, ft^2**	**B, ft**	**Fr**	**y/E**	**HGL:EGL**	**Avg Fr**	**Avg y/E**	**Avg HGL:EGL**
Bare Soil Control	1	0.06	3.38	0.24	0.26	4.37	2.46	0.26	0.84	2.06	0.34	0.80
	2	0.05	3.52	0.25	0.23	4.33	2.71	0.22	0.81			
	3	0.07	2.86	0.20	0.31	4.45	1.89	0.37	0.84			
	4	0.07	2.63	0.18	0.28	4.41	1.82	0.39	0.83			
	5	0.08	2.48	0.17	0.33	4.47	1.61	0.45	0.80			
	6	0.08	2.91	0.21	0.34	4.48	1.86	0.38	0.68			
AE EW	1	0.08	2.82	0.20	0.32	4.46	1.84	0.38	0.89	1.21	0.65	0.91
	2	0.05	3.37	0.23	0.21	4.31	2.67	0.22	0.82			
	3	0.13	2.22	0.21	0.59	4.80	1.12	0.64	0.91			
	4	0.17	1.46	0.20	0.75	5.00	0.66	0.83	0.95			
	5	0.25	1.43	0.28	1.16	5.47	0.55	0.89	0.94			
	6	0.34	1.34	0.36	1.69	6.02	0.44	0.92	0.95			
West Ew	1	0.04	3.66	0.25	0.18	4.27	3.11	0.18	0.82	1.60	0.59	0.90
	2	0.03	3.61	0.23	0.12	4.18	3.71	0.13	0.79			
	3	0.08	2.42	0.17	0.33	4.47	1.57	0.46	0.88			
	4	0.16	1.36	0.19	0.71	4.95	0.63	0.85	0.95			
	5	0.31	0.97	0.32	1.52	5.85	0.34	0.95	0.98			
	6	0.42	0.88	0.43	2.18	6.49	0.27	0.97	0.98			
ET WW	1	0.10	2.67	0.21	0.41	4.58	1.56	0.47	0.90	0.64	0.83	0.96
	2	0.15	2.13	0.22	0.68	4.92	1.01	0.68	0.93			
	3	0.29	1.35	0.32	1.43	5.76	0.48	0.91	0.97			
	4	0.44	1.13	0.46	2.34	6.64	0.34	0.96	0.98			
	5	0.55	0.80	0.56	3.11	7.30	0.22	0.98	0.99			
	6	0.61	1.00	0.63	3.58	7.68	0.26	0.98	0.98			
West WW	1	0.12	2.00	0.19	0.54	4.74	1.05	0.67	0.94	0.48	0.90	0.97
	2	0.22	1.22	0.24	1.00	5.29	0.50	0.90	0.98			
	3	0.33	0.99	0.34	1.63	5.97	0.33	0.96	0.98			
	4	0.36	1.06	0.38	1.85	6.18	0.34	0.95	0.98			
	5	0.45	1.07	0.47	2.44	6.73	0.31	0.96	0.98			
	6	0.52	1.23	0.54	2.90	7.13	0.34	0.96	0.97			
Wint WW	1	0.27	1.22	0.30	1.32	5.64	0.45	0.92	0.98	0.31	0.96	0.98
	2	0.38	1.22	0.40	1.94	6.27	0.39	0.94	0.98			
	3	0.50	1.02	0.52	2.77	7.02	0.29	0.97	0.99			
	4	0.56	0.96	0.58	3.19	7.37	0.26	0.97	0.99			
	5	0.65	0.92	0.67	3.89	7.92	0.23	0.98	0.99			
	6	0.74	1.07	0.75	4.57	8.41	0.26	0.98	0.98			
AE WW	1	0.35	0.84	0.36	1.76	6.09	0.28	0.97	0.99	0.23	0.98	0.99
	2	0.45	0.80	0.46	2.41	6.70	0.24	0.98	0.99			
	3	0.50	0.80	0.51	2.72	6.97	0.23	0.98	0.99			
	4	0.57	0.84	0.58	3.24	7.41	0.22	0.98	0.99			
	5	0.71	0.88	0.73	4.38	8.28	0.21	0.98	0.99			
	6	0.78	0.84	0.79	4.93	8.67	0.20	0.99	0.99			
EC WW	1	0.40	0.80	0.41	2.10	6.42	0.25	0.98	0.99	0.21	0.98	0.99
	2	0.50	0.84	0.51	2.75	7.00	0.24	0.98	0.99			
	3	0.59	0.80	0.60	3.41	7.55	0.21	0.98	0.99			
	4	0.66	0.80	0.67	3.91	7.93	0.20	0.98	0.99			
	5	0.79	0.80	0.80	5.05	8.75	0.19	0.99	0.99			
	6	0.84	0.80	0.85	5.50	9.05	0.18	0.99	0.99			
Geo	1	0.31	1.25	0.33	1.53	5.86	0.43	0.93	0.98	0.27	0.97	0.99
	2	0.40	1.07	0.42	2.08	6.40	0.33	0.96	0.98			
	3	0.53	0.84	0.54	2.97	7.19	0.23	0.98	0.99			
	4	0.62	0.80	0.63	3.62	7.71	0.21	0.98	0.99			
	5	0.74	0.91	0.76	4.64	8.47	0.22	0.98	0.99			
	6	0.86	0.84	0.87	5.65	9.15	0.19	0.99	0.99			

Table 7.5: Medium Flow Rate Wattle Ditch Check Results

Medium Flow (1.12 cfs)												
Wattle	CS	Depth, ft	v, ft/sec	E, ft	A, ft^2	b, ft	Fr	y/E	HGL:EGL	Avg Fr	Avg y/E	Avg HGL:EGL
AE EW	1	0.09	4.06	0.35	0.39	4.55	2.43	0.26	0.79	1.32	0.65	0.88
	2	0.06	4.35	0.36	0.27	4.39	3.09	0.18	0.73			
	3	0.29	2.74	0.41	1.43	5.75	0.97	0.72	0.88			
	4	0.33	1.62	0.37	1.63	5.97	0.55	0.89	0.95			
	5	0.40	1.41	0.44	2.11	6.43	0.43	0.93	0.96			
	6	0.50	1.59	0.54	2.76	7.01	0.45	0.93	0.95			
West Ew	1	0.10	3.94	0.34	0.43	4.60	2.27	0.29	0.81	0.84	0.77	0.93
	2	0.17	2.94	0.31	0.78	5.03	1.32	0.56	0.87			
	3	0.27	1.76	0.32	1.32	5.65	0.64	0.85	0.95			
	4	0.37	1.00	0.39	1.89	6.22	0.32	0.96	0.98			
	5	0.53	0.91	0.54	2.95	7.17	0.25	0.98	0.98			
	6	0.64	0.97	0.65	3.79	7.84	0.25	0.98	0.98			
ET WW	1	0.29	2.32	0.37	1.41	5.74	0.82	0.78	0.93	0.38	0.94	0.97
	2	0.40	1.39	0.43	2.08	6.40	0.43	0.93	0.97			
	3	0.55	1.27	0.58	3.13	7.32	0.34	0.96	0.98			
	4	0.68	0.91	0.69	4.11	8.08	0.23	0.98	0.99			
	5	0.80	0.80	0.81	5.09	8.78	0.19	0.99	0.99			
	6	0.87	1.11	0.89	5.72	9.20	0.25	0.98	0.98			
West WW	1	0.31	1.78	0.36	1.54	5.87	0.61	0.86	0.96	0.36	0.94	0.98
	2	0.40	1.51	0.43	2.05	6.37	0.47	0.92	0.97			
	3	0.51	1.15	0.53	2.81	7.05	0.32	0.96	0.98			
	4	0.54	1.15	0.56	3.06	7.26	0.31	0.96	0.98			
	5	0.65	0.96	0.66	3.87	7.90	0.24	0.98	0.99			
	6	0.73	0.88	0.74	4.49	8.36	0.21	0.98	0.99			
Wint WW	1	0.55	1.53	0.59	3.11	7.30	0.41	0.94	0.98	0.28	0.97	0.98
	2	0.67	1.31	0.69	3.99	7.99	0.33	0.96	0.98			
	3	0.80	1.22	0.82	5.10	8.79	0.28	0.97	0.98			
	4	0.85	1.10	0.86	5.53	9.07	0.25	0.98	0.99			
	5	0.95	1.00	0.96	6.49	9.69	0.21	0.98	0.99			
	6	1.02	0.98	1.04	7.22	10.13	0.21	0.99	0.99			
AE WW	1	0.58	1.21	0.61	3.35	7.50	0.32	0.96	0.98	0.21	0.98	0.99
	2	0.67	0.84	0.68	4.03	8.02	0.21	0.98	0.99			
	3	0.73	0.84	0.74	4.48	8.35	0.20	0.98	0.99			
	4	0.80	0.80	0.81	5.12	8.80	0.19	0.99	0.99			
	5	0.91	0.80	0.92	6.10	9.45	0.18	0.99	0.99			
	6	0.98	0.88	0.99	6.81	9.89	0.19	0.99	0.99			
EC WW	1	0.59	0.80	0.60	3.41	7.55	0.21	0.98	0.99	0.19	0.99	0.99
	2	0.69	0.80	0.70	4.16	8.12	0.20	0.99	0.99			
	3	0.78	0.80	0.79	4.94	8.67	0.19	0.99	0.99			
	4	0.86	0.80	0.87	5.63	9.14	0.18	0.99	0.99			
	5	0.96	0.91	0.97	6.59	9.75	0.20	0.99	0.99			
	6	1.02	0.91	1.04	7.25	10.15	0.19	0.99	0.99			
Geo	1	0.64	1.19	0.66	3.81	7.85	0.30	0.97	0.99	0.23	0.98	0.99
	2	0.74	1.16	0.76	4.57	8.41	0.28	0.97	0.99			
	3	0.88	1.03	0.90	5.87	9.30	0.23	0.98	0.99			
	4	0.99	0.89	1.00	6.87	9.92	0.19	0.99	0.99			
	5	1.10	0.91	1.11	7.98	10.57	0.19	0.99	0.99			
	6	1.21	0.91	1.23	9.28	11.29	0.18	0.99	0.99			

This data was used to evaluate the performance of a ditch check. Initially this was performed by comparing the HGL:EGL ratio to the water depth (y) created at each cross section. This would seem to be a reasonable comparison whereas the HGL:EGL ratio would represent how well the flow is impounded and the kinetic energy of the water velocity is reduced and transformed into potential energy by increasing the water depth. This relationship for the low flow condition is shown in Figure 7.10.

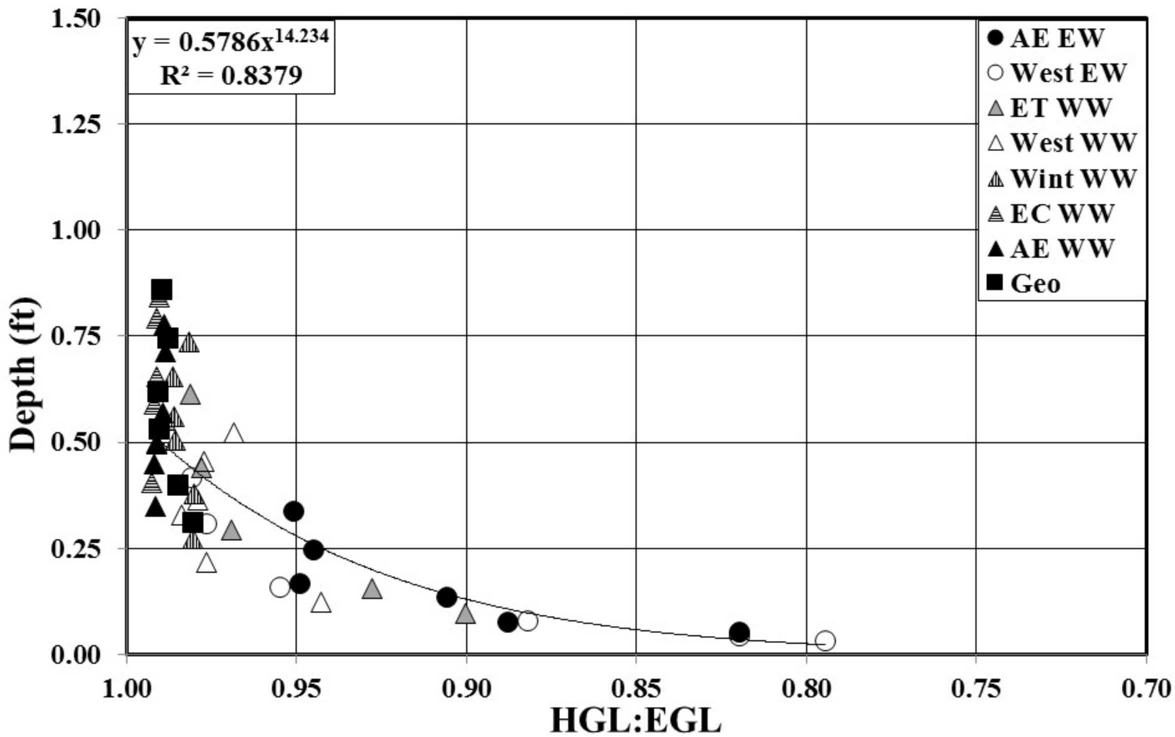


Figure 7.10: Low Flow Relationship of HGL:EGL vs. Depth.

Based upon the material performance of the wattles as ditch checks, it is apparent that the two excelsior wattles, which impounded the least amount of water have the lowest water depths as well as HGL:EGL ratios. These are represented by the circular markers shown in Figure 7.10. The triangular markers shown in Figure 7.10 represent the wheat straw wattles. The wheat straw wattles have higher HGL:EGL ratios as well as greater depths for the same low flow condition. Also notice that the GeoHay wattle, represented by the square markers, performed similarly to the wheat straw wattles despite the low density of that product.

Figure 7.10 shows an apparent correlation between depth and HGL:EGL. Fitting an exponential regression line yields an R^2 value of 0.84. This relationship, however, makes defining a specific pass | fall criteria from this data difficult. Therefore, further data analyses was required. Since water depth and velocity was measured for each cross-section, and the channel geometry is known, the Froude number (Fr) for each cross-section could be calculated. The Froude number determines proportions of inertial forces to gravitational forces of flow. The Froude number basically describes which force is stronger, the velocity force which is in the direction of flow, or the gravitational force which is acting against the velocity force for all flow directions except for the case where water would be falling vertically downward. For Froude numbers greater than one, the inertial force (v) is greater than the gravitational forces and therefore flow is controlled by velocity. However, for Froude numbers less than one, the inertial force is less than the gravitational forces and therefore the flow is not controlled by velocity. This relationship is similar to the HGL:EGL ratio, which describes which flow parameter is stronger, the velocity head or flow depth. As flow depths increase and velocity head decreases, kinetic energy is transformed into potential energy, and the ratio of HGL:EGL moves closer to 1. As this ratio decreases, flow energy is more controlled by the velocity head and depths decrease. From this understanding, the relationship of the Froude number and the HGL:EGL ratio was plotted again for each cross-section and is shown in Figure 7.11.

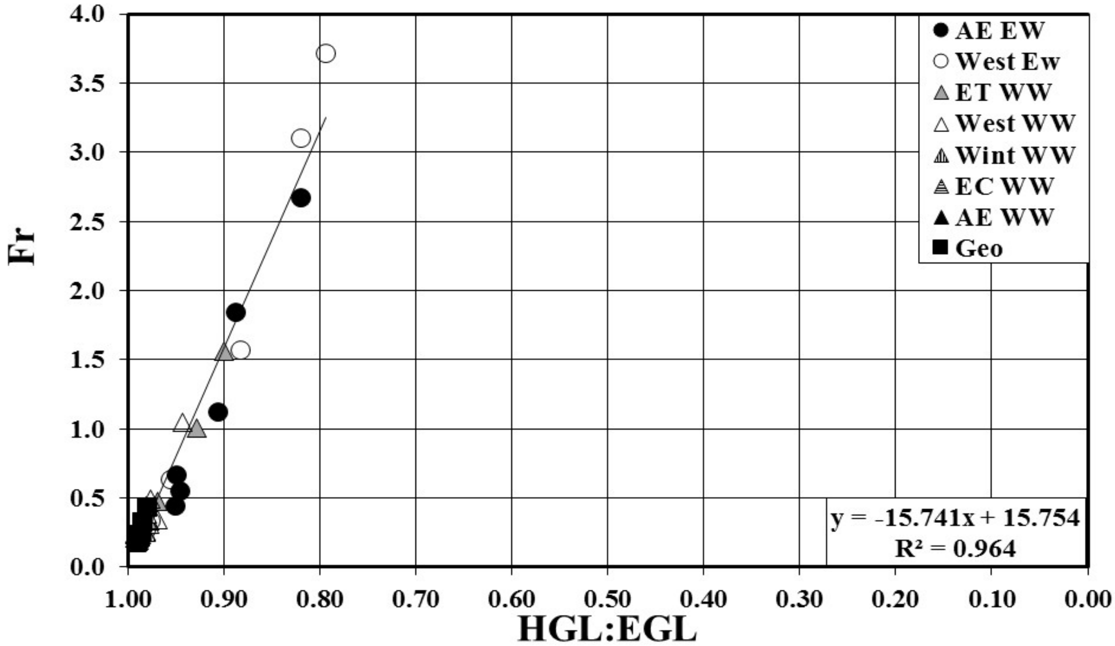


Figure 7.11: Low Flow Relationship of HGL:EGL vs. Froude Number.

Comparing Figure 7.10 and Figure 7.11, Figure 7.11 appears to develop a stronger correlation. A linear regression line was applied to this data resulting in an R^2 value of 0.96. Data points that are shallow with high velocity tend to have a Froude number greater than 1 and an HGL:EGL ratio of less than 0.95. However, upon evaluating the equations for the Froude number and the HGL:EGL ratio, it was realized that the HGL:EGL ratio takes into consideration the channel slope by including the measurement of z for each cross-section at a horizontal datum which is located at an elevation below the channel bottom. This is not a consideration of the Froude number and is not necessary since head loss between cross sections is not a concern of this research objective. Therefore, if z is removed from HGL, water depth (y) is remaining, and if z is removed from EGL, then the specific energy (E) which is the result of depth plus the velocity head for each cross-section is remaining. Therefore, when the HGL:EGL relationship is substituted for y/E, a new correlation is developed and is shown in Figure 7.12.

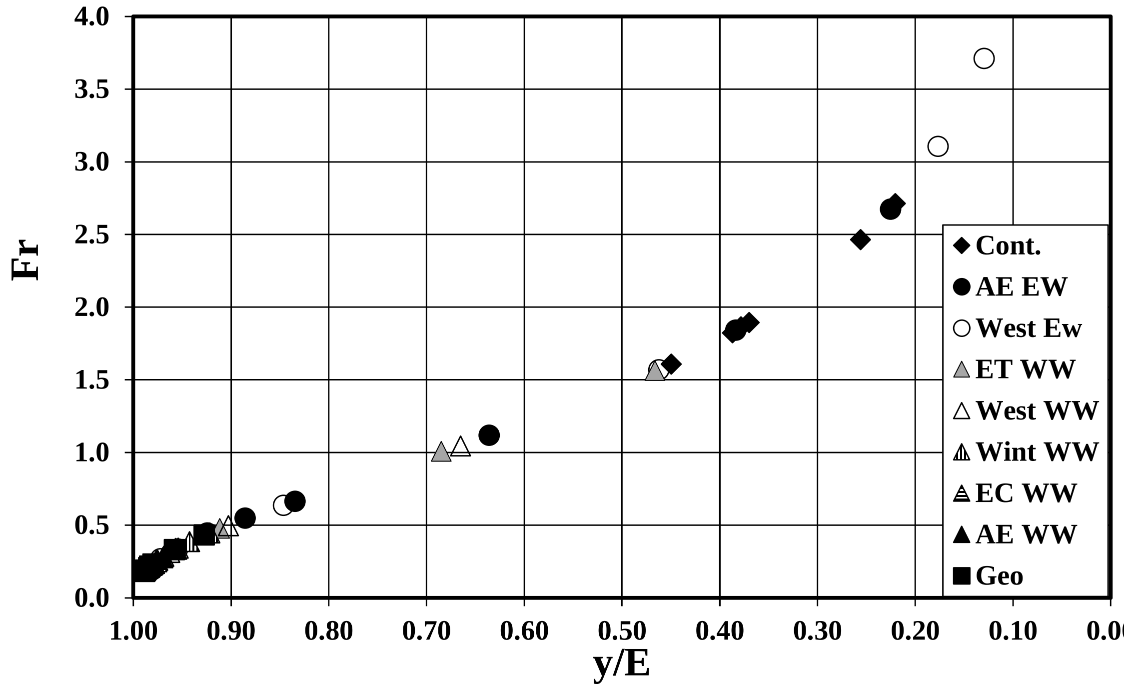


Figure 7.12: Low Flow Relationship of y/E and Froude Number.

Figure 7.12 shows a much stronger correlation between the Froude number and y/E when compared to the other relationships in shown in Figure 7.10 and Figure 7.11. Note, Figure 7.12 also includes the control test (Cont.) for reference of this relationship with no ditch check installed. Once this relationship was discovered, the x-axis was reformatted to a (0,0) x-y intercept axis. This is shown in Figure 7.13. A best fit, 3rd order polynomial trendline was added to the data set. From this, two distinct patterns developed. The trendline increases in an exponential manner as the Froude number increased. However, an inflection point appears to occur below Froude number equal to 1. This inflection point represents the point of the function at which the flow becomes depth dominate and the influence of kinetic energy is minimized.

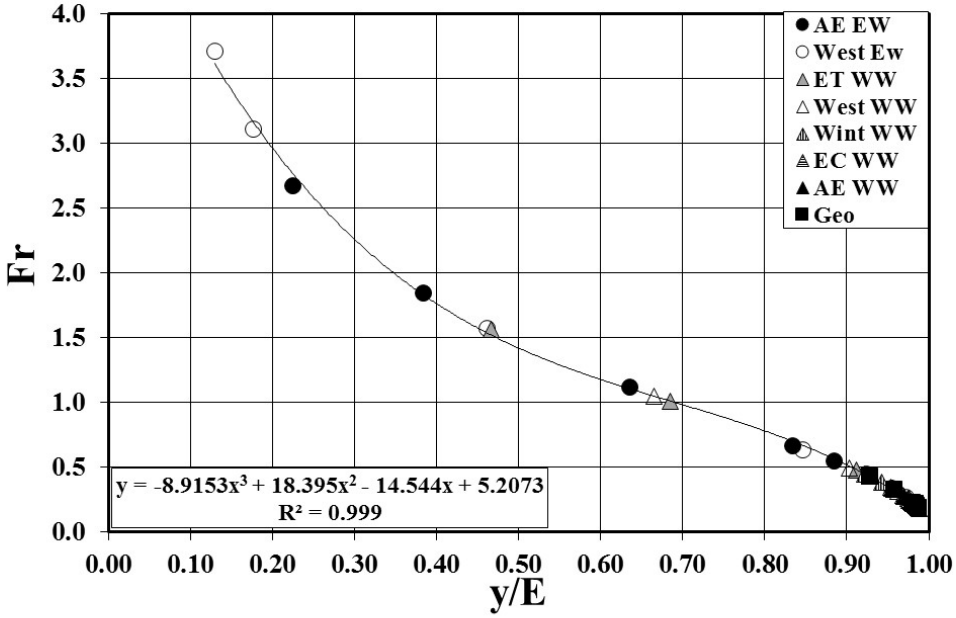


Figure 7.13: Reformatted Low Flow Relationship of y/E and Froude Number.

The medium flow data was also plotted using this relationship and is displayed in Figure 7.14. The lack of data points which creates the trendline makes determining the exact location of the inflection point difficult for this data set function. This is due to the gap in the data that is evident in both Figure 7.13 and Figure 7.14.

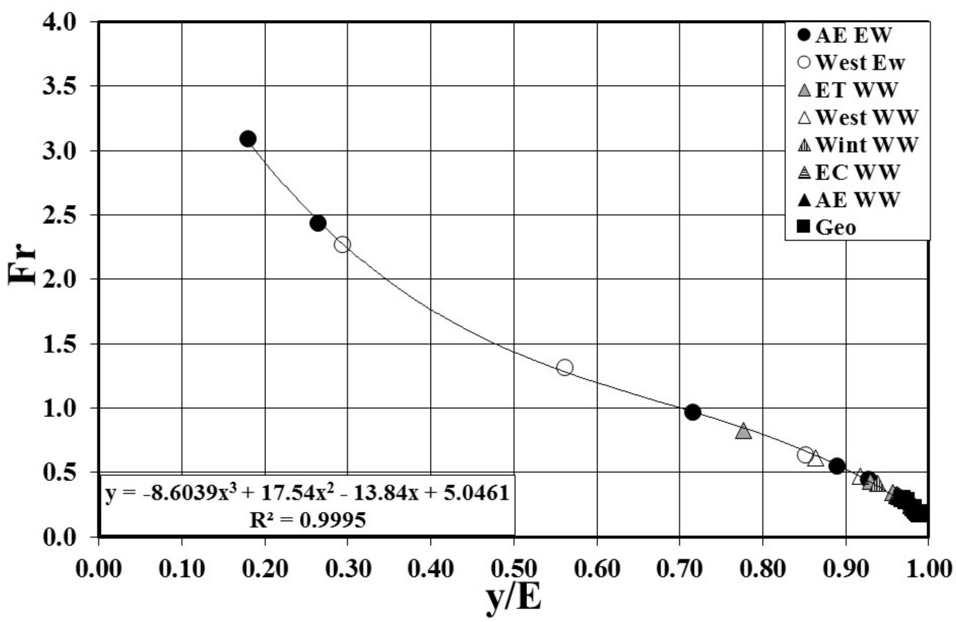


Figure 7.14: Medium Flow Fr vs. y/E Relationship.

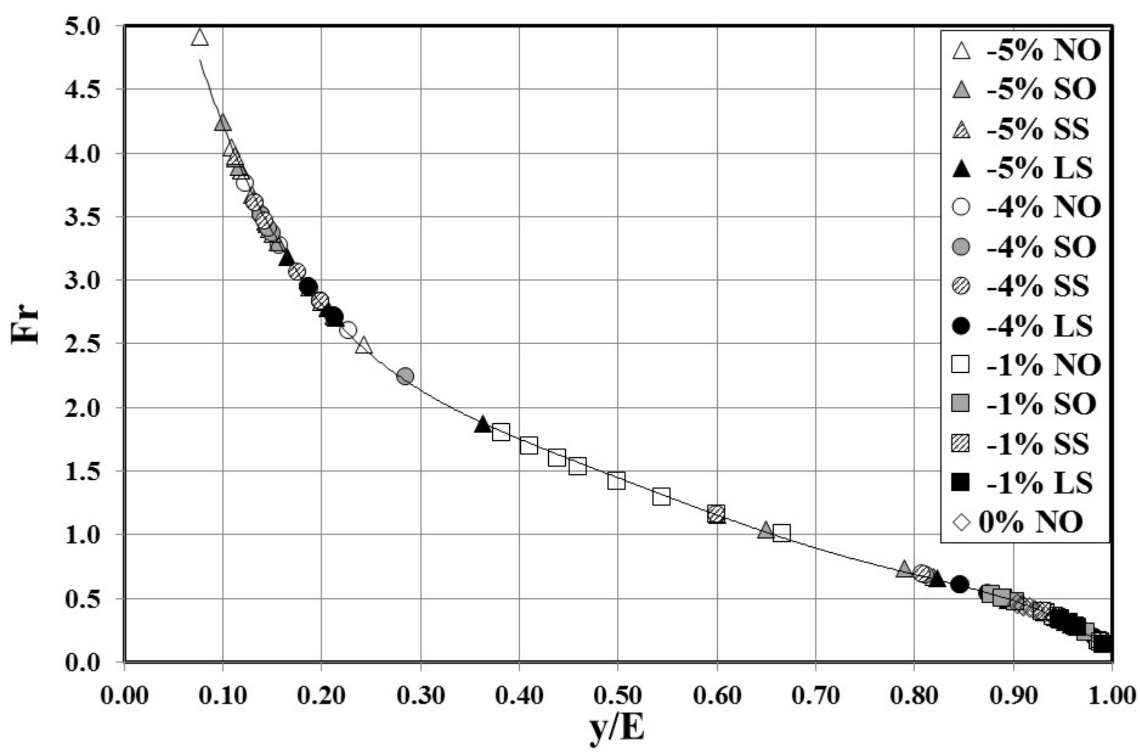
Therefore, further investigation was required to determine if this data creates a consistent line function with an inflection occurring at the same location. This is important since the y/E function does not take into consideration channel geometry. A consistent inflection point would allow data to be compared between tests performed on channels of different dimensions using different flow rates. Further investigation was therefore required to determine if this relationship could be unique to specific channel dimensions or is inherit across different geometries and flows. Similar testing was performed using a small scale 15 ft long rectangular channel with a 1 ft wide bottom. This channel was capable of tilting to different longitudinal channel slopes. Four different slope conditions were tested: 5, 4, 1, and 0% longitudinal slope. For the 5, 4, and 1 % slope, four different scenarios were tested: (1) no obstruction (NO), (2) screen placed in flow path to obstruct flow (SO), (3) a 2⁵⁄₈ by 4¼ in. small sponge was placed in front of the screen to further constrict flow (SS), and (4) a 3¾ by 7 in. large sponge was placed in front of the screen to even further constrict flow (LS). For the 0% slope condition, only the flow with no obstruction was tested since the flow was already in the subcritical flow state. Velocity and water depth measurements were taken at 9 different measurement points spaced 1 ft apart with the first measurement occurring directly upstream of the screen. Table 7.6 shows the average Froude number and average y/E for each flow scenario for a flow rate of approximately 0.2 cfs.

Table 7.6: Results of Small Scale Channel Test of Froude Number vs. y/E for 0.2 cfs

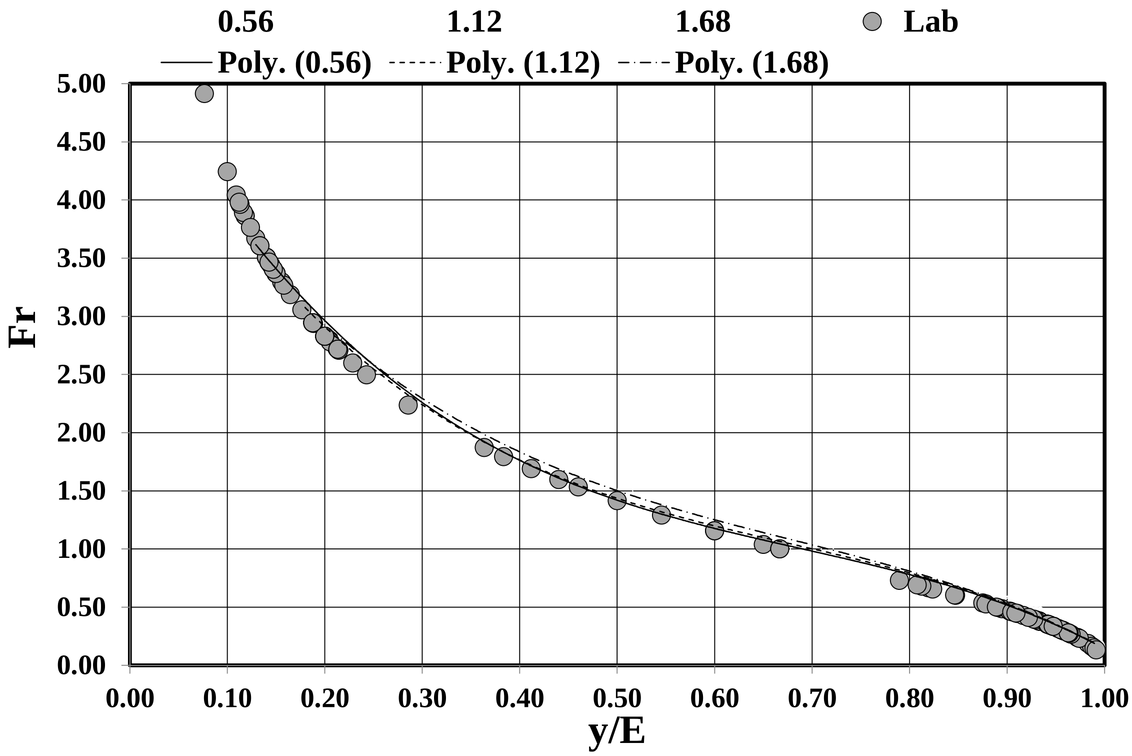
	Average Froude Number				Average y/E			
Slope	**NO**	**SO**	**SS**	**LS**	**NO**	**SO**	**SS**	**LS**
5%	3.51	2.92	2.46	1.75	0.15	0.27	0.37	0.52
4%	3.22	2.46	2.05	1.17	0.17	0.34	0.45	0.69
1%	1.39	0.39	0.33	0.27	0.52	0.93	0.95	0.96
0%	0.4	N/A	N/A	N/A	0.92	N/A	N/a	N/A

Two distinct patterns are evident from this data. As flow is constricted further by adding less permeable obstructions, the Froude number decreases and the y/E average increases.

Similarly, as the channel slope decreases, the Froude number for the same obstructions also decrease and the y/E increases. Also notice that for the channel conditions when the average flow is subcritical across all the cross sections which occurred for the SO, SS, and LS conditions at 1%, the Froude number and the y/E are very close to those produced by the 0% slope flow condition. As the obstructions increased further, Fr moved closer to zero and y/E moved closer to 1 than the actual 0% slope. This is due to the obstruction allowing the velocity to decrease closer to zero whereas even at full subcritical flow, without the obstruction, velocity does not decrease as quickly when subjected only to the frictional forces caused by the smooth channel bottom. All the tested scenarios are shown in Figure 7.15(a). A similar function was created by this data using a different flow rate and channel geometry. A similar gap in the data set was also observed which occurs near Froude number of 0.75 up to 2. Figure 7.15 (b) shows the lab data against the ditch check test channel data and the slight variation in trend lines.



(a) all lab scenarios



(b) lab versus ditch check channel

Figure 7.15: All Lab Scenarios Plotted Froude Number vs. y/E.

Theoretical data was generated to determine if this relationship could be further understood. A theoretical curve was developed from incremental depth of 0.0001 using a specified flow rate and calculating the cross sectional area and resulting velocity using Equation 7.7.

$$\mathbf{Q = VA} \qquad \text{(Equation 7.7)}$$

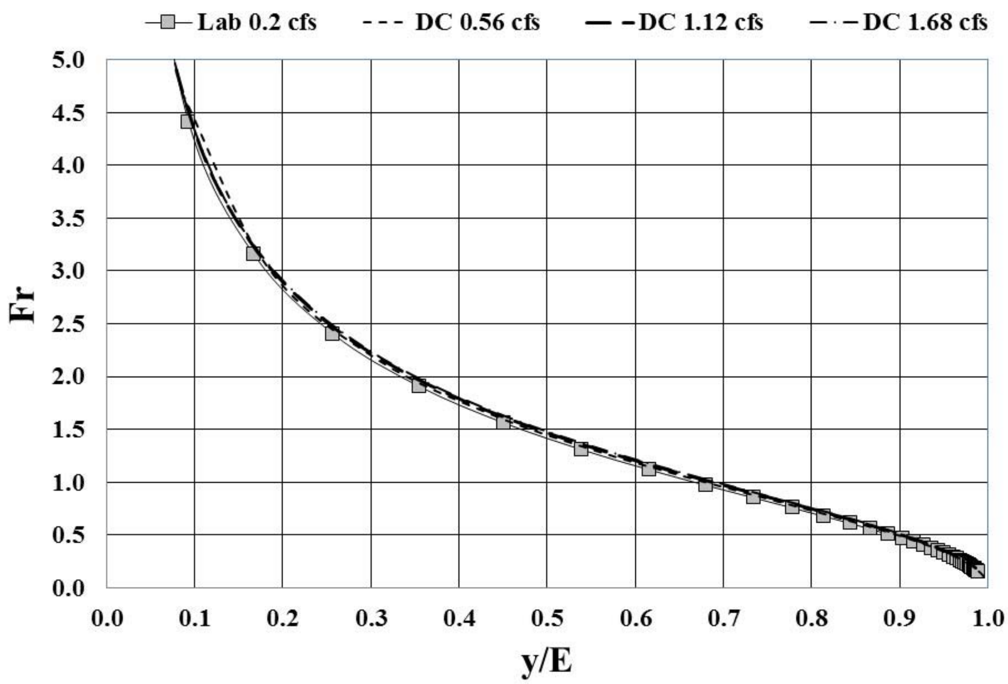
where,

Q = flow rate, cfs

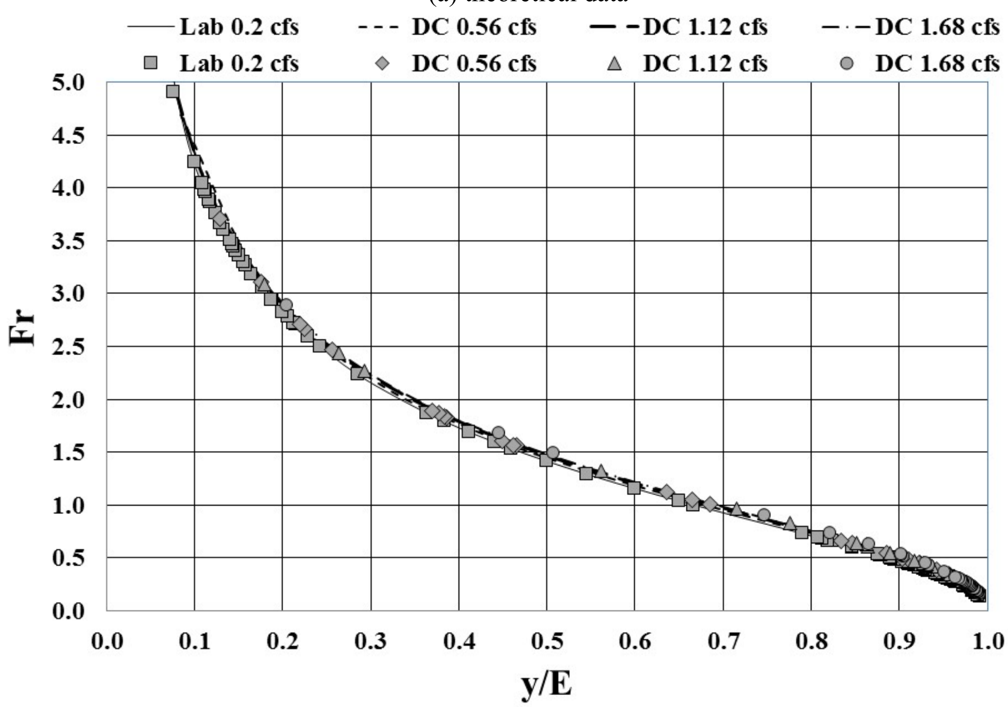
V = average velocity, ft/s

A = cross sectional area of flow, ft^2

Figure 7.16(a) shows the theoretical data points created by using the lab channel dimensions and flow rate. This figure also shows theoretical curves from using the ditch check channel dimensions and three test flow rates. Figure 7.16(b) also shows the theoretical data lines for all four channel conditions along with the actual data points collected. The data points and theoretical curves appear to show a correlation.



(a) theoretical data



(b) theoretical vs. measured data

Figure 7.16: Froude Number vs. y/E for Theoretical and Measured Data.

From testing, it has been determined that the maximum flow rate for wattle ditch checks is 1.12 cfs. This flow rate will create the maximum impoundment pool while maintaining the structural integrity of the installation. Therefore, the performance criteria for the AU-ESCTF should be based upon the data from this flow rate and this channel geometry. However, it is also important to realize that other channel geometries and other flow rates may also be used to test products, and knowing if a performance criteria can be developed from these conditions is also of interest.

The incremental, theoretical data allows the specific energy equation (E) and Froude number to be determined from the incremental water depth as each depth is related to flow rate, channel bottom width, and side slopes. Using this theoretical data, the gaps in the data which were shown from the test data can be filled and used to evaluate the functional relationship between the Froude number and y/E. The data set can be analyzed using a numerical method to take the first and second derivative of the function created by the data set. The first derivative identifies the locations of minima, maxima, and points of inflection using Equation 7.8. The dependent variable (y) is the Froude number, where as the independent variable (x) is the relationship of y/E. Therefore, Equation 7.8 represents the relationship of the change in Froude number with respect to the change in y/E.

$$\frac{\mathbf{dy}}{\mathbf{dx}} = \frac{\mathbf{\Delta Fr}}{\mathbf{\Delta(\frac{\mathit{y}}{\mathit{E}})}} \qquad \text{(Equation 7.8)}$$

where,

$\frac{\mathbf{dy}}{\mathbf{dx}}$ = first derivative of the Fr vs. y/E relationship

ΔFr = change in slope with respect to Fr ($Fr_i - Fr_{i+1}$)

$\Delta(\frac{y}{E})$ = change in slope with respect to y/E ($y/E_i - y/E_{i+1}$)

The first derivative will show the approximate location of the inflection point, however, taking the second derivative will better pinpoint the location within the data set. The second derivative can also be estimated numerically as shown in Equation 7.9.

$$\frac{d^2y}{dx^2} = \frac{\Delta[\frac{\Delta Fr}{\Delta\left(\frac{y}{E}\right)}]}{\Delta\left(\frac{y}{E}\right)} \qquad \text{(Equation 7.9)}$$

where,

$\frac{d^2y}{dx^2}$ = second derivative of the Fr vs. y/E relationship

$\Delta[\frac{\Delta Fr}{\Delta\left(\frac{y}{E}\right)}]$ = change in change of slope of the Fr vs. y/E relationship ($\frac{\Delta Fr}{\Delta\left(\frac{y}{E}\right)}_i - \frac{\Delta Fr}{\Delta\left(\frac{y}{E}\right)}_{i+1}$)

$\Delta(\frac{y}{E})$ = change in slope with respect to y/E ($y/E_i - y/E_{i+1}$)

Using the theoretical data calculated from the incremental depth, five different scenarios were evaluated to determine the affect flow rate and channel geometry has on the location of the Fr versus y/E function. Table 7.7 tabulates these scenarios and shows the resultant location of the inflection point with respect to the y/E relationship. These scenarios are the AU-ESCTF ditch channel geometry and three test flow rates, the small scale lab test setup and the test setup described by ASTM - D7208.

Table 7.7: Location of Inflection for Different Ditch Check Testing Scenarios

Scenario	Flow Rate (cfs)	Bottom Width (ft)	Side Slope (z:1)	Location of Inflection (y/E)
AU-ESCTF: 0.56	0.56	4	3	0.75
AU-ESCTF: 1.12	1.12	4	3	0.75
AU-ESCTF: 1.68	1.68	4	3	0.75
Lab Setup	0.25	1	0	0.75
ASTM D 7208	3	2	2	0.74

Table 7.7 shows the location of the inflection point is approximately located at y/E equals 0.75. This is important because the inflection point signifies a change in behavior. For this

function, the change in behavior occurs as a result of the Energy equation becoming depth dominate as velocity nears zero and the relationship of y/E moves closer to equaling 1.

Upstream cross-sections 1 through 6 represent 15 ft upstream of the ditch check. If the ditch checks were spaced based upon geometry, then the required spacing for a 1.5 ft tall ditch check would be 30 ft. Based upon this spacing, if the average Froude versus y/E falls within the criteria of y/E equal to or greater than 0.75, then this signifies that at least half the channel between each ditch check is protected by a low velocity impoundment which will help protect the channel from erosion and also create conditions favorable for sedimentation to occur. This criteria is shown in Figure 7.17 with the average performance data for each product for the flow rate of 1.12 cfs. From Figure 7.17 it can be seen that the American Excelsior excelsior wattle falls outside of this criteria. The Western Excelsior excelsior wattle's y/E average is 0.77 falls very close to the minimum requirements.

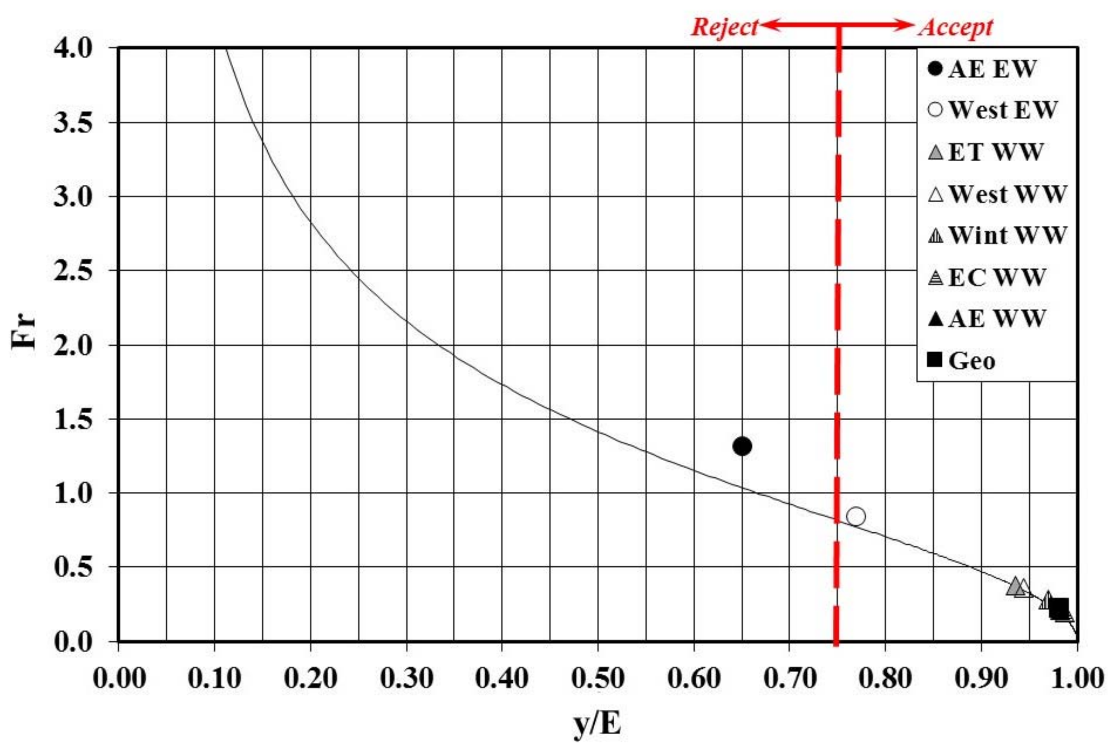


Figure 7.17: Performance Criteria Based Upon Avg. Performance and Theoretical Data

The American Excelsior excelsior wattle does not fall on the line because the Froude number to y/E relationship is not a linear line and the American Excelsior's resultant Froude number for CS1 and CS2 are very high which skews the average off the line.

The Froude versus y/E relationship has provided a criteria which ALDOT possibly other FHAs can use to communicate with manufacturers the minimum performance potential products must meet. This relationship may also allow researchers to normalize the data so that direct comparison of performance data from ditch check tests performed at different facilities using different flow rates and channel dimensions can be made by comparing the actual data to the theoretical curve. Further evaluation of this relationship is recommended to determine whether this relationship can be a useful tool for comparing data between testing facilities. Factors which may affect this relationship's functionality may include channel slope and flow rate versus bottom width.

7.5 Summary

Since wattles are manufactured in different dimensions and materials, determining the hydraulic performance of various products is a necessity for proper erosion and sediment control planning. The wattles tested for this research were manufactured using three different materials: (1) excelsior wood fibers, (2) wheat straw, and (3) synthetic carpet fibers. To determine whether or not wattle materials and dimensions affect performance simply due to their material makeup, each wattle was tested using the same tier flow conditions and installation. Combining the measured performance of depth created by each wattle and corresponding wattle density, a ratio was developed which allowed for direct comparison of wattle performance. The research showed that the excelsior wattle, wheat straw wattles, and synthetics wattles had an average density of 1.89 lbs/ft^3, 3.18 lbs/ft^3, and 1.67 lbs/ft^3 respectively. Though the synthetic carpet fiber wattle was 90% less dense than the wheat straw wattle, it was able to impound water 23%, 31% and 32% deeper than the average wheat straw wattle at the low (0.58 cfs), medium (1.12 cfs) and high (1.68 cfs) flow rates, respectively. The synthetic wattle was also 13% less dense than the excelsior wattle average, however, its impoundment depth was 153%, 112%, and 87% greater than the average excelsior wattles depth for low, medium and high flows, respectively. Though the excelsior wood fiber and wheat straw wattles are filled with two different materials, they performed similarly when comparing density to depth impoundment ratios for medium and high flow conditions. Statistical analysis showed that for medium and high flow conditions, the material fill did not significantly affect performance, and therefore density was the greatest mitigating factor for controlling runoff depth. However, for low flow conditions, it was determined that the excelsior and wheat straw wattles performed significantly different because flow was not restricted by the excelsior's flow-through properties and therefore performance was related to the material fill as well as the density.

These results are flow dependent and should only be considered for the flow ranges tested. It should also be noted that it was determined through this research that the optimum flow rate for wattles as ditch checks was 1.12 cfs based upon the test conditions of this research due to issues with structural integrity of the wattles when tested under the higher flow rate of 1.68 cfs. The greater impoundment depths created by the higher flow rates increased the hydrostatic pressure on the wattles and stakes causing some dislodgement and structural failure to occur.

Determining a performance criteria for wattle products requires understanding how wattles are intended to perform and determining the threshold in which the products no longer function in the manner intended. To accomplish this, all products were compared analytically using open channel flow principles with the intended goal to create hydraulic conditions that decrease velocity, increase water depth, and therefore create conditions favorable for sedimentation to occur. This was accomplished through evaluating the average performance of each ditch check by comparing the products resulting flow depths and velocity. Using this measured data, the specific energy developed by the impoundment and the hydraulic condition, which can be measured by the Froude number, was determined and a performance relationship was created. This relationship compares the Froude number and the depth of flow (y) divided by the specific energy for each upstream cross section. Using this relationship, the recommendation average minimum depth to specific energy ratio of 0.75 would be required for the ditch check to be included as a practice for ALDOT construction projects for products tested at the AU-ESCTF. This relationship may also be used to compare data between test facilities with different test channel geometries and flow rates.

CHAPTER EIGHT

COST ANALYSIS

8.1 Introduction

Ditch checks come in many different forms, sizes, and are made of differing materials. Cost can be affected by material composition, quantity, labor required for installation, overall performance, and maintenance requirements. Material cost is easily quantifiable and is market driven. The quantity depends on whether the ditch check is prefabricated or is constructed on-site. Prefabricated ditch checks such as wattles are bound by their manufactured dimensions. The labor required for each installation depends on the ease of installation, the learning curve associated with installing a new device, and equipment requirements. These factors can be quantified using historical data and engineering design. However, performance and maintenance requirements may not be known or completely understood, and therefore may actually govern the economic efficiency of using ditch checks on construction sites. Properly protecting the channel from erosion may minimize labor requirements for channel erosion maintenance. Ease of maintenance may also limit the labor required to maintain the device. The silt fence installation's V-shape could inhibit equipment from removing upstream sediment if the sediment is deposited directly upstream of the ditch check. Therefore it is important that the ditch check properly impound water so sediment can be deposited further upstream of the device which will allow greater ease of sediment removal.

8.2 Cost of Ditch Checks

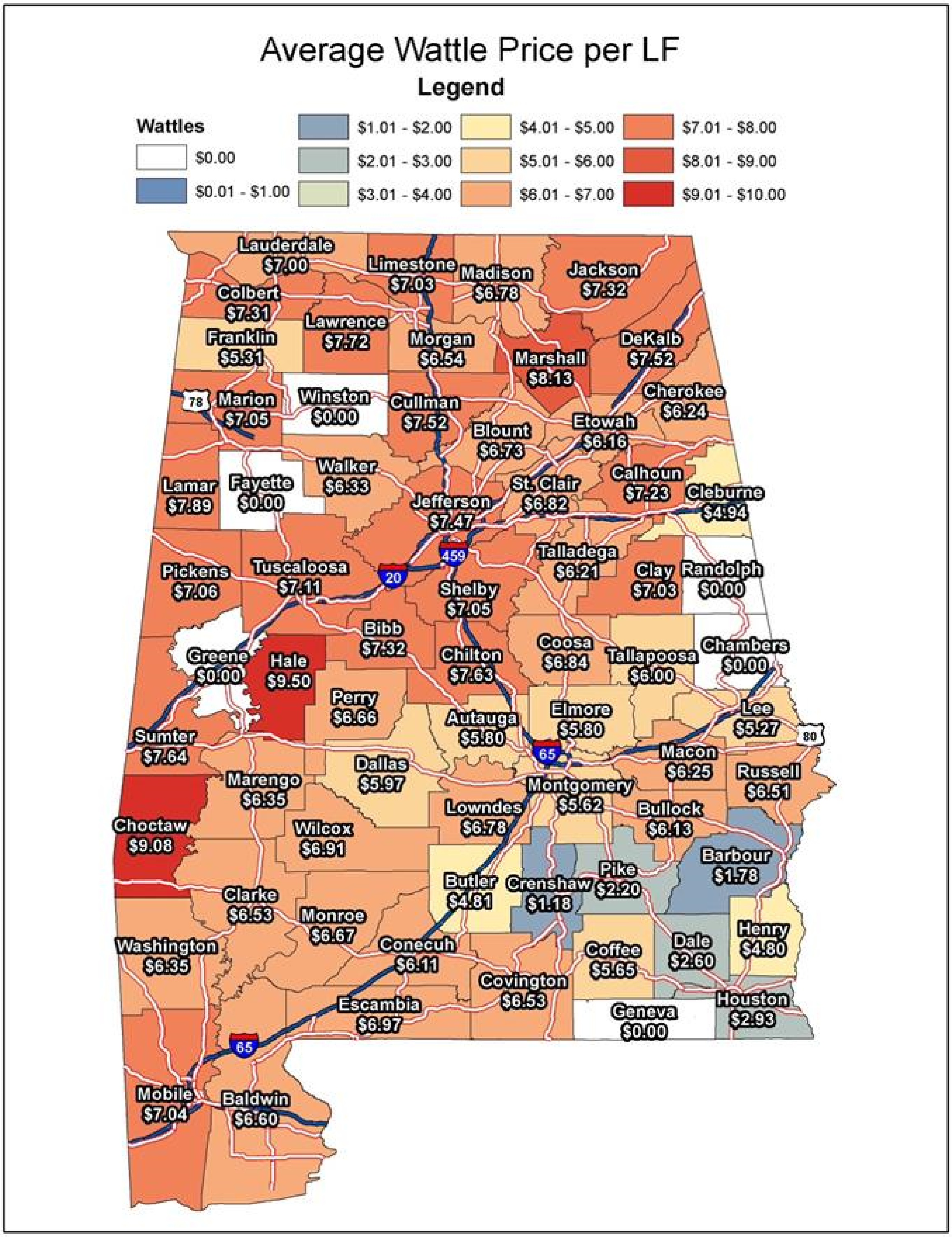
Ditch checks vary beyond just the structure that is used to impound water. Materials required to further enhance installation by securing the products and by armoring the channel near the ditch check to minimize scour adds cost but also increasing performance. This cost increase is particularly evident when comparing the improved ditch check installations recommended in Chapter 6 of this dissertation to the standard ALDOT installations. These improved installations were developed by using the approach that successful installations maximize structural stability and minimize damage to the channel as a result of erosion near the ditch check which can lead to undercutting and decreasing the effectiveness of the device. Accomplishing this typically required adding additional materials to the installation to increase structural stability and to armor the channel against scour in the vicinity of the ditch check installation.

8.2.1 Unit Cost

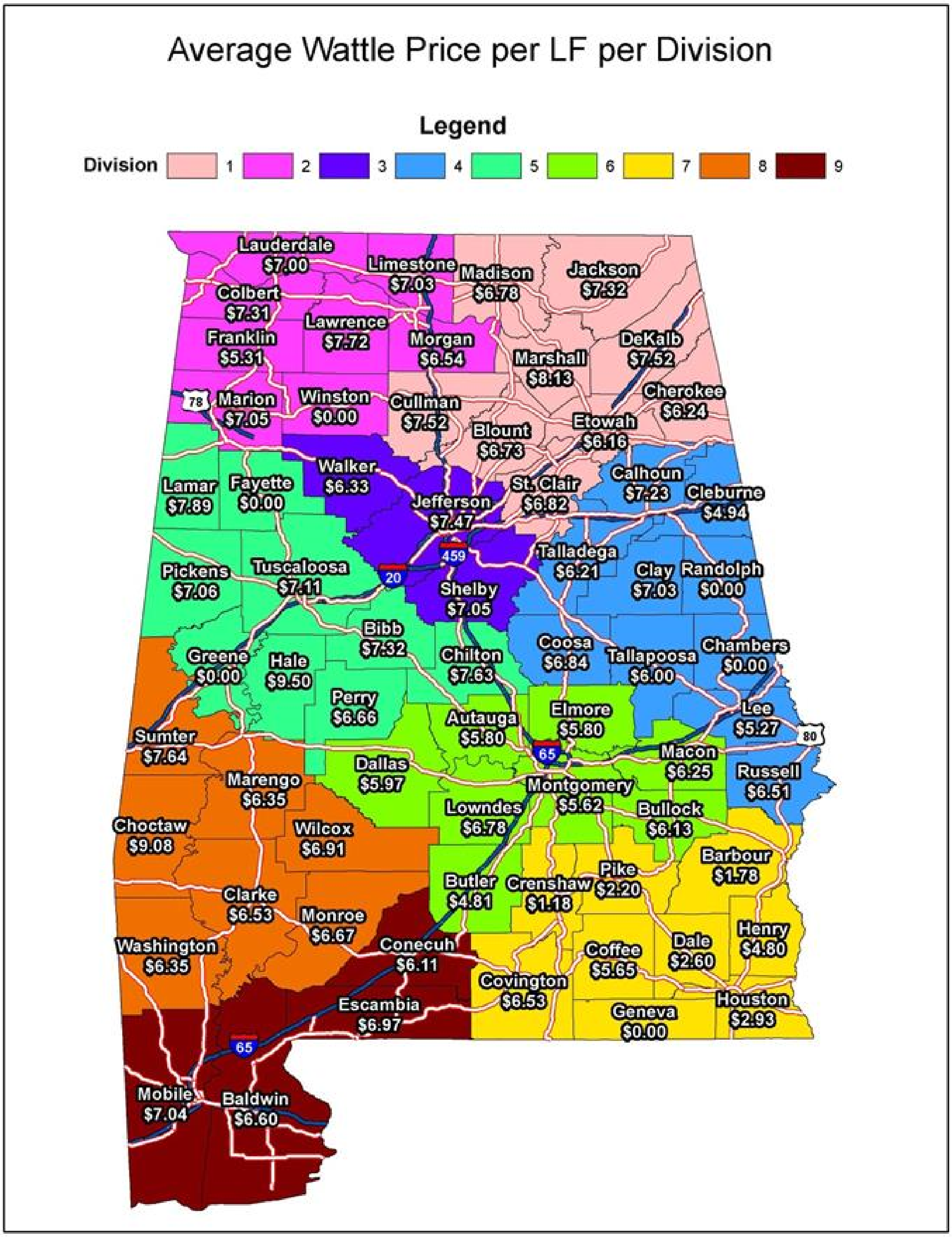
ALDOT maintains a database that records historical material costs used during construction with the ALDOT Items Bid Summary which was used as means to evaluate practice costs that are representative of industry costs within the southeast U.S. The October 1, 2012 summary was referenced for this study. The items listed in this summary are specified in unit cost (i.e., per linear foot (lf), cubic yard (cy), square foot (sf), ton, etc.). This summary is categorized by materials used for specific projects and includes the average unit cost of the material. This data is also further categorized by county and ALDOT division within the state. The materials used in this research that are quantified by ALDOT cost data include: wattles, silt fence, nonwoven geotextile, riprap, and sand bags.

8.2.1.1 Wattle Cost Analysis

Standard installation materials such as wattles and grade stakes used to secure wattles in place as well as labor costs for installation are included in the unit cost price. It is assumed that the unit cost is based upon the standard ALDOT wattle ditch check installation, therefore the price covers the cost of the wattles, the stakes required to secure the wattle in place and the labor to install the device. The average cost of wattles for the state of Alabama was $6.52 per LF. ALDOT specifies a 20 ft long wattle to be used for ditch checks, therefore the cost of the standard ALDOT installation for the state is $130.40. The 8 oz. nonwoven filter fabric underlay falls into the *filter blanket* category. The state of Alabama average for this is $3.69 per yd^2. The underlay is 7.5 ft by 15 ft which results in 12.5 yd^2 of fabric. This addition will increase the new ALDOT wattle installation to $176.53, a cost increase of 35%. ArcMap by ESRI® was used to map the wattle cost per linear foot for each county and division and is shown in Figure 8.1.



(a) wattle cost per LF for each AL county



(b) wattle cost per LF per ALDOT division

Figure 8.1: Unit Cost of Wattles per Linear Foot for ALDOT Construction Projects.

The cost per linear foot varies by county and division as evident by Figure 8.1. Table 8.1 shows the average cost of the new installation for each division.

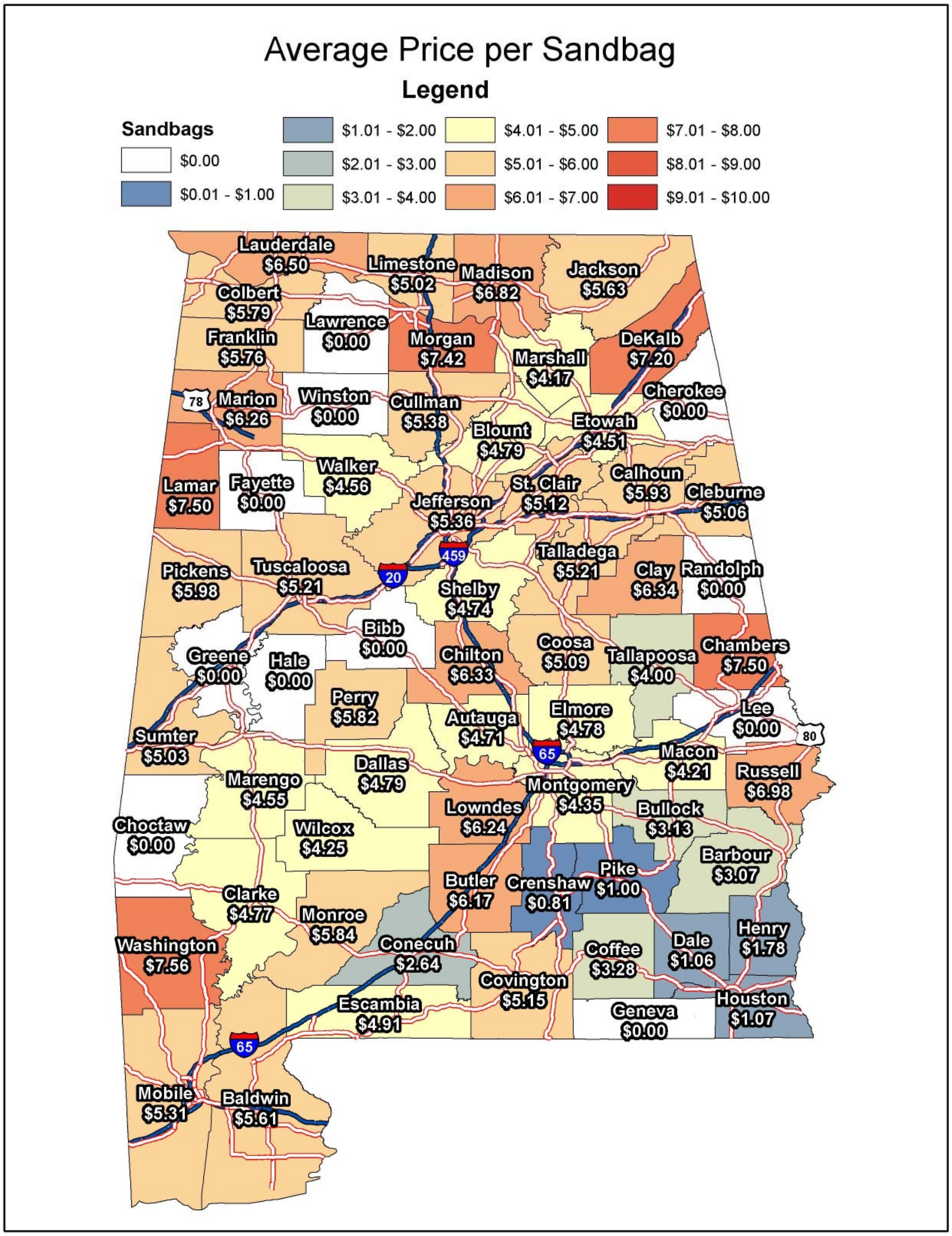
Table 8.1: Cost Analysis of New Wattle Installation per ALDOT Division

ALDOT Division	Cost ($ per LF)	Cost of New Installation
1	$6.95	$169.74
2	$6.88	$168.34
3	$7.17	$174.14
4	$6.55	$161.74
5	$7.47	$180.14
6	$5.80	$146.74
7	$4.29	$116.54
8	$6.79	$166.54
9	$6.46	$159.94

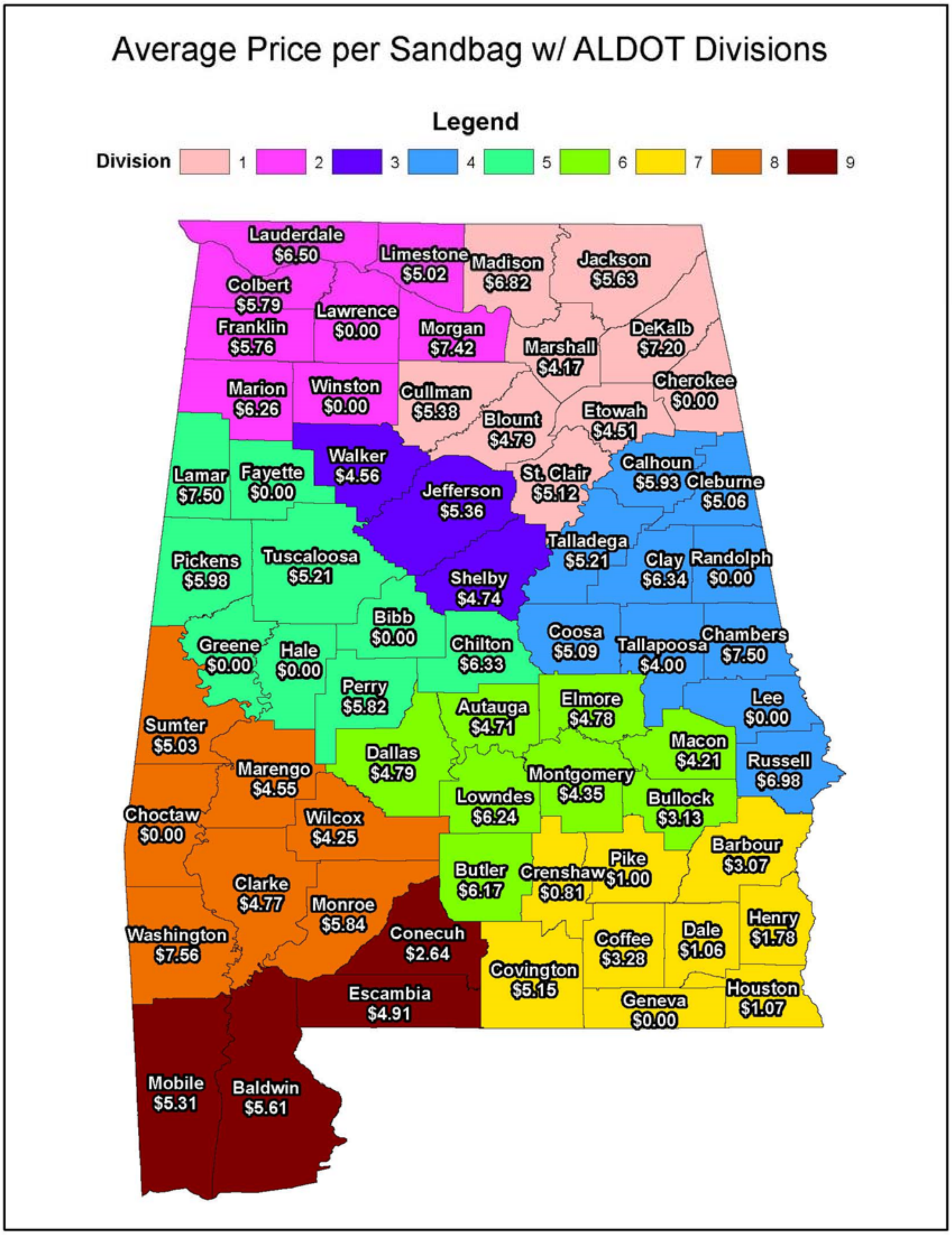
Division 7 reports a much lower cost for wattles when compared to other ALDOT divisions. If the average wattle cost for Division 7 is removed from the average state cost, then the average wattle cost per installation increases to $181.31, an average increase of $4.77 per new installation. ALDOT is currently evaluating this discrepancy.

8.2.1.2 Sand Bag Cost Analysis

It was suggested by ALDOT that sand bag ditch checks could be a viable option for replacing wattle ditch check if the cost difference decreased the ditch check cost. The average cost of sand bags was also evaluated per county and division and is shown in Figure 8.2.



(a) sand bag cost for each AL county



(b) sand bag cost for each ALDOT division

Figure 8.2: Unit Cost of Sand Bags per Bag for ALDOT Construction Projects.

The state average per sand bag for the state was $5.07 per bag. The average measured length of sand bags is 1.38 ft. Since the standard installation stacks the sand bags 5 bags high (two bags for the bottom and middle layer and one bag for the top layer), there are approximately 3.62 bags per linear foot. The sand bag ditch check as installed at the AU-ESCTF is 10 sand bags long which is approximately 13.8 ft long. This means that the sand bag ditch check costs $18.35 per linear foot, an increase of 181%. The total standard ALDOT sand bag installation is $278.85. Using the burrito method would increase the cost by $42.23, an increase of 15% from the standard installation. Adding the reinforcement bags to the downstream side of the ditch check would increase the cost by $125.08, an increase of 45% to the standard installation. Due to this cost increase and the fact that sand bags had similar structural integrity issues as wattles for high flow tests, sand bag ditch checks are not recommended to be a replacement for wattle ditch checks.

8.2.2 Further Ditch Check Cost Comparisons

Riprap and silt fence ditch checks are also used by ALDOT for construction site channelized stormwater control. The reported silt fence cost per linear foot is based upon silt fence installed as a perimeter control with post spacing of 10 ft. Often machinery can be used to trench-in the fence which can minimize labor costs and increase production. This creates a problematic comparison since silt fence ditch checks are more difficult to install using machinery due to the channels side slopes. Silt fence ditch checks also require more posts for structural stability in comparison to silt fence used as a perimeter control. The installation used in this study had a post spacing of 2.5 ft which means 3 extra posts are installed every 10 ft of silt fence when compared to the standard perimeter control silt fence installation. It is difficult to access this cost increase based upon the data in the ALDOT bid summary when the increased labor requirements is factored in for possibly installing the silt fence ditch check by hand and requiring three additional posts per 10 ft of fence be driven into the ground and the wire backing attached to each post. However, taking this into consideration, the cost of an ALDOT silt fence ditch check if using the perimeter installation cost would be $121.80. This cost is $54.73 lower than the new wattle installation cost, however, it could be assumed that this cost would increase if a specific cost item for silt fence ditch checks was reported due to the increase in materials and labor of the silt fence ditch check versus the perimeter control installation.

Riprap is typically reserved for higher flow rates and cost can differ based upon the installation configuration and the stone gradation used for the ditch check. ALDOT specifies the stone gradation should be determined based upon the expected flows. Class II riprap is larger than Class I riprap, however the installation cost is comparable at $239.95 and $230.47 respectively, a difference of only $9.48 per installation with filter fabric choker. The filter fabric choker adds an additional cost of $61.50 per installation.

8.3 Cost Comparisons versus Performance

Material and installation can affect how costly a specific ditch check can be to specify for use on a construction project. However, specific design elements can also affect how economically efficient ditch checks are utilized in various situations. Using a ditch check that impounds a greater amount of water could limit the number of ditch checks required to meet the erosion and sediment control needs of a specific drainage ditch. However, the cost increase for the ditch check that creates the longest impoundment length maybe greater than the cost for installing more ditch checks that are less efficient at impounding water. Figure 8.3 shows the spacing requirements for ditch checks based upon the ditch check height and the channel slope. The dotted line that correlates with a spacing of 100 ft shows the minimum allowable spacing by ALDOT for all ditch checks regardless of ditch check height or channel slope. This was originally explained as means of reducing cost.

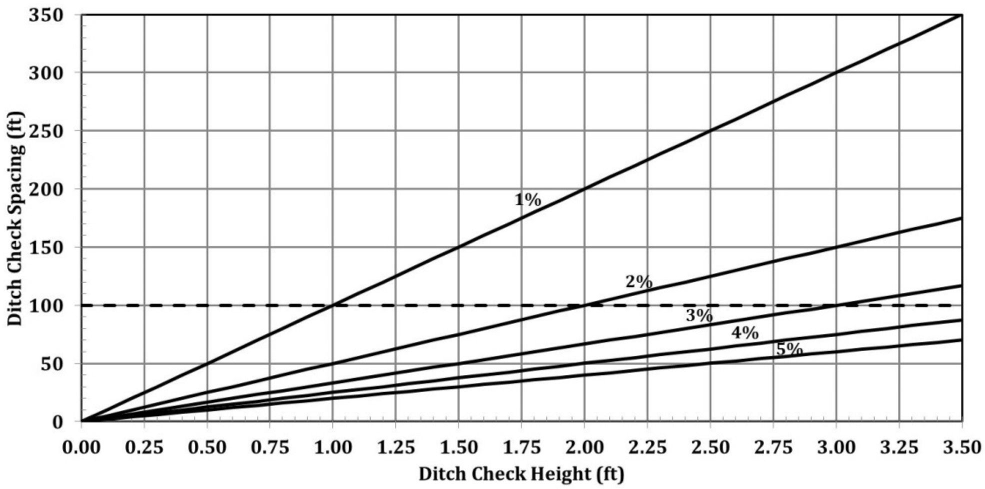


Figure 8.3: Spacing Guide Based upon Channel Slope and Ditch Check Height.

However, geometric spacing does not take into consideration actual performance. It has been shown that wattles impound water to different degrees under same flow rates. This is due to the inherit flow-through properties of these products. If wattle ditch checks were spaced based upon geometry, for a 5% ditch, the ditch checks would be spaced at a distance of 30 ft. Table 8.2 shows that based upon impoundment capabilities, more wattles are required to maintain subcritical flows throughout the channel reach for wattles that do not impound water correctly.

Table 8.2 Wattle Ditch Check Performance versus Spacing Requirements

Product		***Avg. Impoundment Length (ft)***	***# of Ditch Checks per 30 ft***	***Avg. Impoundment Length (ft)***	***# of Ditch Checks per 30 ft***
		Low Flow		**Medium Flow**	
Excelsior	**AE EW**	1.5	20	7.7	3.9
	West EW	8	3.8	11.7	2.6
Wheat Straw	**Ero Tech**	11.8	2.5	18	1.7
	West WW	16.5	1.8	19	1.6
	Winters	20.9	1.4	28.4	1.1
	AE WW	22.7	1.3	27.7	1.1
	East Coast	24.7	1.2	27.3	1.1
Synth.	**GeoHay**	21.6	1.4	31	1

Recognizing that a 1.5 ft. tall ditch check in a 1% channel slope should impound water 150 ft, Table 8.3 was developed to show the cost difference for the different ditch checks tested if they were used in a 150 ft long channel, installed based upon the new installation parameters, and spaced based upon geometry only. For a 1.5 ft tall ditch check, an increase slope grade % results in an increase of one required ditch check per 150 ft (i.e., 1 ditch check for 1% slope is required every 150 ft, 2 for 2% slope, 3 for 3% slope, etc.)

Table 8.3: Cost of Channel Erosion Control (per 150 ft)[1]

Slope	Wattle[2]	Riprap		Silt Fence[3]	Sand Bags	Rolled Erosion Control Product				
		Class I	Class II			C2	C4	C6	C8	C10
1%	$176.53	$230.47	$239.95	$121.80	$446.28					
2%	$353.06	$460.94	$479.90	$243.60	$892.56					
3%	$529.59	$691.41	$719.85	$365.40	$1,338.84	$171.00	$355.00	$331.00	$357.00	$380.00
4%	$706.12	$921.88	$959.80	$487.20	$1,785.12					
5%	$882.65	$1,152.35	$1,199.75	$609.00	$2,231.40					

Note:
1. MFE-I
2. 20 FT LONG WATTLE
3.BASED UPON ALDOT SILT FENCE SPECS (Testing Material Costs $53.42)

Table 8.2 also shows the cost of installing 150 ft of rolled erosion control products (RECP) based upon approximate expected shear stresses for the channel bottom. The values of 2 through 10 in the listed RECPs of C2 to C10 refer to the amount of approximate shear stress the RECP is designated to withstand (i.e., C2 can withstand up to 2 lbs/ft^2 of shear stress, C4 up to 4 lbs/ft^2, etc.). ALDOT calculates approximate shear stress using Equation 8.1

$$\boldsymbol{\tau = gDS_b} \qquad \text{Equation 8.1}$$

where,

τ = approximate shear stress (lbs/ft^2)

g = unit weight of water (62.4 lbs/ft^3)

D = maximum expected depth of water in the channel (ft)

S_b = slope of channel bottom (ft/ft)

Based upon Table 8.3 the wattle ditch check is cost comparable only to the C2 for a channel slope of 1%. The cost of the filter fabric does increase the overall cost of the wattle installation which is inherit to all of the new installations that use filter fabric. Alternatives to using filter fabric as an underlay could result in decreased cost of the installation and should be further investigated as long is performance is not reduced. The silt fence ditch check is the cheapest alternative based upon the ALDOT bid summary data, however, it should be noted that this installation cost which is based off on installing the silt fence as a perimeter control does not mimic the installation requirements a silt fence ditch check. Cost should be assumed to increase with the use of the silt fence ditch check installation. This cost increase should also be further investigated to properly quantify these cost differences. Using riprap is a cheaper alternative to the C4 through C10 RECPs for channels at a 1% slope. However, as the channel slope increases, the RECP cost becomes more economical.

8.4 Summary

Performance of the ditch check should be taken into consideration when specifying which ditch check or product to be used on the construction site. However, typically policy or geometry determines spacing requirements. These spacing requirements can affect the cost associated with specifying a particular product or practice. When based upon geometry only, the channel slope and ditch check height will affect the overall cost of the erosion and sediment control provisions because these factors will influence the spacing requirements to ensure the channel is properly protected. Based solely on cost parameters and spacing, wattles are a comparable practice for channels with a longitudinal slope of 1%. Riprap is also a comparable product for channels at 1% if flow is expected to be high, however, riprap should not be used in low flow conditions due to the cost requirements. Silt fence ditch checks perform adequately as ditch checks, however, actual cost of silt fence ditch checks was not available based upon the ALDOT bid summary data, and therefore a direct comparison could not be made. Sand bags are the most expensive practice and should only be used in instances where other alternatives are not able to be installed or may not meet performance capabilities.

Though the previous comparisons to the RECPs shows that for controlling erosion in the channel, it is often more cost effective to stabilize the channels with RECPs, erosion control is not the only benefit of ditch checks. Though their primary function should be for controlling channel erosion, minimizing velocity, and increasing water depth also creates conditions suitable for controlling sediment as well, and is a secondary benefit to wattles which RECPs cannot mimic. Therefore, choosing the proper ditch check practice for implementation on construction sites should depend on a number of factors which include channel geometry, expected flow conditions, cost, and functional requirements.

CHAPTER NINE

CONCLUSIONS AND RECOMMENDATIONS

9.1 Summary and Recommendations

Selection of erosion and sediment control practices on construction sites requires a need for understanding the functionality of various practices. Ditch checks are typically used in stormwater runoff channels as a means to dissipate water velocity and create impoundment. This is necessary to protect earthen channels from stormwater induced erosion and to create conditions conducive for sedimentation. With new, more stringent EPA guidelines for erosion and sediment controls, it was important to test current ditch check practices for the purpose of evaluating and improving overall performance. Due to the influx of new manufactured devices geared towards the erosion and sediment control industry, a need was also identified for determining a performance criteria for ditch checks to meet when evaluating products as ditch checks.

Four ditch check practices were evaluated for the purpose of this study: (1) wattles, (2) silt fence, (3) riprap, and (4) sand bags. These practices are all currently used on ALDOT construction projects, however, proper understanding of their functionality was not completely known. Two criteria were used to evaluate each ditch checks performance: (1) structural stability, and (2) impoundment capability.

9.2 Installation Evaluation and Improvements Summary

The standard ALDOT wattle installation did not appear to be structurally unstable once tested, however the impoundment capabilities were limited and undermining was a concern. After testing seven different installation configurations, a new installation was recommended to ALDOT which required the use of an underlay to minimize channel scour and undermining as well as the use of sod staples to pull the wattles netting downward against the channel bottom, increasing ground contact and further decreasing undermining. However, ALDOT rejected the recommended use of the staples due to concerns with proper installation and inspection. ALDOT did adopt the use of a filter fabric underlay as a means for reducing undercutting as well as the new teepee staking pattern that was tested. The teepee staking pattern does not require piercing the wattle to secure it in place and therefore is nondestructive which could increase the longevity of the installed product. This new staking pattern alone did not, however, significantly increase the devices hydraulic performance.

Though the standard silt fence V-installation impounded water adequately, the height associated with the installation was a concern at 32 in. The standard installation impounds a substantial amount of flow and could limit its functionality in shallower channels where impounded flow could overtop the channels banks. Therefore an installation which mimics the TDOT enhance silt fence check that uses a weir cut 18 in. high across the width of the channel bottom was tested. This installation reduced the amount of water the ditch check impounded which could increase the structural longevity of the device, decreasing maintenance requirements. The new installation was also tested with a method that pinned the toe of the silt fence to the channel bottom instead of trenching the excess fabric. This installation option was tested to determine the longevity of this practice. Over course of 6 tests, no piping occurred and approximately 91% of sediment by volume was retained using this technique and the TDOT installation.

Riprap ditch checks are meant for higher flow and velocity conditions, and therefore, the structural integrity of the device was not necessarily a concern. However, due to the size and shape of the stones, flow-through was a concern due to the large pore passages. ALDOT typically recommends choking, which is a means of clogging the pore passages and forcing the impoundment to overtop. ALDOT recommends placing either smaller aggregate or filter fabric on the upstream side of the ditch check to reduce flow-through. However, it was not completely understood how well either of these two applications functioned. Both choking methods were tested and compared against a riprap ditch check installation with no choker. Using modified no. 4 stone, the choker increased the impoundment length by 47%. Using the filter fabric choker, impoundment was increased by 98%. Therefore, it is the recommendation of this study that filter fabric should be used as a choking method to maximize the impoundment capabilities of the ditch check.

Sand bag ditch checks were also evaluated. The dense structure and shape of sand bags create a structure that easily impounds water. However, since no device is used to secure the bags in place, friction and the weight of the sand bag is required to hold the structure in place. Through testing, it was determined that the standard ALDOT ditch check was capable of impounding water for low flows, but a flow of 1.12 cfs consistently dislodged the bags causing complete structural failure. A method that wraps the bags in filter fabric has already been employed on some ALDOT construction sites and garnered good results. This new sand bag installation method was also tested and showed that the installation could withstand the force caused by the 1.12 cfs flows and also withstood the forces exerted by the 1.68 cfs flow the majority of the time. However, 1 out of 4 installations failed at the high flow rate. Therefore a modification to the bag placement in an attempt to further stabilize the ditch check was also tested. The new reorientation of the middle layer of sand bags and also the placement of reinforcing bags on the downstream side made the installation capable of withstanding the flow rates for all the replicate tests and is the recommended installation.

9.3 Product Performance Criteria

A product performance criteria for which to compare manufactured ditch check devices was also developed from this research. To determine this criteria, 8 products that are currently allowed on ALDOT construction projects were tested using the newly adopted wattle installation. These products ranged in material properties as well as product dimensions. Two excelsior wood fiber filled wattles, five wheat straw filled wattles, and one recycled carpet fiber wattle product were tested. Through this testing it was discovered that density played a role in product performance when comparing the excelsior and wheat straw filled wattles. The carpet fiber wattle was the least dense product tested, however this product was capable of greatly impounded water due to the absorption capabilities of the carpet fibers and performed as efficiently as the densest wheat straw wattles. The less dense excelsior wattles, however inadequately impounded water in low flow conditions. It was determined that in low flow conditions, the excelsior wattles did not impound water as efficiently as the wheat straw wattles when considering the products density. It was concluded that this was due to the porous structure of the excelsior wattles. During low flow testing the flow through rate of the excelsior wattles was not greatly exceeded and water was not impounded adequately. In the medium and high flow conditions, impoundment improved for the excelsior wattles when comparing the density to flow relationship as the flow through capabilities of the products were exceeded and impoundment increased.

Using the measured water velocity and depths for each test, average performance for each product was developed. This data was used to develop the HGL:EGL relationship, y/E relationship, and the Froude number for each upstream cross-section. This data yielded a criteria which can be used for wattle product performance. Realizing that the optimum conditions that are created by ditch checks, are long subcritical pools of water that have low velocity and high impoundment depths, a product performance criteria is listed in Table 9.1. This criteria is based upon the function of a line fitted to theoretical data and confirmed by the actual data collected. These lines contain an inflection point which is located near y/E = 0.75. This inflection point signifies the point at which the function's behavior is altered due to the low flow velocity and the impoundment depth now governing the shape of the function.

Table 9.1: Required Ditch Check Product Performance Criteria

Hydraulic Performance Relationships	
Required Average Performance	*y/E*
	minimum of 0.75

Ditch checks can be tested and compared to this criteria in Table 9.1 before being used on construction projects. This criteria reflects the devices capability of impounding water, reducing flow velocity which results in less channel erosion and creates conditions favorable for sedimentation to occur.

9.4 Cost Summary

Ditch check applications are dependent not only on performance but also cost and functionality. The cost of the four ditch check installations developed and recommended were evaluated using ALDOT bid summary data. Cost for each device was developed and compared to the cost of stabilizing the channel with a rolled erosion control product. This is shown in Table 9.2.

Table 9.2: Cost of Channel Erosion Control (per 150 ft)[1]

Slope	Wattle[2]	Riprap		Silt Fence[3]	Sand Bags	Rolled Erosion Control Product				
		Class I	Class II			C2	C4	C6	C8	C10
1%	$176.53	$230.47	$239.95	$121.80	$446.28					
2%	$353.06	$460.94	$479.90	$243.60	$892.56					
3%	$529.59	$691.41	$719.85	$365.40	$1,338.84	$171.00	$355.00	$331.00	$357.00	$380.00
4%	$706.12	$921.88	$959.80	$487.20	$1,785.12					
5%	$882.65	$1,152.35	$1,199.75	$609.00	$2,231.40					

Note:
1. MFE-I
2. 20 ft long wattle
3. Based upon ALDOT silt fence specs (Actual testing material costs $53.42)

Though the cost data shows that ditch checks can be more expensive to implement, especially in channels of greater slope, it should be noted that ditch checks are capable of creating conditions favorable for sedimentation which is a secondary benefit that RECPs do not emulate. Therefore, the cost of the product should be taken into consideration along with the desired functionality. Based upon the cost data, sand bags should not be used as replacement to other ditch checks to their overall cost unless it is deemed necessary by the expected conditions.

9.5 Conclusions

Based upon this research, some important conclusions have been drawn.

1) The Auburn University-Erosion and Sediment Control Testing Facility was designed, constructed, and used to perform this research. A testing methodology was successfully developed to scientifically test ditch check practices. This facility will be a platform for future research, testing, product evaluation, and training efforts in the erosion and sediment control industry.

2) Ditch check performance is related to the installation being used and the material properties of the device. Performance should be based upon the device capability to withstand forces resulting from flow yet capable of impounding water efficiently so that water velocity is adequately reduced and impoundment areas are created. Using an underlay or splash pad underneath or directly downstream of the ditch check will greatly increase device impoundment capability by decreasing scour and undermining. Limiting the flow or the impoundment depth is important for maintaining the structural integrity of ditch checks. A maximum flow rate of 1.12 cfs for wattles was determined through this research due to the structural instability of installation for wattles with high densities tested at higher flow rates. It was deemed more important to control flows that are produced by storm events less than the 2-year, 24 hour storm event since these types of storms result in flow rates are more frequent than the 2-year, 24 hour storm events. This assumption takes into consideration that substantial stormwater control measures are used downstream in the event that a structural failure occurs in a larger flow situation.
3) Wattle product performance criteria can be developed using the Froude number to y/E relationships in order to quantify the ability of a device to impound water effectively. The performance criteria developed for ALDOT, which may also be used by other SHAs, uses a minimum average y/E ratio of 0.75. This relationship may also be used to compare performance of ditch check practices between testing facilities with different channel dimensions.
4) Cost of ditch checks vary by materials, labor, and installation alternatives. Excessive cost can be caused if these devices are over used or improperly used. Channel stabilization using RECPs should be considered to balance cost limitations as the number of ditch

checks required increase as slope increases. However, it should be recognized that channel stabilization with RECPs will not assist in any sediment control concerns and therefore other measures should be in place if ditch check practices are not used.

9.6 Limitations and Recommendations for Future Work

This research attempted to quantify ditch check performance based upon testing these devices as singular practices whereas typical field situations require these devices be installed in series. A large-scale test section and methodology should be developed for testing and quantifying ditch checks in series. This will help further understand the benefits or drawbacks of these devices and practices.

Further investigation into the sediment control aspect of ditch checks should also be considered and could be tested in conjunction with the previously suggested tests of ditch checks in series. This is especially important due to the environmental impacts construction sites have on downstream watersheds. Testing these products in series will allow practitioners to understand sediment removal capacities and allow them to design erosion and sediment control plans accordingly. However, sediment introduction by mechanical means makes mimicking natural sediment-laden flows difficult. Over saturation of the test flows by mechanically adding the sediment to clean water is a concern and can bias the sediment retention results as sediment drops out of suspension due to the over saturation and not necessarily by alteration of the flow properties. The optimal conditions for testing sediment removal capacity would be to produce runoff in sheet flow across a bare earthen slope and collect this runoff in an earthen channel with a series of ditch checks installed. This would create ideal sediment-laden flow conditions to determine optimal sediment removal. Optimal sediment removal capabilities can then be evaluated to determine whether these can be developed by the hydraulic conditions created by the devices or in conjunction with a flocculation product that assists the functionality of the ditch check device for promoting sedimentation in sediment-laden stormwater runoff.

This new large-scale testing methodology may also help verify other results that were found to be unlike what is actually encountered on construction sites. Most notably the deposition pattern created by the ditch check installation test. Typically sedimentation is found directly upstream of the ditch check on construction sites, however, deposition during testing occurs well upstream of the ditch check for most devices tested and coincides with the location of the hydraulic jump. The American Excelsior excelsior wattle sediment-laden test resulted in sediment deposition directly upstream of the ditch check, however, which may coincide with the products lack of hydraulic performance to create a long impoundment. It is uncertain if the deposition pattern which typically occurs near the hydraulic jump will translate to the field when ditch checks are installed properly on construction sites. If the minimization of undercutting is created in the field which creates the maximum impoundment capabilities, this pattern may translate. However, sediment could be transported and deposited directly upstream of the ditch check during very low flow conditions when minimal impoundment is created. Flow could also be entering the channel at points perpendicular to the channel which results in large sediment particles being discharged directly downstream of a disturbed slope at all points within the channel. This may also be causing larger particles deposition directly upstream of the ditch check in the field.

Though this new testing methodology may be more labor intensive, the scale required to perform such testing may also create conditions much more similar to field conditions. The scale of this testing would most likely require the use of large, heavy construction equipment. Using this equipment would allow researchers to install and test products under conditions more similar to construction sites using methods similar to the ones field installers use.

It is apparent from this research that ditch checks are only as effective as the installation and application. The practices developed in this dissertation should lead to better ditch check performance and applications for designers as well as give product manufacturers and independent product testers a performance criteria that has been lacking in the industry. This research should set the foundation for possible future research for maximizing performance of ditch checks on construction sites. This should also lead to a better understanding of .the functionality of ditch checks and result in improved implementation and performance on construction sites for reducing sediment discharge into nearby watersheds.

9.7 Acknowledgements

This dissertation is based on a study sponsored by ALDOT. The authors gratefully acknowledge this financial support. The findings, opinions, and conclusions expressed in this dissertation are those of the authors and do not necessarily reflect the view of the sponsor.

REFERENCES

1. Alabama Department of Transportation (ALDOT). *Standard specifications for Highway Construction.* Montgomery: Alabama, 2012.
2. America Society of Civil Engineers (ASCE) and Water Environment Federation (WEF), *Design and Construction of Urban Stormwater Management Systems*, ASCE Manuals and Reports of Engineering Practice No. 77, WEF Manual of Practice FD-20, 1992.
3. American Excelsior Company: Earth Science Division. *Technical Support Library: Curlex Sediment Logs Channel,* http://www.americanexcelsior.com/erosioncontrol/library/CAD%20Details/Curlex%20Sediment%20Logs%20Channel.PDF. Accessed July 30, 2013.
4. ASTM Standard D7208, 2006, *Standard Test Method for Determination of Temporary Ditch Check Performance in Protecting Earthen Channels from Stormwater-Induced Erosion*, ASTM International, West Conshohocken, PA, 2007.
5. ASTM Standard D698, 2007, *Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort*, ASTM International, West Conshohocken, PA, 2007.
6. Benik, S.R., Wilson, B.N., Biesboer, D.D., Hansen, B., Stenlund, D. (2003). *Performance of Erosion control Products on a Highway Embankment.* Transactions of the American Society of Agricultural Engineers, 46 (4) 1113-1119.

7. Bidelspack, D.A., Jarrett, A.R., and Vaughn, B.R. (2004). *Influence of Increasing the Delay Time between the Inflow and Outflow Hydrographs of a Sediment Basin.* Transactions of the American Society of Agricultural Engineers. 42 (2) 439-444.

8. Brater E.F., H. W. King, J. E. Lindell, C. Y. Wei, *Handbook of Hydraulics,* 1996.

9. Cabalka, Dwiht A and Paul E Clopper. (1997) *Design, Construction, and Operation of a State-of-the-Art Erosion Technology Test Facility*: American Excelsior Company. Proceedings of IECA, 28th Annual Conference. Nashville, TN: Inter. Conf. Proc.7.

10. California Department of Transportation (CalTRANS), *CalTRANS Erosion Control New Technology Report*, June 2003.

11. Clean Water Act, section 101(a), 33 U.S.C. 1251(a).

12. Cleveland, T.G., and A. Fashokun. 2006. *Construction-associated solids loads with a temporary sediment control BMP*. American Society of Civil Engineers, Journal of Construction Management, Vol. 132, No. 10, pp 1122-1125.

13. Crowe, C. T., D. F. Elger, and J. A. Roberson. *Engineering Fluid Mechanics 7th Ed.* New York, NY. 2001

14. Dalton, J. Evaluation of Check Dam Systems for Erosion and Sediment Control at Military Facilities, Public Works Technical Bulletin 200-1-97, U.S. Army Corps of Engineers, Washington, D.C., May 2011.

15. Davies-Colley, R.J, and Smith, D.G. (2001), *Turbidity, Suspended Sediment and Water Clarity: A Review*, Journal of the American Water Resources Association. 37 (5) 1085-1101

16. Eli, R. (1986). *V-Notch Weir Calibration Using New Parameters*. Journal Hydraulic Engineering, 112(4), 321–325.

17. Faucette, L. B., et al (2005). *Evaluation of Stormwater from Compost and Conventional Erosion Control Practices in Construction Activities*. Journal of Soil and Water Conservation, 60(6): 288-297.

18. Fennessey, L.A. and Jarrett, A.R. (1997). *Influence of Principal Spillway Geometry and Permanent Pool Depth of Sediment retention of Sedimentation Basins.* Transactions of the American Society of Agricultural Engineers. 40 (1) 53-59.

19. Gharabagi, B., R. Rudra, E. Mcbean, K. Finney, A. Kristoferson, L. Carlson, S. Murray, M. Rudra, C. Desrochers, R. Breivik, D. Pepall, R. Bach, T. Costelo, V. Taleban, K. Chapi, and C. Inkratas. 2007. A Laboratory and Field Scale Evaluation of Compost Biofilters for Stormwater Management, School of Engineering, University of Guelph, Guelph, Ontario, N1G 2W1, Canada.

20. Google Inc. (2013). Google Earth [Software].

21. Jiang, N., Hirschi, M.C., Cooke, R.A., and Mitchell, J.K. (1997). *Equation for Flow through Filter Fabric*. Transactions of the American Society of Agricultural Engineers. 40 (4) 987-991.

22. Kang, J., McCaleb, M.M., and McLaughlin, R.A. *Check Dam and Polyacrylamide Performance under Simulated Stormwater Runoff.* Journal of Environmental Management 129 (2013) 593-598.

23. King, S.E., R. A. McLaughlin. *Fiber Check Dams and Polyacrylamide for Water Quality Improvement* North Carolina State University, 2009.

24. Levy, G. J., J. Levin and I. Shainberg (1994). *Seal Formation and Interrill Soil Erosion*. Soil Science Society of America Journal, 58(1): 203-209.

25. Line, D.E. and White, N.M. (2001). *Efficiencies of Temporary Sediment Traps on Two North Carolina construction Sites*. Transactions of the American Society of Agricultural Engineers. 44 (5) 1207-1215.

26. Mamo, M., Bubenzer, G.D., (2004). *Decomposition Parameters of Straw Erosion Control Blankets*. Transactions of the American Society of Agricultural Engineers. 47 (3) 721-725.

27. *Manual for Erosion and Sediment Control in Georgia*, Georgia Water and Soil Conservation Commission (GWSCC), Athens, GA, 2000.

28. McCaleb, M.M., and McLaughlin, R.A. (2008). *Sediment Trapping by Five Different Sediment Detention Devices on Construction Sites*. Transactions of the American Society of Agricultural and Biological Engineers. 51 (5) 1613-1621.

29. McEnroe, B. M., and B. J. Treff. *Performance of KDOT Temporary Erosion Control Measures*, Report No. K-TRAN: KU-97-2 Final Report, Kansas Department of Transportation, December 1997.

30. McFalls, Jett, et al. (2010) *Proposed Test Protocol for Evaluating the Performance of Sediment Control Devices for Roadside Stormwater Runoff*. Transportation Research Record: Journal of the Transportation Research Board, Issue 2170, 2010, pp 74-83.

31. McLaughlin, R. A., N. Rajbhandari, W. F. Hunt, D. E. Line, R. E. Sheffield, and N. M. White (2001). Soil Erosion Research for the 21st Century, *The Sediment and Erosion Control Research and Education Facility at North Carolina State University*, Proceedings of the International Conference of American Society of Agricultural Engineers (ASAE) Pub # 701P0007.

32. McLaughlin, R.A., Hayes, S.A., Clinton, D.L., McCaleb, M.S., and Jennings, G.D (2009a). *Water Quality Improvements Using Modified Sediment Control Systems on Construction Sites*. Transactions of the American Society of Agricultural and Biological Engineers. 52 (6) 1857-1867.

33. McLaughlin, R. A., S. E. King, and G. D (2009b). Jennings, Improving Construction Site Runoff Quality with Fiber Check Dams and Polyacrylamide, *Journal of Soil and Water Conservation*, Vol. 64, pp.144–154.

34. Mishra, S. and Singh, V. (1999). *Another Look at SCS-CN Method*. Journal of Hydrologic Engineering. 4(3), 257–264.

35. *North Dakota Erosion and Sediment Control Handbook*, North Dakota Department of Transportation, 2004.

36. Norton, Earl. Photos Taken at Texas Research International Denver Downs Research Facility in 2008.

37. Novotny, V. 2003. Water Quality: Diffuse Pollution and Watershed Management. New York: John Wiley & Sons, Inc.

38. Ooshaksaraie, L., Basri, N., Bakar, A.A., and Maulud, K (2009). *Erosion and Sediment Control Plans to Minimize Impacts of Housing Construction Activities on Water Resources in Malaysia.* European Journal of Scientific Research. 33 (3) 461-470.

39. Ponce, V. M., and Hawkins, R. H. (1996). *Runoff curve number: Has it reached maturity?* Journal of Hydrologic Engineering. ASCE, 1(1), 11–19.

40. Prosser, Ian P. and Paul Rustomji. (2000). *Sediment Transport Capacity Relations of Overland Flow*. Progress in Physical Geography: 179-193.

41. Smolen, M. D., D. W. Miller, L. C. Wyall, J. Linchthardt, and A. L. Lanier. 1988. *Erosion and Sediment Control Planning and Design Manual.* North Carolina Sedimentation Control Commission and North Carolina Department of Natural Resources and Community Development, Raleigh, NC.

42. *Temporary Rock Silt Check Type B Specification, Erosion Control and Roadside Development*, North Carolina Department of Transportation (NCDOT), Raleigh, NC, 2012, 1633.01.

43. *Tennessee Erosion and Sediment Control Handbook,* Tennessee Department of Environment and Conservation Activities (TDECA), 2012, 4th Edition.

44. Tennessee Department of Transportation. *Enhanced Silt Fence Check (Trapezoidal Ditch)* August 1, 2012.

45. Texas Research International: TRI Environmental, Inc. (2008). *Erosion and Sediment Control Testing of Erosion Control Products at TRI/Environmental, Inc.* 1-6.

32. U.S. Department of Agriculture (USDA), *Urban Hydrology for Small Watersheds: Technical Release 55*, Washington, D.C., June 1986.

46. U.S. Environmental Protection Agency (EPA), *Development Document for Final Effluent Guidelines and Standards for the Construction and Development Category,* Washington, D.C., November 2009.

47. U.S. Environmental Protection agency (EPA), *Erosion, Sediment and Runoff Control for Roads and Highways*, Office of Water, EPA-841-F-95-008d. December 1995.

48. U.S. Environmental Protection Agency (EPA), *Menu of BMP's: Check Dams,* May 2006.

49. U.S. Environmental Protection Agency (EPA), *Proposed Revisions to the Construction and Development Effluent Guidelines*, Office of Water. February 2013.

50. U. S. Environmental Protection Agency Office of Water. *Stormwater Phase 2 Final Rule Fact Sheet 1.0. Guideline Fact Sheet*. Washington, D. C., 2005.Faucette et al 2005.

51. U.S. Environmental Protection Agency. *The Use of Best Management Practices (BMPs) in Urban Watersheds*, September 2004.

52. *Wattle Detail, Roadside and Environmental Unit*, North Carolina Department of Transportation (NCDOT), Raleigh, NC, 2012.

53. Yoder, D.C., Tyner, J.S., Balousek, J.D., Panuska, J.C., Buchanan, J.R., Kirsch, K.J., and Lyon, J.P. (2007). *Conservation Planning for Construction Sites*. Transactions of the American Society of Agricultural and Biological Engineers. 50 (5) 1613-1618.

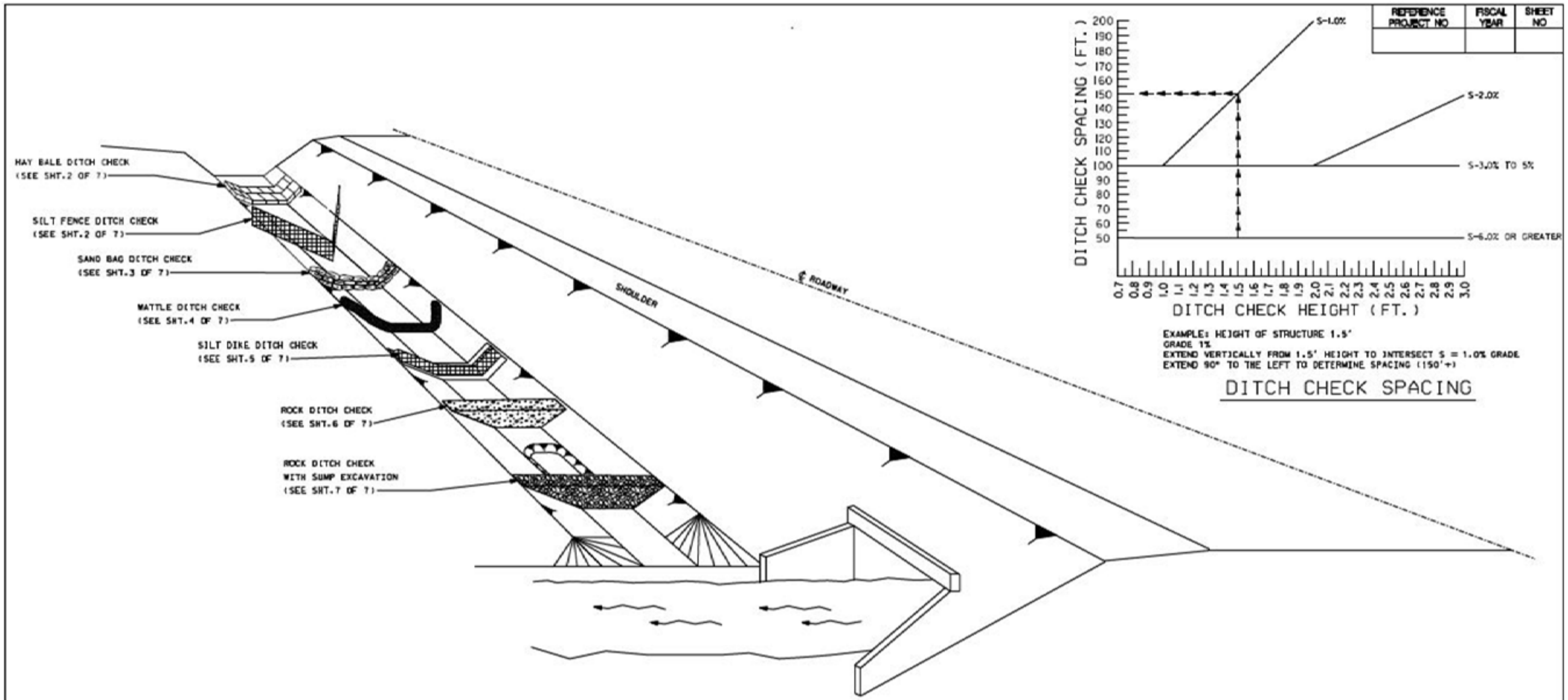
54. Zhang, G.H., Liu, B.Y., Nearing, M.A., Huang, C.H., and Zhang, K.L. (2002). *Soil Detachment by Shallow Flow.* Transactions of the American Society of Agricultural Engineers. 45 (2) 351-357.

55. Zech, W.C., Halverson, J.L., and Clement, T.P. (2008). *Designs to Control Sediment Discharge from Highway Construction Sites.* Journal of Hydrologic Engineering. 13 (6) 497-504.

56. Zech, W.C., McDonald, J.S., and Clement, T.P. (2009). *Field Evaluation of Silt Fence Tieback Systems at a Highway Construction Site.* Practice Periodical on Structural Design and Construction. 14 (3) 105-112.

APPENDIX A:

ALDOT ESC – 300: DITCH CHECK STRUCTURES, TYPICAL APPLICATIONS AND DETAILS



EXAMPLE: HEIGHT OF STRUCTURE 1.5'
GRADE 1%
EXTEND VERTICALLY FROM 1.5' HEIGHT TO INTERSECT S = 1.0% GRADE
EXTEND 90° TO THE LEFT TO DETERMINE SPACING (150'+)

DITCH CHECK SPACING

NOTES:

1. THE DITCH CHECK PERSPECTIVE ILLUSTRATES A TOOL BOX OF TEMPORARY PRACTICES THAT MAY BE DIRECTED OR PERMITTED BY THE ENGINEER. DITCH CHECKS ARE INSTALLED TO CONTROL RUNOFF VELOCITY AND THUS REDUCE EROSION AND PROVIDE FOR TRAPPING OF SEDIMENTS.
2. SELECTION OF THE APPROPRIATE DITCH CHECK SHOULD BE A FUNCTION OF CONSTRUCTION PHASE, DRAINAGE AREA, DITCH GRADIENT, SOIL TYPE, ECONOMY AND SAFETY.
3. DITCH CHECKS CAN BE REMOVED FOR MAINTENANCE AND/OR REPLACEMENT BUT MUST REMAIN IN PLACE UNTIL UPSLOPE AREAS HAVE BEEN PERMANENTLY STABILIZED. MAINTENANCE INCLUDES REMOVAL OF SEDIMENT BEGINNING WHEN SEDIMENT ACCUMULATION REACHES 1/3 THE CAPACITY OR HEIGHT OF THE STRUCTURE AND NEVER ALLOWING FOR SEDIMENT TO ACCUMULATE MORE THAN 1/2 THE VOLUME OR HEIGHT OF THE DITCH CHECK STRUCTURE.
4. HAY BALES ARE USED TO INTERCEPT LOW VOLUME FLOWS IN LOW TO MODERATE GRADIENT DITCHES.
5. SAND BAG DITCH CHECKS ARE USED FOR VELOCITY REDUCTION AND MINIMAL SEDIMENT TRAPPING IN CONCRETE PAVED DITCHES OR IN DITCHES THAT HAVE ROCKY BOTTOMS.
6. WATTLE DITCH CHECKS ARE APPROPRIATE FOR VELOCITY REDUCTION AND CONTROL OF SEDIMENT TRANSPORT UNDER LOW TO MEDIUM FLOW CONDITIONS NOT EXCEEDING 1.0 CU FT/SEC.
7. SILT DIKES CAN BE USED IN DITCHES WITH CONCENTRATED FLOWS WITHIN THE CLEAR ZONE WHERE RIPRAP CAN NOT BE USED, AS CONSTRUCTION PROGRESSES.
8. THE TYPE AND SIZE OF ROCK USED TO CONSTRUCT ROCK DITCH CHECKS WILL BE SELECTED BY THE DESIGNER AND SHOWN ON THE PLANS. THE SIZE OF ROCK CHOSEN SHOULD BE PROPORTIONAL TO EXPECTED FLOWS AND VELOCITIES. SEDIMENT TRAPPING EFFECTIVENESS MAY BE ADJUSTED BY CHOKING.
9. ROCK DITCH CHECK WITH SUMP EXCAVATION CAN BE PLACED IN DITCHES TO FACILITATE ON-SITE SEDIMENT TRAPPING. DITCH CHECK WITH SUMP EXCAVATION IS USED WHEN DITCHES IS USED RECEIVE DRAINAGE FROM CUT OR FILL SLOPES OR OTHER CRITICAL AREAS WHERE SOIL EROSION IS EXPECTED. DRAINAGE AREA FOR A TEMPORARY SEDIMENT TRAP SHALL NOT EXCEED 3 ACRES. THEY CAN BE USED IN SERIES TO INCREASE ON-SITE SEDIMENT TRAPPING EFFICIENCY.
10. DITCH CHECKS SHALL NOT BE PLACED IN LIVE STREAMS.
11. CONFIGURATION AND SPACING MAY BE ADJUSTED IF APPROVED BY THE ENGINEER TO ACCOMODATE TRAVELWAY SAFETY, WATER FLOW, OR SOIL AND INSTALLATION CHALLENGES.
12. SILT FENCE DITCH CHECKS ARE USED WHERE IT HAS BEEN DETERMINED THAT HAY BALE CHECKS ARE INADEQUATE, OR SILT FENCE DITCH CHECKS CAN BE JUSTIFIED BASED ON COST. SILT FENCE DITCH CHECKS ARE USED TO INTERCEPT LOW VOLUME FLOWS IN LOW TO MODERATE GRADIENT DITCHES.

REFERENCE PROJECT NO	FISCAL YEAR	SHEET NO

—SPECIFICATIONS—
CURRENT ALABAMA DEPARTMENT OF TRANSPORTATION

ALABAMA DEPARTMENT OF TRANSPORTATION

DESIGN BUREAU SPECIAL DRAWING

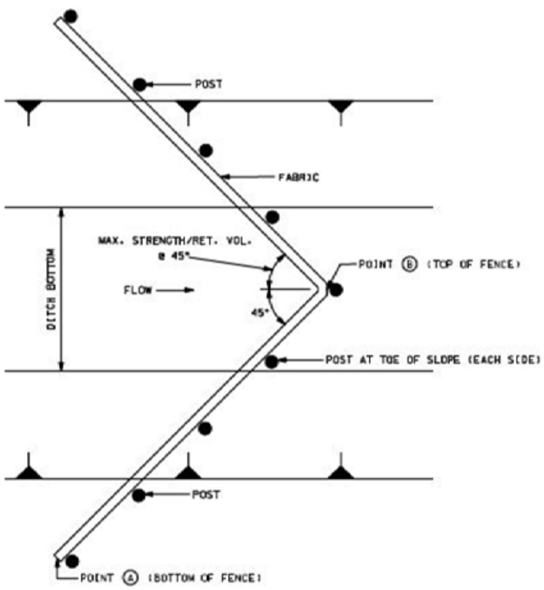
DITCH CHECK STRUCTURES,
TYPICAL APPLICATIONS AND DETAILS

DATE 2006

ESC-300 (SHEET 1 OF 7) 1162

NOT TO SCALE

REFERENCE PROJECT NO	FISCAL YEAR	SHEET NO



PLAN VIEW

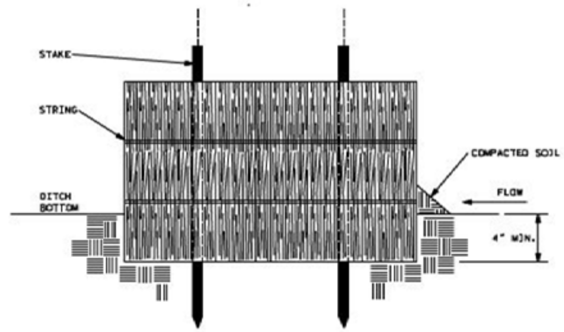
BOTTOM OF FENCE EL. AT Ⓐ = TOP OF FENCE EL. AT Ⓑ

NOTES:

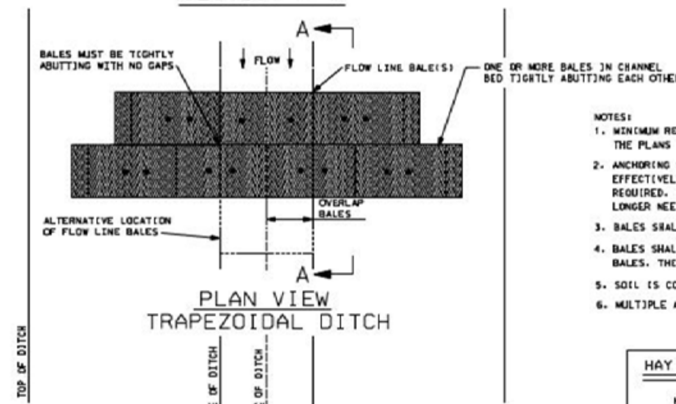
1. ANCHOR AND INSTALL SILT FENCE DITCH CHECK PER DETAILS SHOWN ON SPECIAL DRAWING ESC-200. THE LOCATIONS AND SPACING OF SILT FENCE DITCH CHECKS ALONG A DITCH SHOULD BE 100 FEET UNLESS OTHERWISE SHOWN ON PLANS OR APPROVED BY THE ENGINEER.
2. A "W" SHAPE MAY BE USED FOR WIDER DITCHES.

SILT FENCE DITCH CHECK SELECTION GUIDELINES

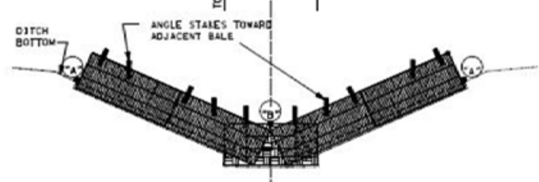
SILT FENCE DITCH CHECKS ARE USED WHERE IT HAS BEEN DETERMINED THAT HAY BALE CHECKS ARE INADEQUATE, OR SILT FENCE DITCH CHECKS CAN BE JUSTIFIED BASED ON COST. SILT FENCE DITCH CHECKS ARE USED TO INTERCEPT LOW VOLUME FLOWS IN LOW TO MODERATE GRADIENT DITCHES.



SECTION A-A



PLAN VIEW
TRAPEZOIDAL DITCH



PROFILE VIEW
TRAPEZOIDAL DITCH

END POINTS "A" MUST BE HIGHER THAN FLOW LINE POINT "B"

NOTES:

1. MINIMUM RECOMMENDED CHECK SPACING IS 100 FEET UNLESS SHOWN OTHERWISE ON THE PLANS OR APPROVED BY THE ENGINEER. SEE SPACING GUIDANCE ON SHT.1 OF 7.
2. ANCHORING STAKES SHALL BE SIZED, SPACED, AND BE OF A MATERIAL THAT EFFECTIVELY SECURES THE CHECK. A MINIMUM OF TWO STAKES PER BALE IS REQUIRED. ALL NON-DEGRADABLE MATERIALS SHALL BE REMOVED WHEN NO LONGER NEEDED.
3. BALES SHALL BE EMBEDDED IN THE SOIL A MIN. OF 4".
4. BALES SHALL BE PLACED IN A ROW WITH ENDS TIGHTLY ABUTTING THE ADJACENT BALES. THE BALES SHALL BE PLACED WITH BINDINGS PARALLEL TO THE GROUND.
5. SOIL IS COMPACTED ALONG THE BASE OF THE UPSTREAM FACE TO PREVENT PIPING.
6. MULTIPLE ADJACENT ROWS OF BALES ARE REQUIRED AS SHOWN.

HAY BALE DITCH CHECK SELECTION GUIDELINES

HAY BALES ARE USED TO INTERCEPT LOW VOLUME FLOWS IN LOW TO MODERATE GRADIENT DITCHES.

—SPECIFICATIONS—
CURRENT ALABAMA DEPARTMENT OF TRANSPORTATION

ALABAMA DEPARTMENT OF TRANSPORTATION
1409 COLISEUM BOULEVARD
MONTGOMERY, AL 36130-3050

DESIGN BUREAU SPECIAL DRAWING

DETAILS OF EROSION CONTROL
HAY BALE AND SILT FENCE
DITCH CHECKS

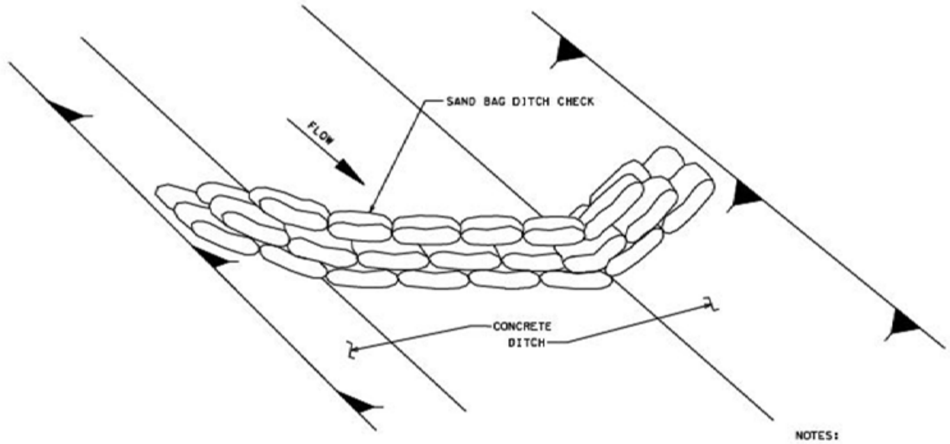
DATE DRAWN 2006

ESC-300 (SHEET 2 OF 7)

1162-A

NOT TO SCALE

REFERENCE PROJECT NO	FISCAL YEAR	SHEET NO



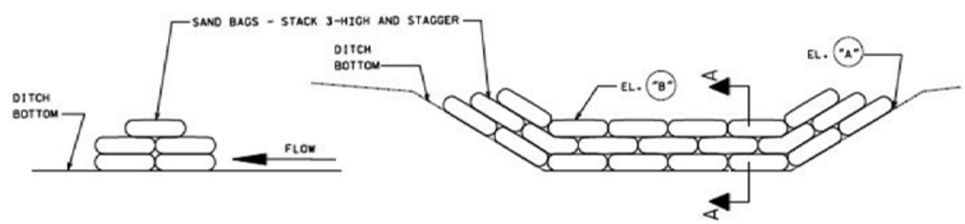
DETAIL (DITCH CHECK)

NOTES:
1. MINIMUM RECOMMENDED PLACEMENT INTERVAL BETWEEN SAND BAG DITCH CHECK IS 100' UNLESS SHOWN OTHERWISE ON THE PLANS OR APPROVED BY THE ENGINEER. SEE SPACING GUIDANCE ON SHT.1 OF 7.

2. PREVENTING SEDIMENT FROM ENTERING A PAVED DITCH IS PREFERABLE TO CAPTURING SEDIMENT WITHIN PAVED DITCH.

SAND BAG DITCH CHECK SELECTION GUIDELINES

SAND BAG DITCH CHECKS ARE USED FOR VELOCITY REDUCTION AND MINIMAL SEDIMENT TRAPPING IN CONCRETE PAVED DITCHES OR IN DITCHES THAT HAVE ROCKY BOTTOMS.



SECTION A-A

ELEVATION DETAIL

NOTE: END POINTS "A" MUST BE HIGHER THAN FLOWLINE POINT "B"

—SPECIFICATIONS—
CURRENT ALABAMA DEPARTMENT OF TRANSPORTATION

ALABAMA DEPARTMENT OF TRANSPORTATION

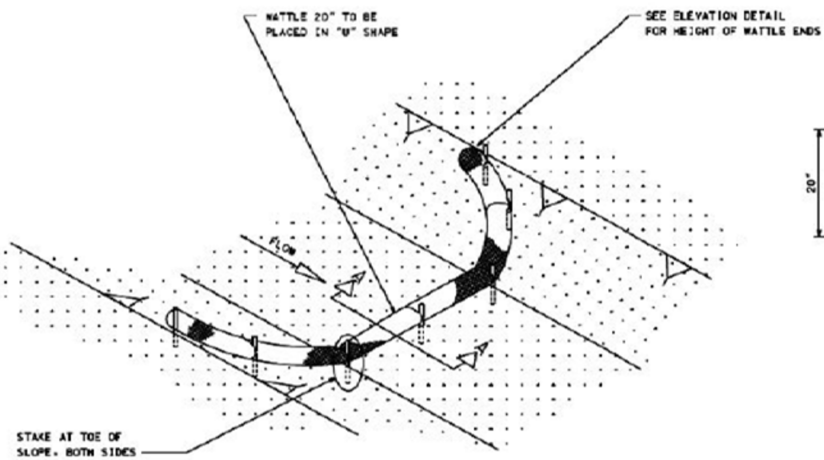
DESIGN BUREAU SPECIAL DRAWING

DETAILS OF EROSION CONTROL
SANDBAG DITCH CHECK

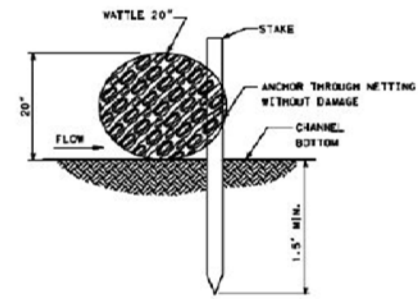
ESC-300 (SHEET 3 OF 7) | 1162-B

NOT TO SCALE

REFERENCE PROJECT NO	FISCAL YEAR	SHEET NO



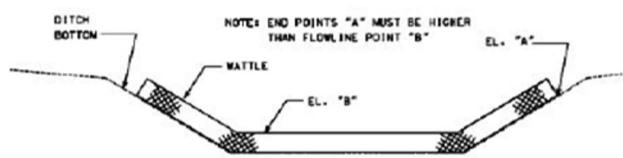
DETAIL (DITCH CHECK)



SECTION A-A

NOTES:

1. MINIMUM RECOMMENDED PLACEMENT INTERVAL BETWEEN WATTLE DITCH CHECK IS 100' UNLESS SHOWN OTHERWISE ON THE PLANS OR APPROVED BY THE ENGINEER. SEE SPACING GUIDANCE ON SHT.1 OF 7.
2. ANCHORING STAKES SHALL BE SIZED, SPACED, DRIVEN, AND BE OF A MATERIAL THAT EFFECTIVELY SECURES THE CHECK. STAKE SPACING SHALL BE A MAXIMUM OF THREE FEET. ALL NON-DEGRADABLE MATERIALS SHALL BE REMOVED WHEN NO LONGER NEEDED.
3. TRENCHING OF WATTLES MAY BE NECESSARY IF PIPING BECOMES EVIDENT.
4. WATTLES SHOULD NOT BE USED IN HARD BOTTOM CHANNELS.



ELEVATION DETAIL

WATTLE DITCH CHECK SELECTION GUIDELINES

WATTLE DITCH CHECKS ARE APPROPRIATE FOR VELOCITY REDUCTION AND CONTROL OF SEDIMENT TRANSPORT UNDER LOW TO MEDIUM FLOW CONDITIONS.

--SPECIFICATIONS--
CURRENT ALABAMA DEPARTMENT OF TRANSPORTATION

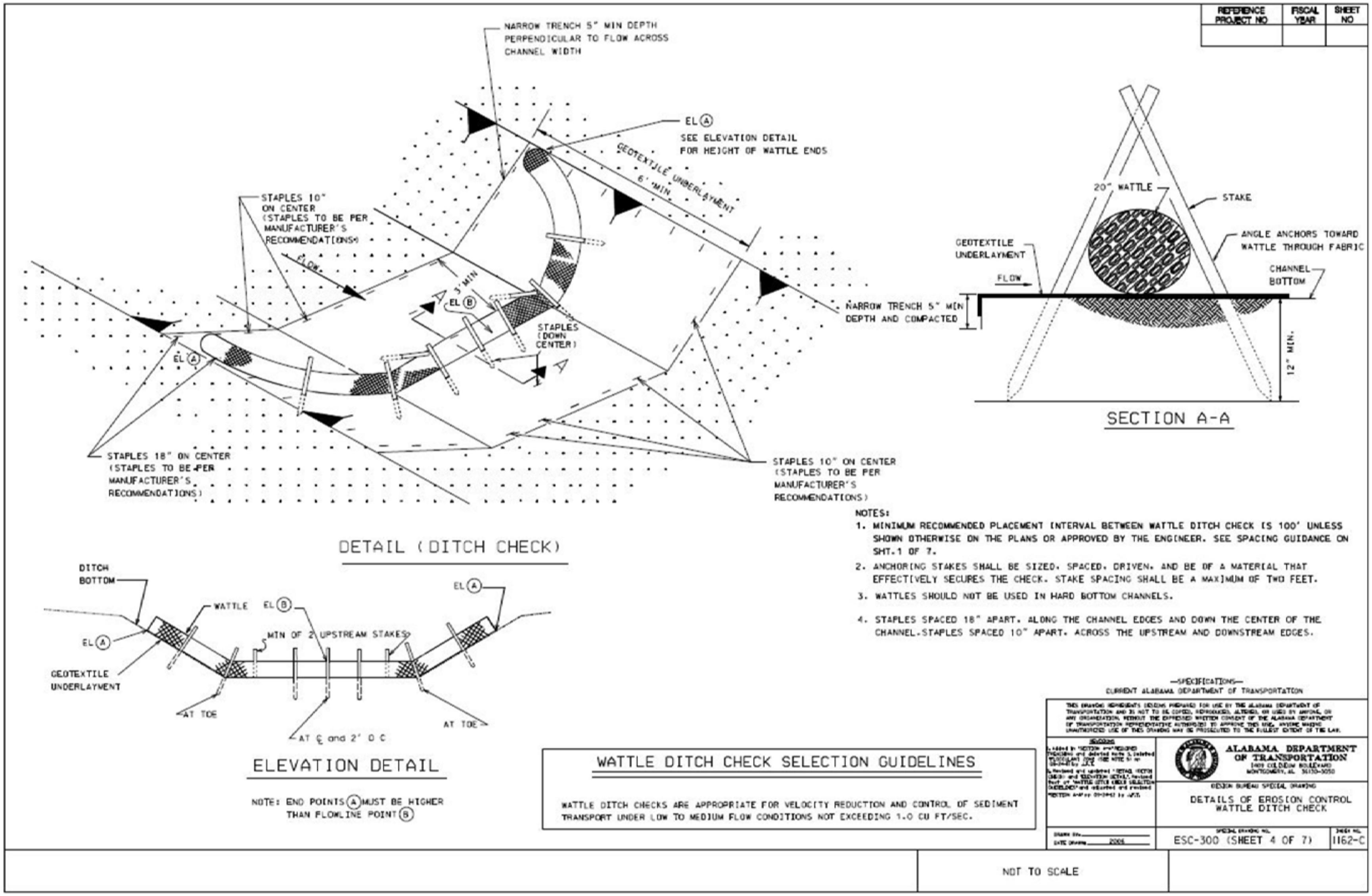
ALABAMA DEPARTMENT OF TRANSPORTATION

DETAILS OF EROSION CONTROL
WATTLE DITCH CHECK

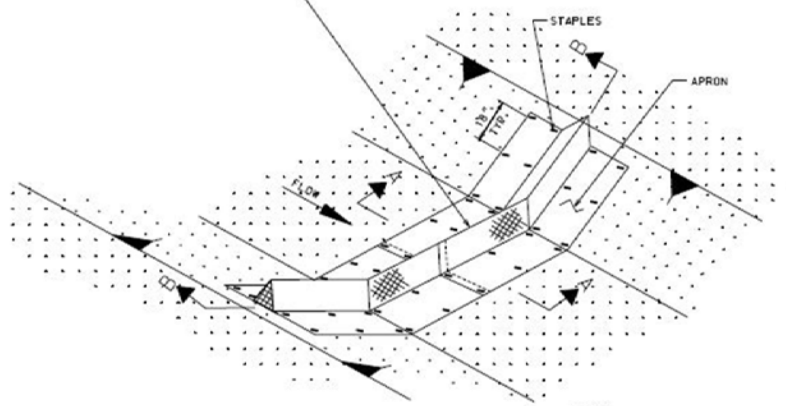
2006

ESC-300 (SHEET 4 OF 7) | II62-C

NOT TO SCALE


REFERENCE PROJECT NO
FISCAL YEAR
SHEET NO
NARROW TRENCH 5" MIN DEPTH PERPENDICULAR TO FLOW ACROSS CHANNEL WIDTH
EL (A)
SEE ELEVATION DETAIL FOR HEIGHT OF WATTLE ENDS
GEOTEXTILE UNDERLAYMENT
6' MIN
STAPLES 10" ON CENTER (STAPLES TO BE PER MANUFACTURER'S RECOMMENDATIONS)
FLOW
3' MIN
EL (B)
STAPLES (DOWN CENTER)
A
EL (A)
NARROW TRENCH 5" MIN DEPTH AND COMPACTED
STAPLES 18" ON CENTER (STAPLES TO BE PER MANUFACTURER'S RECOMMENDATIONS)
STAPLES 10" ON CENTER (STAPLES TO BE PER MANUFACTURER'S RECOMMENDATIONS)
DETAIL (DITCH CHECK)
20" WATTLE
STAKE
ANGLE ANCHORS TOWARD WATTLE THROUGH FABRIC
GEOTEXTILE UNDERLAYMENT
FLOW
CHANNEL BOTTOM
12" MIN.
SECTION A-A
NOTES:
1. MINIMUM RECOMMENDED PLACEMENT INTERVAL BETWEEN WATTLE DITCH CHECK IS 100' UNLESS SHOWN OTHERWISE ON THE PLANS OR APPROVED BY THE ENGINEER. SEE SPACING GUIDANCE ON SHT.1 OF 7.
2. ANCHORING STAKES SHALL BE SIZED, SPACED, DRIVEN, AND BE OF A MATERIAL THAT EFFECTIVELY SECURES THE CHECK. STAKE SPACING SHALL BE A MAXIMUM OF TWO FEET.
3. WATTLES SHOULD NOT BE USED IN HARD BOTTOM CHANNELS.
4. STAPLES SPACED 18" APART. ALONG THE CHANNEL EDGES AND DOWN THE CENTER OF THE CHANNEL. STAPLES SPACED 10" APART. ACROSS THE UPSTREAM AND DOWNSTREAM EDGES.
DITCH BOTTOM
WATTLE
EL (B)
EL (A)
MIN OF 2 UPSTREAM STAKES
EL (A)
GEOTEXTILE UNDERLAYMENT
AT TOE
AT ℄ and 2' O C
AT TOE
ELEVATION DETAIL
NOTE: END POINTS (A) MUST BE HIGHER THAN FLOWLINE POINT (B)
WATTLE DITCH CHECK SELECTION GUIDELINES
WATTLE DITCH CHECKS ARE APPROPRIATE FOR VELOCITY REDUCTION AND CONTROL OF SEDIMENT TRANSPORT UNDER LOW TO MEDIUM FLOW CONDITIONS NOT EXCEEDING 1.0 CU FT/SEC.
—SPECIFICATIONS—
CURRENT ALABAMA DEPARTMENT OF TRANSPORTATION
ALABAMA DEPARTMENT OF TRANSPORTATION
DETAILS OF EROSION CONTROL WATTLE DITCH CHECK
2006
ESC-300 (SHEET 4 OF 7)
1162-C
NOT TO SCALE

REFERENCE PROJECT NO	FISCAL YEAR	SHEET NO



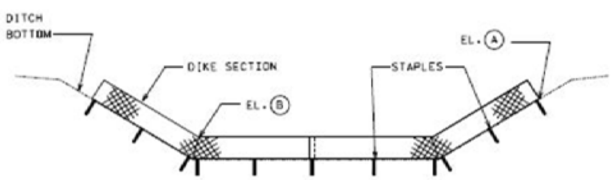
PLAN VIEW

SILT DIKE DITCH CHECK SELECTION GUIDELINES

SILT DIKES CAN BE USED IN DITCHES WITH CONCENTRATED FLOWS WITHIN THE CLEAR ZONE WHERE RIPRAP CAN NOT BE USED.

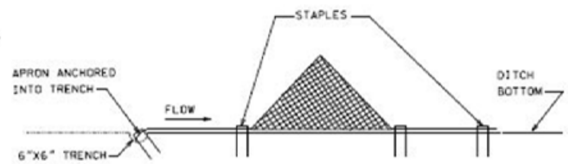
NOTE:

1. MINIMUM RECOMMENDED PLACEMENT INTERVAL BETWEEN SILT DIKE DITCH CHECK IS 100' UNLESS SHOWN OTHERWISE ON THE PLANS OR APPROVED BY THE ENGINEER. SEE SPACING GUIDANCE ON SHT.1 OF 7.
2. INSTALLATION SHALL BE IN ACCORDANCE WITH MANUFACTURERS RECOMMENDATIONS.



SECTION B-B

POINT Ⓐ MUST BE HIGHER THAN POINT Ⓑ TO ENSURE THAT WATER FLOWS OVER THE DIKE AND NOT AROUND THE ENDS



NOTE: STAPLES SHALL BE PLACED WHERE THE UNITS OVERLAP AND IN THE CENTER OF THE UNIT

SECTION A-A

SILT DIKE INSTALLATION FOR ROADWAY DITCHES

—SPECIFICATIONS—
CURRENT ALABAMA DEPARTMENT OF TRANSPORTATION

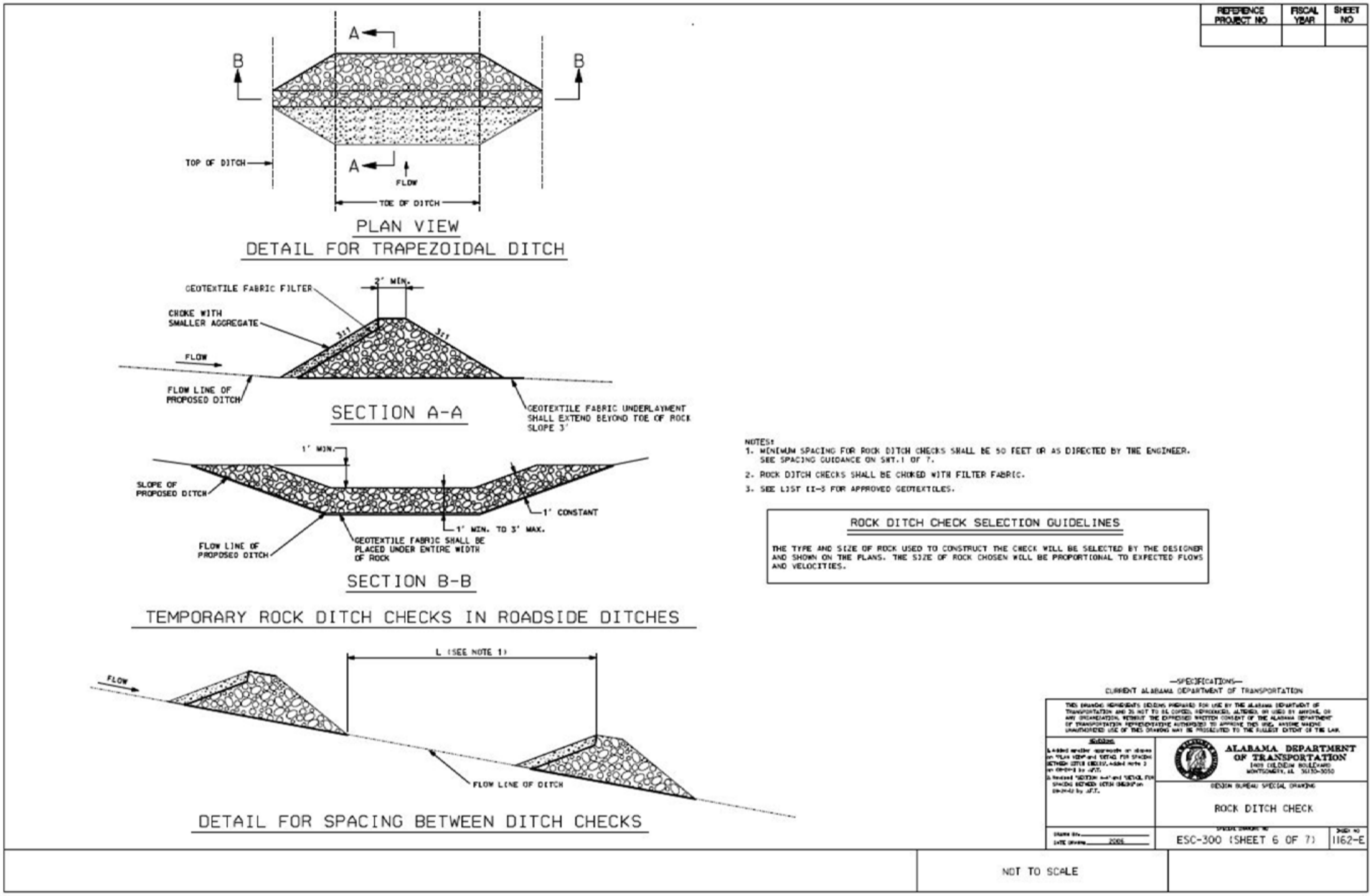
ALABAMA DEPARTMENT OF TRANSPORTATION

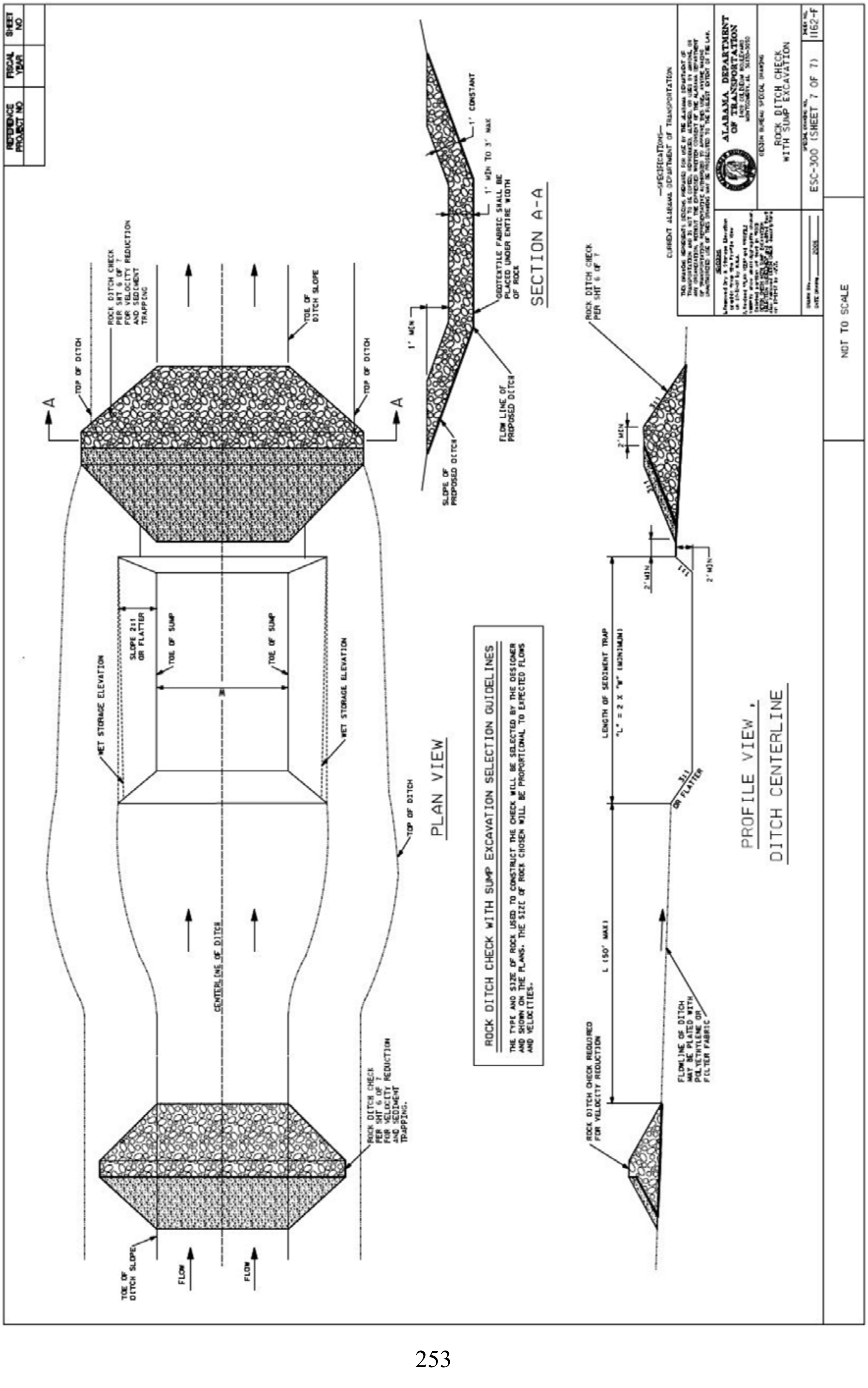
DESIGN BUREAU SPECIAL DRAWING

DETAILS OF EROSION CONTROL
SILT DIKE DITCH CHECK

ESC-300 (SHEET 5 OF 7) | 1162-D

NOT TO SCALE


REFERENCE PROJECT NO
FISCAL YEAR
SHEET NO
A
B
B
A
TOP OF DITCH
FLOW
TOE OF DITCH
PLAN VIEW
DETAIL FOR TRAPEZOIDAL DITCH
GEOTEXTILE FABRIC FILTER
2' MIN.
CHOKE WITH SMALLER AGGREGATE
3:1
3:1
FLOW
FLOW LINE OF PROPOSED DITCH
SECTION A-A
GEOTEXTILE FABRIC UNDERLAYMENT SHALL EXTEND BEYOND TOE OF ROCK SLOPE 3'
1' MIN.
SLOPE OF PROPOSED DITCH
1' CONSTANT
1' MIN. TO 3' MAX.
FLOW LINE OF PROPOSED DITCH
GEOTEXTILE FABRIC SHALL BE PLACED UNDER ENTIRE WIDTH OF ROCK
SECTION B-B
TEMPORARY ROCK DITCH CHECKS IN ROADSIDE DITCHES
L (SEE NOTE 1)
FLOW
FLOW LINE OF DITCH
DETAIL FOR SPACING BETWEEN DITCH CHECKS
NOTES:
1. MINIMUM SPACING FOR ROCK DITCH CHECKS SHALL BE 50 FEET OR AS DIRECTED BY THE ENGINEER. SEE SPACING GUIDANCE ON SHT. 1 OF 7.
2. ROCK DITCH CHECKS SHALL BE CHOKED WITH FILTER FABRIC.
3. SEE LIST II-3 FOR APPROVED GEOTEXTILES.
ROCK DITCH CHECK SELECTION GUIDELINES
THE TYPE AND SIZE OF ROCK USED TO CONSTRUCT THE CHECK WILL BE SELECTED BY THE DESIGNER AND SHOWN ON THE PLANS. THE SIZE OF ROCK CHOSEN WILL BE PROPORTIONAL TO EXPECTED FLOWS AND VELOCITIES.
—SPECIFICATIONS—
CURRENT ALABAMA DEPARTMENT OF TRANSPORTATION
ALABAMA DEPARTMENT OF TRANSPORTATION
DESIGN BUREAU SPECIAL DRAWING
ROCK DITCH CHECK
ESC-300 (SHEET 6 OF 7)
1162-E
2006
NOT TO SCALE



REFERENCE PROJECT NO
FISCAL YEAR
SHEET NO
PLAN VIEW
TOP OF DITCH
ROCK DITCH CHECK PER SHT 6 OF 7 FOR VELOCITY REDUCTION AND SEDIMENT TRAPPING
TOE OF DITCH SLOPE
WET STORAGE ELEVATION
SLOPE 2:1 OR FLATTER
TOE OF SUMP
CENTERLINE OF DITCH
FLOW
ROCK DITCH CHECK PER SHT 6 OF 7 FOR VELOCITY REDUCTION AND SEDIMENT TRAPPING.
ROCK DITCH CHECK WITH SUMP EXCAVATION SELECTION GUIDELINES
THE TYPE AND SIZE OF ROCK USED TO CONSTRUCT THE CHECK WILL BE SELECTED BY THE DESIGNER AND SHOWN ON THE PLANS. THE SIZE OF ROCK CHOSEN WILL BE PROPORTIONAL TO EXPECTED FLOWS AND VELOCITIES.
SECTION A-A
1' MIN
1' CONSTANT
1' MIN TO 3' MAX
GEOTEXTILE FABRIC SHALL BE PLACED UNDER ENTIRE WIDTH OF ROCK
SLOPE OF PROPOSED DITCH
FLOW LINE OF PROPOSED DITCH
PROFILE VIEW
DITCH CENTERLINE
ROCK DITCH CHECK PER SHT 6 OF 7
ROCK DITCH CHECK REQUIRED FOR VELOCITY REDUCTION
L (50' MAX)
LENGTH OF SEDIMENT TRAP
"L" = 2 X "W" (MINIMUM)
2:1 OR FLATTER
2' MIN
FLOWLINE OF DITCH MAY BE PLATED WITH POLYETHYLENE OR FILTER FABRIC
ALABAMA DEPARTMENT OF TRANSPORTATION
ROCK DITCH CHECK WITH SUMP EXCAVATION
ESC-300 (SHEET 7 OF 7)
1162-F
NOT TO SCALE

APPENDIX B:

WATTLE PRODUCT SPECIFICATION SHEETS

AMERICAN EXCELSIOR: CURLEX SEDIMENT LOGS

American Excelsior Company®
Earth Science Division

Proud Participant in NTPEP and Proud Member of:

Curlex® Sediment Logs®
SPECIFICATION

PART I - GENERAL

1.01 Summary

A. The sediment log contains excelsior wood fiber for the purpose of slowing water velocity and trapping sediment as described herein.

B. This work shall consist of furnishing and installing the sediment log; including fine grading, installing, staking, and miscellaneous related work, in accordance with these standard specifications and at the locations identified on drawings or designated by the owner's representative. This work shall include all necessary materials, labor, supervision, and equipment for installation of a complete system.

C. All work of this section shall be performed in accordance with the conditions and requirements of the contract documents.

D. The sediment log shall be used to slow water velocity, trap sediment, and enhance revegetation. Based on a project-by-project engineering analysis, the sediment log shall be suitable for the following applications:

1. Channels, Swales, and Ditches
2. Inlet and outlet protections
3. Slope interruption
4. Perimeter control

1.02 Performance Requirements

A. Sediment log shall provide temporary, biodegradable channel and slope interruption by slowing water velocity to reduce shear stress and soil erosion while enhancing revegetation. Sediment log performance capabilities shall be determined by large-scale testing deemed acceptable by the design engineer.

850 Avenue H East ● Arlington, Texas 76011
Phone 1-800-777-SOIL ● Fax 817-385-3585 ● www.Curlex.com

B. Sediment log performance requirements:

Slope Erosion*:	Reduce by a minimum of 70% of bare soil slopes
Channel Erosion**:	Reduce by a minimum of 50% of bare soil channels
pH Absorption***:	Ending pH shall not exceed 8.3
Functional Longevity****:	≤ 24 months
Oil Sorbent Material:	U.S. E.P.A. documentation for preapproval

*Based on large-scale rainfall testing as outlined in Kelsey, K., T. Johnson, and R. Vavra. 2006. "Needed Information: Testing, Analyses, and Performance Values for Slope Interruption and Perimeter Control BMPs." IECA Conference Proceedings. P. 171-181.

**Based on ASTM D7208

***Based on ASTM D1117, modified

***Functional Longevity varies from region to region because of differences in climatic conditions.

1.03 Submittals

A. Submittals shall include complete design data, Product Netting Information, SDS, Installation Guidelines, Manufacturing Material Specifications, Manufacturing Certifications, Staking Pattern Guide, CAD details, and a Manufacturing Quality Control Program. In addition, the Manufacturer shall provide a test report providing data showing the performance capabilities of the sediment log, along with reference installations similar in size and scope to that specified for the project.

1.04 Delivery, Storage, and Handling

A. Sediment log shall be furnished on pallets or master packs.

B. Curlex Sediment Logs may be compressed when packaged. The unique packaging can result in a less than symmetrical shape upon arrival to the jobsite. This will not affect the performance capability of Curlex Sediment Logs because unique Curlex fibers naturally expand upon wetting and return to a symmetrical tubular shape.

C. Sediment log shall be free of defects and voids that would interfere with proper installation or impair performance.

D. Sediment log shall be stored by the Contractor in a manner that protects them from damage by construction activities.

PART II - PRODUCTS

2.01 Sediment Log

A. Sediment logs shall be Curlex Sediment Logs, as manufactured by American Excelsior Company, Arlington, TX (800-777-7645).

B. Curlex Sediment Log consist of a specific cut of seed free Great Lakes Aspen wood excelsior with 80% of the fiber ≥ 6 inches in length inside a durable, flexible tubular netting with metal clips or knotted ends. Curlex Sediment Log is designed to provide intimate contact with the soil, which prevents blowouts and undermining. Curlex Sediment log allows water to flow through the excelsior, minimizing overtopping, slowing high flow water velocities, and intercepting and stopping silt movement. Curlex Sediment Logs may be installed over bare soil, over rolled erosion control products, on steep slopes, around inlets and outlets, or around jobsites for perimeter control. Curlex Sediment Log shall be Manufactured in the U.S.A. at company locations where QA/QC is implemented and managed by the manufacturer. Field fabricated products and products made by anyone other than the manufacturer (i.e. distributors, dealers, etc.) shall not be accepted.

C. Sediment logs shall have the following nominal material characteristics:

PROPERTY	ENGLISH	METRIC
Product Name	6 in 9 in 12 in 20 in	15.2 cm 22.9 cm 30.5 cm 50.1 cm
Minimum Diameter	5.5 in 8.0 in 11.0 in 18.0 in	14.0 cm 20.3 cm 27.9 cm 45.7 cm
Log Density* (± 10 %)	(6 in) 2.44 lb/ft^3 (9 in) 2.26 lb/ft^3 (12 in) 2.54 lb/ft^3 (20 in) 1.38 lb/ft^3	39.15 kg/m^3 36.26 kg/m^3 40.80 kg/m^3 22.00 kg/m^3
Fiber Length (80% min.)	≥ 6.0 in	≥ 15.2 cm
Log Dimensions (W x L) (± 10 %)	6 in x 25 ft 9 in x 25 ft 12 in x 10.0 ft 20 in x 10.0 ft	0.1520 m x 7.620 m 0.2290 m x 7.620 m 0.3048 m x 3.048 m 0.5080 m x 3.048 m

* Weight is based on a dry fiber weight basis at time of manufacture. Baseline moisture content of Great Lakes Aspen excelsior is 22%.

2.02 Stakes

A. Stakes shall be wooden, 1 1/8" wide x 1 1/8" thick by a minimum of 30" long for 6", 9", and 12" Curlex Sediment Logs and 48" long for 20" Curlex Sediment Logs.

B. 6 inch and 9 inch Curlex Sediment Logs may also be anchored with E-Staples®, 1" x 6", U-shaped, 11 gauge wire staples, 2" x 8", U-shaped, 8 gauge wire staples. Anchoring with staples shall not be used in channelized flow applications. Stakes may be used in conjunction with staples for additional anchoring of 6 inch and 9 inch Curlex Sediment Logs, as deemed necessary by the Engineer.

PART III - EXECUTION

3.01 Sediment Log Supplier Representation

A. Contractor shall coordinate with the log supplier for a qualified representative to be present on the job site at the start of installation to provide technical assistance as needed. Contractor shall remain solely responsible for the quality of installation.

3.02 Site Preparation

A. Before placing sediment logs, the Contractor shall certify that the subgrade has been properly compacted, graded smooth, has no depressions, voids, soft or uncompacted areas, is free from obstructions such as tree roots, protruding stones or other foreign matter, and is seeded and fertilized according to project specifications where applicable. The Contractor shall not proceed until all unsatisfactory conditions have been remedied. By beginning construction, Contractor signifies that the preceding work is in conformance with this specification.

B. Contractor shall fine grade the subgrade by hand dressing where necessary to remove local deviations.

C. No vehicular traffic shall be permitted directly on the sediment log.

3.03 Installation

A. Sediment log shall be installed as directed by the owner's representative in accordance to manufacturer's Installation Guidelines, Staking Pattern Guide, and CAD details. The extent of sediment logs shall be as shown on the project drawings.

B. Sediment log should be installed to intercept water flow and collect sediment on site. They may be placed over bare soil or on top of erosion control blankets. Sediment logs are typically installed laying on flat ground and not trenched.

C. They shall be secured to the subgrade by wood stakes every two lineal feet across the length of the sediment log. The stakes shall be intertwined with the outer mesh of the sediment log only and driven into the ground a minimum of 16 inches on the downstream side of the sediment log.

D. 6 inch and 9 inch Curlex Sediment Logs can also be installed to the subgrade with E-Staples® or wire staples. Staples shall be installed every two lineal feet across the length on each side of the sediment log. The two rows of staples shall be staggered by one foot along the length of the sediment log. All staples shall be fully inserted into the subgrade below the sediment log.

E. Sediment log installed in a swale or channel bottom shall allow the installation to continue up the slopes three feet above the anticipated high water mark and perpendicular to the flow of water.

F. Spacing of sediment logs shall be such that the elevation of the bottom of the sediment log upstream will be equal to the elevation of the top of the log downstream.

G. Sediment log shall remain in place until fully established vegetation and root systems are present.

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3.04 Quality Assurance

A. Sediment log shall not be defective or damaged. Damaged or defective materials shall be replaced at no additional cost to the owner.

B. Product shall be manufactured in accordance to a documented Quality Control Program. At a minimum, the following procedures and documentation shall be provided upon request:
   1. Manufacturing Quality Control Program Manual
   2. Additional inspections for product conformance shall be conducted during the run after the first piece inspection.
   3. Moisture content readings recorded for each manufacturing day.
   4. Each individual sediment log shall be inspected, weighed, and documented prior to packaging for conformance to manufacturing specifications.
   5. Documentation and record retention for at least two years.

3.05 Clean-up

A. At the completion of this scope of work, Contractor shall remove from the job site and properly dispose of all remaining debris, waste materials, excess materials, and equipment required of or created by Contractor. Disposal of waste materials shall be solely the responsibility of Contractor and shall be done in accordance with applicable waste disposal regulations.

3.06 Method of Measurement

A. Sediment log shall be measured for payment as individual items and the unit of measure shall be each.

3.07 Basis of Payment

A. The accepted quantities of sediment log shall be paid for at the contract unit price per each unit, complete in place.

Payment shall be made under:

Pay Item	**Pay Unit**
Sediment Log	Individual Item

Disclaimer: Curlex Sediment Logs is a system for sediment control in channels and on slopes. American Excelsior Company (AEC) believes that the information contained herein to be reliable and accurate for use in sediment control applications. However, since physical conditions vary from job site to job site and even within a given job site, AEC makes no performance guarantees and assumes no obligation or liability for the reliability or accuracy of information contained herein, for the results, safety, or suitability of using Curlex Sediment Logs, or for damages occurring in connection with the installation of any erosion control product whether or not made by AEC or its affiliates, except as separately and specifically made in writing by AEC. These guidelines are subject to change without notice.

MADE IN THE U.S.A.

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AMERICAN EXCELSIOR: AEC PREMIER STRAW WATTLE

AEC Premier Straw® Wattle
SPECIFICATION

PART I - GENERAL

1.01 Summary

A. The straw wattle contains agricultural straw for the purpose of slowing water velocity and trapping sediment as described herein.

B. This work shall consist of furnishing and installing the straw wattle; including fine grading, installing, staking, and miscellaneous related work, in accordance with these standard specifications and at the locations identified on drawings or designated by the owner's representative. This work shall include all necessary materials, labor, supervision, and equipment for installation of a complete system.

C. All work of this section shall be performed in accordance with the conditions and requirements of the contract documents.

D. The straw wattle shall be used to slow water velocity, trap sediment, and enhance revegetation. Based on a project-by-project engineering analysis, the straw wattle shall be suitable for the following applications:

1. Slope interruption
2. Channels, Swales, and Ditches
3. Inlet and outlet protections

1.02 Performance Requirements

A. Straw wattle shall provide temporary, degradable channel and slope interruption by slowing water velocity to reduce shear stress and soil erosion while enhancing revegetation. Straw wattle performance capabilities shall be determined by large-scale testing deemed acceptable by the design engineer.

B. Straw wattle performance requirements:

Functional Longevity*: $\leq$ 18 months

*Functional Longevity varies from region to region because of differences in climatic conditions.

1.03 Submittals

A. Submittals for approval shall include complete design data, Product Netting Information, SDS, Installation Guidelines, Manufacturing Material Specifications, Manufacturing Certifications, Staking Pattern Guide, CAD details, and a Manufacturing Quality Control Program.

1.04 Delivery, Storage, and Handling

A. Straw wattle shall be furnished on pallets or master packs.

B. Straw wattle shall be of consistent density with fibers distributed evenly over the entire area of the wattle.

C. Straw wattle shall be free of defects and voids that would interfere with proper installation or impair performance.

D. Straw wattle shall be stored by the Contractor in a manner that protects them from damage by construction activities.

PART II - PRODUCTS

2.01 Straw Wattle

A. Straw wattle shall be AEC Premier Straw Wattles, as manufactured by American Excelsior Company, Arlington, TX (1-866-9FIBERS).

B. AEC Premier Straw Wattle consists of certified seed free agricultural straw inside a flexible and durable tubular netting with metal clips or knotted ends. AEC Premier Straw Wattle is designed to provide intimate contact with the soil, which prevents blowouts and undermining. AEC Premier Straw Wattle may be placed across channel bottoms, on hillslopes, or around inlet structures. AEC Premier Straw Wattle shall be Manufactured in the U.S.A.

C. Straw wattle shall have the following nominal material characteristics:

PROPERTY	ENGLISH	METRIC
Product Name	9 in 12 in 20 in	22.9 cm 30.5 cm 50.8 cm
Minimum Diameter	8.5 in 11.5 in 19.0 in	21.3 cm 29.2 cm 48.3 cm
Wattle Density (± 10%)	(9 in) 4.53 lb/ft^3 (12 in) 3.82 lb/ft^3 (12 in) 4.24 lb/ft^3 (20 in) 2.75 lb/ft^3	72.62 kg/m^3 61.25 kg/m^3 67.98 kg/m^3 44.10 kg/m^3
Wattle Dimensions (W x L) (± 10%)	9 in x 25 ft 12 in x 10.0 ft 12 in x 15.0 ft 20 in x 10.0 ft	0.2290 m x 7.620 m 0.3048 m x 3.048 m 0.3048 m x 4.572 m 0.508 m x 3.048 m

2.02 Stakes

A. Stakes shall be wooden, 1 1/8" wide x 1 1/8" thick x 30" long. Stakes shall not extend above the straw wattle more than 2".

PART III - EXECUTION

3.01 Straw Wattle Supplier Representation

A. Contractor shall coordinate with the wattle supplier for a qualified representative to be present on the job site at the start of installation to provide technical assistance as needed. Contractor shall remain solely responsible for the quality of installation.

3.02 Site Preparation

A. Before placing straw wattles, the Contractor shall certify that the subgrade has been properly compacted, graded smooth, has no depressions, voids, soft or uncompacted areas, is free from obstructions such as tree roots, protruding stones or other foreign matter, and is seeded and fertilized according to project specifications were applicable. The Contractor shall not proceed until all unsatisfactory conditions have been remedied. By beginning construction, Contractor signifies that the preceding work is in conformance with this specification.

B. Contractor shall fine grade the subgrade by hand dressing where necessary to remove local deviations.

C. No vehicular traffic shall be permitted directly on the straw wattle.

3.03 Installation

A. Straw wattle shall be installed as directed by the owner's representative in accordance to manufacturer's Installation Guidelines, Staking Pattern Guide, and CAD details. The extent of straw wattle shall be as shown on the project drawings.

B. Straw wattle should be installed to intercept water flow and collect sediment on site. They may be placed over bare soil or on top of erosion control blankets. Straw wattles are typically installed in a 2 inch trench with the ends of the wattle facing upstream.

C. They shall be secured to the subgrade by wood stakes every four lineal feet across the length of the straw wattle. The stakes shall be driven through the center of the straw wattle only and driven into the ground a minimum of 24 inches.

D. Straw wattle installed in a swale or channel bottom shall allow the installation to continue up the slopes three feet above the anticipated high water mark and perpendicular to the flow of water.

E. Spacing of straw wattle shall be such that the elevation of the bottom of the straw wattle upstream will be equal to the elevation of the top of the straw wattle downstream.

F. Straw wattle shall remain in place until fully established vegetation and root systems are present.

3.04 Quality Assurance

A. Straw wattle shall not be defective or damaged. Damaged or defective materials shall be replaced at no additional cost to the owner.

B. Product shall be manufactured in accordance to a documented Quality Control Program. At a minimum, the following procedures and documentation shall be provided upon request:
   1. Manufacturing Quality Control Program Manual
   2. Additional inspections for product conformance shall be conducted during the run after the first piece inspection.
   3. Moisture content readings recorded for each manufacturing day.
   4. Each individual straw wattle shall be inspected, weighed, and documented prior to packaging for conformance to manufacturing specifications.

3.05 Clean-up

A. At the completion of this scope of work, Contractor shall remove from the job site and properly dispose of all remaining debris, waste materials, excess materials, and equipment required of or created by Contractor. Disposal of waste materials shall be solely the responsibility of Contractor and shall be done in accordance with applicable waste disposal regulations.

3.06 Method of Measurement

A. Straw wattle shall be measured for payment as individual items and the unit of measure shall be each.

3.07 Basis of Payment

A. The accepted quantities of straw wattle shall be paid for at the contract unit price per each unit, complete in place.

   Payment shall be made under:

Pay Item	Pay Unit
Straw wattle	Individual Item

EAST COAST EROSION: ECWATTLE

443 Bricker Road Bernville, PA 19506
1.800.582.4005
+1.610.488.8496 Fax +1.610.488.8494
www.eastcoasterosion.com

Material and Performance Specification

ECWATTLE Sediment Retention Fiber Rolls

Description: ECWATTLES are flexible, cylindrical Sediment Retention Fiber Rolls (SRFRs) comprised of various types of compressed matrices, designed to reduce hydraulic energy and filter sediment-laden stormwater runoff on slopes and in channels. Each pallet is shrink-wrapped and labeled. SRFRs are designed to be used as perimeter controls, slope interceptor devices, check dams, around temporary soil stockpiles, at curb cuts and drain inlets. SRFRs should be installed in accordance to East Coast Erosion Blankets, LLC's Wattle Installation Guidelines and secured with wooden stakes.

TYPE: 100% Agricultural Straw Netting: UV Degradable Polyethylene

Diameter:	**9.0 in (22.9 cm)**	**12.0 in (30.5 cm)**	**20.0 in (50.8 cm)**
Length:	25 ft (7.62 m)	10.0 ft (3.05 m)	10.0 ft (3.05 m)
Weight ±10%:	50 lbs (22.7 kg)	35 lbs (15.9 kg)	55 lbs (41.9 kg)
Density:	4.52 lbs/ft^3 (72.4 kg/ m^3)	4.46 lbs/ft^3 (71.5 kg/ m^3)	2.53 lbs/ft^3 (40.6 kg/ m^3)
#/Pallet:	14	20	10
Pallets/truck	26	28	28

100% Agricultural Straw ECWATTLES have been tested by an independent laboratory to have an 83% filtering efficiency.

TYPE: 100% Aspen Wood Fibers Netting: UV Degradable Polyethylene

Diameter:	**12.0 in (30.5 cm)**	**20.0 in (50.8 cm)**
Length:	10.0 ft (3.05 m)	10.0 ft (3.05 m)
Weight ±10%:	25 lbs (11.4 kg)	45 lbs (20.4 kg)
Density:	3.18 lbs/ft^3 (51.0 kg/ m^3)	2.08 lbs/ft^3 (33.3 kg/ m^3)
#/Pallet:	20	10
Pallets/truck	28	28

TYPE: Mulch Compost Blend - Filtrexx® Netting: Black Polyester

Diameter:	**8.0 in (20.3 cm)**	**12.0 in (30.5 cm)**
Length:	160.0 ft (48.8 m)	100.0 ft (30.5 m)
Weight ±10%:	1,400 lbs (634.9 kg)	1,850 lbs (839.0 kg)
Density:	25.6 lbs/ft^3 (410.3 kg/ m^3)	23.6 lbs/ft^3 (378.2 kg/ m^3)
#/Pallet:	1	1

TYPE: ECO-LOG Biodegradable Straw Netting: Organic Jute Material

Diameter:	**12.0 in (30.5 cm)**
Length:	12.0 ft (3.65 m)
Weight ±10%:	55 lbs (24.9 kg)
Density:	5.84 lbs/ft^3 (93.6 kg/ m^3)
#/Pallet:	10

Custom lengths on all varieties of above sediment fiber rolls available upon request.

Proud Member of:

The values presented are for guidance purposes and do not constitute the practice of engineering. East Coast Erosion Blankets LLC (ECEB) ascertains that at the time of manufacture, all information presented herein is accurate and reliable and falls within the ECEB manufacturing product specification variances. If the product does not meet the stated values and ECEB is notified in writing prior to installation, the product will be replaced at no cost to the purchaser. ECEB will not be held liable for any type of damage or losses, directly, or indirectly for failure of this product. Current revision supersedes all previous versions for this product.

Revised 01012013

EROSION TECH: WHEAT STRAW SEDIMENT LOGS

ErosionTech

EROSION TECH WHEAT STRAW SEDIMENT LOGS

Description:
Erosion Tech Straw Sediment Logs are made with a wheat straw matrix using a Heavy-Duty Poly Propylene Grid Casing. They are designed to filter sediment out of water before entering storm drain inlets. They also serve as a functional method of slowing down the velocity of water runoff in channels and slopes.

Name	Diameter (inches)	Length (Feet)	Weight (lbs)	Netting Type
WTL3	9	16	20	Heavy Duty- Poly Propylene Grid
WTL12	9	25	28	Heavy Duty- Poly Propylene Grid
WTL8	12	10	25	Heavy Duty- Poly Propylene Grid
WTL11	12	20	50	Heavy Duty- Poly Propylene Grid
WTL9	20	10	60	Heavy Duty- Poly Propylene Grid
GEO-LOG				
WTL2	9	18	51	Woven Monofilament Geotextile
WTL7	9	8	25	Woven Monofilament Geotextile

GeoLog Fabric Specifications		
Description	**Test Method**	**Results**
Grab Tensile	ASTM-D-4632	300/200lbs
Flow Rate	ASTM-D-4491	35 gal/min/ft^2
UV Stability @500hours	ASTM-D-4355	90%@500HR
AOS	ASTM-D-4751	40 sieve

www.erosiontechusa.com

For more information, contact:

105 Plant Camellia Road
Juliette, GA 31046
478.994.6009

314-C Industrial Park Drive
Waynesville, NC 28786
Phone: 828.452.6900

ECTC Corporate Associate Member

GEOHAY: GEOWATTLE

110 Commercial Road
Spartanburg, SC 29303
864-472-7000
www.geohay.com

Technical Specifications

Purpose

a. Control erosion
b. Prevent water pollution
c. Control sediment
d. Control water flow rate

Composition

a. 100% recycled synthetic carpet fibers (approximately 50% post industrial and 50% post consumer)
b. Reusable

Environmental Safety

a. Contain no weeds or invasive plants
b. Resist mildew or mold

Technical

Erosion abatement system must

a. Meet the following specifications
    i. Contain pre-made stake holes
    ii. Meet the EPA's Toxicity Characteristics Leachate Procedure (TCLP) Standards
    iii. Produced into filter medium with needle punch fibers
    iv. Made reusable by rinsing out within an acceptable area and procedures
    v. Was tested by TRI, Environmental per ASTM D standards 5141 and 7351
b. Maintain its structural integrity during normal rainfall and wet conditions
c. Maintain its structural integrity over the length of the project

Dimensional & Application

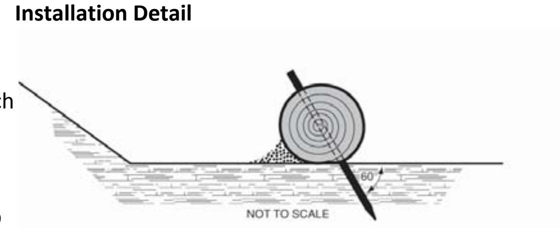
Erosion abatement system must

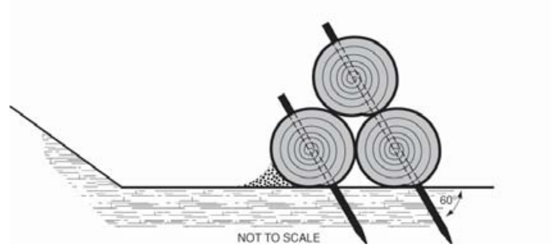
a. Be of a uniform size and weight
    i. 9" diameter x 4', 6', 8', 10', 12' 14', 16',
    – available in a low density weighing 1.25 pounds per foot with a water flow rate of 8.962 GPM/ft²
    available in a high density weighing 2.2 pounds per foot with a water flow rate of 5.223 GPM/ft²
    ii. 12" diameter x 45", 7.5', & 10'
    – approximately 1.33 lbs per linear foot
    iii. 15" diameter x 45", 7.5', & 10'
    – approximately 2.13 lbs per linear foot
    iv. 18" diameter x 45", 7.5', & 10'
    – approximately 2.93 lbs per linear foot
    v. Custom matting will vary in size & weight
b. Be designed for end to end installation
c. Contain pre-made holes for stakes for easy installation
d. Be reusable

Warranties & Certification

GeoHay is manufactured on production lines with identified process controls to provide consistent, uniform products. Each unit is expected to perform per the technical specifications when properly installed. Consistent installation techniques are necessary to insure that GeoHay performs per the technical specifications.

Installation Detail





WESTERN EXCELSIOR: ASPEN EXCELSIOR LOGS

IECA Member

ECTC Directing Member

Specifications

Western Excelsior manufactures Aspen Excelsior Logs in addition to a full line of Rolled Erosion Control Products (RECPs). Aspen Excelsior Logs are a Sediment Retention Fiber Roll (SRFR) consisting of a machine produced High Altitude Rocky Mountain Aspen Excelsior Matrix confined by a synthetic net to form a log of specific length and diameter. Aspen Excelsior Logs are designed to reduce hydraulic energy and filter sediment laden flow in channels and on slopes. The logs are flexible to conform to the soil surface and are secured by staking. Aspen Excelsior Sediment Logs can be ordered in custom lengths to meet specific job conditions.

Each Aspen Excelsior Log is made in the USA and manufactured under Western Excelsior's Quality Assurance Program to ensure a continuous distribution of fibers and consistent dimensions. Log dimensions are provided in Table 1 and product characteristics are provided in Table 2. Installation instructions and performance data are available from Western Excelsior's Technical Support Division.

Table 1 - Specified Expected Values

Diameter	10 ft (3.0 m) Length	20 ft (6.0 m) Length	25 ft (7.6 m) Length
9 in (0.23 m)	25.0 lbs (11.3 kg) 2.5 lbs/ft (3.7 kg/m) 5.8 lbs/ft³ (93.3 kg/m³)	50.0 lbs (22.7 kg) 2.5 lbs/ft (3.7 kg/m) 5.7 lbs/ft³ (92.2 kg/m³)	62.5 lbs (28.4 kg) 2.5 lbs/ft (3.7 kg/m) 5.7 lbs/ft³ (91.9 kg/m³)
12 in (0.31 m)	30.0 lbs (13.6 kg) 3.0 lbs/ft (4.5 kg/m) 4.0 lbs/ft³ (63.5 kg/m³)	60.0 lbs (27.2 kg) 3.0 lbs/ft (4.5 kg/m) 3.9 lbs/ft³ (62.5 kg/m³)	75.0 lbs (34.0 kg) 3.0 lbs/ft (4.5 kg/m) 3.9 lbs/ft³ (62.3 kg/m³)
18 in (0.46 m)	50.0 lbs (22.7 kg) 5.0 lbs/ft (7.4 kg/m) 3.0 lbs/ft³ (47.9 kg/m³)	100.0 lbs (45.4 kg) 5.0 lbs/ft (7.4 kg/m) 2.9 lbs/ft³ (46.7 kg/m³)	125.0 lbs (56.7 kg) 5.0 lbs/ft (7.4 kg/m) 2.9 lbs/ft³ (46.4 kg/m³)
20 in (0.51 m)	50 lbs (22.7 kg) 5.0 lbs/ft (7.4 kg/m) 2.4 lbs/ft³ (39.0 kg/m³)	100.0 lbs (45.4 kg) 5.0 lbs/ft (7.4 kg/m) 2.4 lbs/ft³ (37.9 kg/m³)	125.0 lbs (56.7 kg) 5.0 lbs/ft (7.4 kg/m) 2.3 lbs/ft³ (37.7 kg/m³)

Table 2 - Netting

Fiber Composition	High Altitude Machine Curled Aspen Excelsior
Fiber Dimensions	80% Greater than 6 in.
Netting	0.50" x 0.50" Heavy Duty Synthetic
Configuration	Cylindrical with Closed Ends
End Closure	Hog Ring or Tied

*All values shown measured at the time of manufacture.

Document # WE_EXCEL_AEL_SPEC. This document has been developed to provide the characteristic properties of the product described and supersedes all previous versions. For question, to request performance data or installation recommendations, contact Western Excelsior at 1-866-540-9810 or wexcotech@westernexcelsior.com. Updated 6/4/2013.

WESTERN EXCELSIOR: EXCEL STRAW LOGS

Specifications

Western Excelsior manufactures Excel Straw Logs in addition to a full line of Rolled Erosion Control Products (RECPs). Excel Straw logs consist of 100% clean, certified weed free straw fiber matrix confined by a synthetic net to form a log of specific length and diameter. Excel Straw Logs are designed to reduce hydraulic energy and filter sediment laden flow in channels and on slopes. The logs are flexible to conform to the soil surface and are secured by staking. Excel Straw Logs can be ordered in custom lengths to meet specific job conditions.

Each Excel Straw Log is made in the USA and manufactured under Western Excelsior's Quality Assurance Program to ensure a continuous distribution of fibers and consistent dimensions. Log dimensions are provided in Table 1 and product characteristics are provided in Table 2. Installation instructions and performance data are available from Western Excelsior's Technical Support Division.

Table 1 - Specified Expected Values			
Diameter	10 ft (3.0 m) Length	20 ft (6.0 m) Length	25 ft (7.6 m) Length
9 in (0.23 m)	14.0 lbs (6.4 kg) 1.4 lbs/ft (2.1 kg/m) 3.3 lbs/ft³ (52.3 kg/m³)	28.0 lbs (12.7 kg) 1.4 lbs/ft (2.1 kg/m) 3.2 lbs/ft³ (51.6 kg/m³)	35.0 lbs (15.9 kg) 1.4 lbs/ft (2.1 kg/m) 3.2 lbs/ft³ (51.5 kg/m³)
12 in (0.31 m)	25.0 lbs (11.3 kg) 2.5 lbs/ft (3.7 kg/m) 3.3 lbs/ft³ (53.0 kg/m³)	50.0 lbs (22.7 kg) 2.5 lbs/ft (3.7 kg/m) 3.2 lbs/ft³ (52.1 kg/m³)	62.5 lbs (28.4 kg) 2.5 lbs/ft (3.7 kg/m) 3.2 lbs/ft³ (51.9 kg/m³)
18 in (0.46 m)	35.0 lbs (15.9 kg) 3.5 lbs/ft (5.2 kg/m) 1.7 lbs/ft³ (33.5 kg/m³)	70.0 lbs (31.8 kg) 3.5 lbs/ft (5.2 kg/m) 2.0 lbs/ft³ (32.7 kg/m³)	87.5 lbs (39.7 kg) 3.5 lbs/ft (5.2 kg/m) 2.0 lbs/ft³ (32.5 kg/m³)
20 in (0.51 m)	50.0 lbs (22.7 kg) 5.0 lbs/ft (7.4 kg/m) 2.4 lbs/ft³ (39.0 kg/m³)	100.0 lbs (45.4 kg) 5.0 lbs/ft (7.4 kg/m) 2.4 lbs/ft³ (37.9 kg/m³)	125.0 lbs (56.7 kg) 5.0 lbs/ft (7.4 kg/m) 2.3 lbs/ft³ (37.7 kg/m³)

Table 2 - Netting	
Fiber Composition	100% Clean, Certified Weed Free Straw
Netting	0.50" x 0.50" Heavy Duty Synthetic
Configuration	Cylindrical with Closed Ends
End Closure	Hog Ring or Tied

*All values shown measured at the time of manufacture.

Document # WE_EXCEL_ESL_SPEC. This document has been developed to provide the characteristic properties of the product described and supersedes all previous versions. For question, to request performance data or installation recommendations, contact Western Excelsior at 1-866-540-9810 or wexcotech@westernexcelsior.com. Updated 6/4/2013.

WINTERS EXCELSIOR: WINTERS WATTLES

Winters Excelsior Company

Material Specifications

WintersWattles™

9", 12" & 20" Natural Straw Wattles

WintersWattles™ Products are temporary, degradable, straw logs that are machine-assembled using 100% agricultural straw fibers. The straw fibers are encased in tubular polyethylene netting and captivated on each end.

WintersWattles™ are designed to slow water velocity in channels, on slopes, and in ditches, and to provide protection around inlets. By design, these products, trap sediment, reduce shear stress, and enhance vegetation growth.

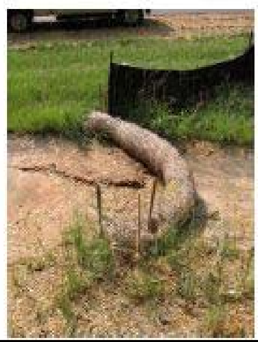
All WintersWattles™ pallets are individually labeled and shrink-wrapped to protect against the weather and damage.

Materials:

100% Certified Weed Free Straw
Polyethylene Netting

Wattle Sizes:

Diameter	**9"**	**12"**	**20"**
Length	25 feet	10 feet	10 feet
Net Openings	.375" x .375"	.5" x .5"	.5" x .5 "
Weight	50 lbs	30 lbs	83 lbs



Performance Characteristics:

Functional Longevity: 12-18 months depending on usage and regional climate and soil conditions.

All figures are based on product at the time it is manufactured.

P.O. Box 39, Highway 21
McWilliams, Al 3 6 7 5 3
800-248-7237
www.WintersExcelsior.com
406 South Obee Rd
Hutchinson, KS 67501

APPENDIX C: WATTLE TIER FLOW AVERAGE TEST DATA

AMERICAN EXCELSIOR: CURLEX SEDIMENT LOG DATA

Cross Sections	Flow Rate: 0.56 cfs (1 pump)															
	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
	06.13.12	06.15.12	06.19.12	Average	06.13.12	06.15.12	06.19.12	Average	06.13.12	06.15.12	06.19.12	Average	06.13.12	06.15.12	06.19.12	Average
CS1	0.17	0.05	0.07	0.10	0.22	0.21	0.21	0.21	0.05	0.16	0.14	0.12	1.79	3.21	3.00	2.67
CS2	0.25	0.06	0.15	0.15	0.28	0.22	0.20	0.23	0.03	0.16	0.05	0.08	1.39	3.21	1.79	2.13
CS3	0.39	0.25	0.24	0.29	0.41	0.30	0.26	0.32	0.02	0.05	0.02	0.03	1.13	1.79	1.13	1.35
CS4	0.53	0.40	0.39	0.44	0.55	0.42	0.41	0.46	0.02	0.02	0.02	0.02	1.13	1.13	1.13	1.13
CS5	0.65	0.49	0.51	0.55	0.66	0.50	0.52	0.56	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS6	0.71	0.56	0.57	0.61	0.74	0.57	0.58	0.63	0.03	0.01	0.01	0.02	1.39	0.80	0.80	1.00
CS7	0.10	0.09	0.09	0.09	0.17	0.18	0.15	0.17	0.07	0.09	0.06	0.07	2.12	2.41	1.97	2.17
CS8	0.10	0.10	0.09	0.10	0.23	0.21	0.18	0.21	0.13	0.11	0.09	0.11	2.89	2.66	2.41	2.65

	Flow Rate: 0.56 cfs (1 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.10	0.11	0.21	0.9	1.11	0	0.90
CS2	0.15	0.07	0.22	0.75	0.97	3	0.93
CS3	0.29	0.03	0.32	0.6	0.92	6	0.97
CS4	0.44	0.02	0.46	0.45	0.91	9	0.98
CS5	0.55	0.01	0.56	0.3	0.86	12	0.99
CS6	0.61	0.02	0.63	0.2	0.83	14	0.98
CS7	0.09	0.07	0.17	0.1	0.27	16	0.73
CS8	0.10	0.11	0.21	0	0.21	18	0.47

	Flow Rate: 1.12 cfs (2 pump)															
Cross Sections	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
	05.30.12	05.31.12	06.05.12	Average	05.30.12	05.31.12	06.05.12	Average	05.30.12	05.31.12	06.05.12	Average	05.30.12	05.31.12	06.05.12	Average
CS1	0.09	0.10	0.08	0.09	0.37	0.36	0.32	0.35	0.27	0.26	0.23	0.26	4.20	4.12	3.88	4.06
CS2	0.07	0.06	0.06	0.06	0.37	0.37	0.34	0.36	0.30	0.31	0.28	0.29	4.40	4.44	4.22	4.35
CS3	0.29	0.20	0.38	0.29	0.39	0.33	0.50	0.41	0.10	0.13	0.12	0.12	2.54	2.93	2.74	2.74
CS4	0.35	0.29	0.34	0.33	0.39	0.32	0.40	0.37	0.04	0.03	0.06	0.04	1.54	1.31	2.02	1.62
CS5	0.40	0.37	0.44	0.40	0.44	0.40	0.47	0.44	0.04	0.02	0.03	0.03	1.54	1.23	1.47	1.41
CS6	0.49	0.44	0.57	0.50	0.53	0.49	0.60	0.54	0.04	0.05	0.03	0.04	1.67	1.79	1.31	1.59
CS7	0.11	0.11	0.12	0.11	0.25	0.29	0.23	0.26	0.14	0.19	0.11	0.14	2.97	3.47	2.66	3.03
CS8	0.12	0.15	0.12	0.13	0.32	0.29	0.28	0.29	0.20	0.14	0.16	0.16	3.59	2.97	3.18	3.24

	Flow Rate: 1.12 cfs (2 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.09	0.26	0.35	0.9	1.25	0	0.79
CS2	0.06	0.29	0.36	0.75	1.11	3	0.73
CS3	0.29	0.12	0.41	0.6	1.01	6	0.88
CS4	0.33	0.04	0.37	0.45	0.82	9	0.95
CS5	0.40	0.03	0.44	0.3	0.74	12	0.96
CS6	0.50	0.04	0.54	0.2	0.74	14	0.95
CS7	0.11	0.14	0.25	0.1	0.35	16	0.60
CS8	0.13	0.16	0.29	0	0.29	18	0.44

Cross Sections	Flow Rate: 1.68 cfs (3 pump)															
	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
	05.30.12	05.31.12	06.05.12	Average	05.30.12	05.31.12	06.05.12	Average	05.30.12	05.31.12	06.05.12	Average	05.30.12	05.31.12	06.05.12	Average
CS1	0.11	0.11	0.11	0.11	0.57	0.47	0.52	0.52	0.47	0.37	0.41	0.42	5.48	4.86	5.16	5.17
CS2	0.28	0.09	0.26	0.21	0.56	0.50	0.41	0.49	0.27	0.40	0.15	0.28	4.20	5.10	3.11	4.13
CS3	0.44	0.34	0.57	0.45	0.54	0.45	0.65	0.55	0.11	0.10	0.08	0.10	2.62	2.58	2.32	2.51
CS4	0.47	0.43	0.54	0.48	0.53	0.54	0.60	0.56	0.06	0.11	0.06	0.08	1.97	2.66	1.97	2.20
CS5	0.52	0.50	0.59	0.54	0.56	0.57	0.65	0.60	0.04	0.07	0.06	0.06	1.67	2.12	2.02	1.94
CS6	0.64	0.58	0.72	0.65	0.69	0.66	0.75	0.70	0.05	0.08	0.03	0.05	1.73	2.22	1.39	1.78
CS7	0.12	0.13	0.13	0.13	0.32	0.31	0.28	0.30	0.19	0.18	0.15	0.17	3.53	3.44	3.07	3.35
CS8	0.14	0.18	0.16	0.16	0.37	0.36	0.35	0.36	0.23	0.18	0.18	0.20	3.88	3.37	3.44	3.56

	Flow Rate: 1.68 cfs (3 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.11	0.41	0.52	0.9	1.42	0	0.71
CS2	0.21	0.27	0.48	0.75	1.23	3	0.78
CS3	0.45	0.10	0.55	0.6	1.15	6	0.91
CS4	0.48	0.07	0.56	0.45	1.01	9	0.93
CS5	0.54	0.06	0.59	0.3	0.89	12	0.93
CS6	0.65	0.05	0.70	0.2	0.90	14	0.95
CS7	0.13	0.17	0.30	0.1	0.40	16	0.57
CS8	0.16	0.20	0.36	0	0.36	18	0.45

AMERICAN EXCELSIOR: EXCEL STRAW LOGS DATA

Cross Sections	Flow Rate: 0.56 cfs (1 pump)															
	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
	01.26.13	01.30.13	03.08.13	Average	01.26.13	01.30.13	03.08.13	Average	01.26.13	01.30.13	03.08.13	Average	01.26.13	01.30.13	03.08.13	Average
CS1	0.46	0.35	0.24	0.35	0.47	0.36	0.26	0.36	0.01	0.01	0.01	0.01	0.80	0.80	0.91	0.84
CS2	0.57	0.44	0.34	0.45	0.58	0.45	0.35	0.46	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS3	0.61	0.41	0.47	0.50	0.62	0.42	0.48	0.51	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS4	0.70	0.50	0.51	0.57	0.71	0.51	0.52	0.58	0.01	0.01	0.01	0.01	0.80	0.80	0.91	0.84
CS5	0.85	0.65	0.64	0.71	0.86	0.66	0.66	0.73	0.01	0.01	0.02	0.01	0.80	0.80	1.02	0.88
CS6	0.91	0.71	0.72	0.78	0.92	0.72	0.73	0.79	0.01	0.01	0.01	0.01	0.80	0.80	0.91	0.84
CS7	0.07	0.02	0.07	0.05	0.13	0.06	0.15	0.11	0.05	0.03	0.08	0.06	1.82	1.35	2.24	1.80
CS8	0.05	0.05	0.09	0.06	0.16	0.13	0.24	0.18	0.11	0.08	0.15	0.11	2.66	2.31	3.03	2.66

	Flow Rate: 0.56 cfs (1 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.35	0.01	0.36	0.9	1.26	0	0.99
CS2	0.45	0.01	0.46	0.75	1.21	3	0.99
CS3	0.50	0.01	0.51	0.6	1.11	6	0.99
CS4	0.57	0.01	0.58	0.45	1.03	9	0.99
CS5	0.71	0.01	0.73	0.3	1.03	12	0.99
CS6	0.78	0.01	0.79	0.2	0.99	14	0.99
CS7	0.05	0.05	0.10	0.1	0.20	16	0.75
CS8	0.06	0.11	0.17	0	0.17	18	0.37

	Flow Rate: 1.12 cfs (2 pump)															
Cross Sections	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
	01.26.13	01.30.13	03.08.13	Average	01.26.13	01.30.13	03.08.13	Average	01.26.13	01.30.13	03.08.13	Average	01.26.13	01.30.13	03.08.13	Average
CS1	0.71	0.61	0.43	0.58	0.72	0.62	0.49	0.61	0.01	0.01	0.06	0.03	0.80	0.91	1.78	1.16
CS2	0.81	0.69	0.52	0.67	0.82	0.70	0.53	0.68	0.01	0.01	0.01	0.01	0.80	0.91	0.80	0.84
CS3	0.84	0.72	0.62	0.73	0.85	0.73	0.63	0.74	0.01	0.01	0.01	0.01	0.80	0.91	0.80	0.84
CS4	0.92	0.80	0.68	0.80	0.93	0.81	0.69	0.81	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS5	1.00	0.87	0.86	0.91	1.01	0.88	0.87	0.92	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS6	1.14	0.91	0.89	0.98	1.15	0.92	0.91	0.99	0.01	0.01	0.02	0.01	0.80	0.80	1.00	0.87
CS7	0.14	0.11	0.07	0.11	0.25	0.22	0.10	0.19	0.11	0.11	0.04	0.08	2.42	2.62	0.89	1.98
CS8	0.11	0.11	0.09	0.10	0.22	0.19	0.14	0.18	0.11	0.07	0.06	0.08	2.64	2.14	1.10	1.96

	Flow Rate: 1.12 cfs (2 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.58	0.02	0.60	0.9	1.50	0	0.99
CS2	0.67	0.01	0.68	0.75	1.43	3	0.99
CS3	0.73	0.01	0.74	0.6	1.34	6	0.99
CS4	0.80	0.01	0.81	0.45	1.26	9	0.99
CS5	0.91	0.01	0.92	0.3	1.22	12	0.99
CS6	0.98	0.01	0.99	0.2	1.19	14	0.99
CS7	0.11	0.06	0.17	0.1	0.27	16	0.77
CS8	0.10	0.06	0.16	0	0.16	18	0.64

	Flow Rate: 1.68 cfs (3 pump)															
Cross Sections	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
	01.26.13	01.30.13	03.08.13	Average	01.26.13	01.30.13	03.08.13	Average	01.26.13	01.30.13	03.08.13	Average	01.26.13	01.30.13	03.08.13	Average
CS1	0.80	0.72	0.62	0.71	0.81	0.73	0.63	0.72	0.01	0.01	0.01	0.01	0.80	0.91	0.65	0.79
CS2	0.90	0.80	0.70	0.80	0.91	0.81	0.73	0.82	0.01	0.01	0.03	0.02	0.80	0.91	1.30	1.01
CS3	0.94	0.84	0.82	0.87	0.95	0.85	0.83	0.88	0.01	0.01	0.01	0.01	0.80	0.91	0.80	0.84
CS4	1.00	0.92	0.87	0.93	1.01	0.93	0.90	0.94	0.01	0.01	0.03	0.02	0.80	0.80	1.30	0.97
CS5	1.11	0.99	1.05	1.05	1.12	1.00	1.10	1.07	0.01	0.01	0.05	0.02	0.53	0.80	1.71	1.02
CS6	1.20	1.15	1.12	1.16	1.21	1.16	1.16	1.18	0.01	0.01	0.04	0.02	0.80	0.80	1.52	1.04
CS7	0.20	0.14	0.07	0.14	0.26	0.24	0.31	0.27	0.06	0.10	0.24	0.13	1.91	2.36	3.16	2.47
CS8	0.13	0.11	0.10	0.11	0.23	0.22	0.18	0.21	0.09	0.11	0.08	0.09	2.22	2.65	1.84	2.23

	Flow Rate: 1.68 cfs (3 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.71	0.01	0.72	0.9	1.62	0	0.99
CS2	0.80	0.02	0.82	0.75	1.57	3	0.99
CS3	0.87	0.01	0.88	0.6	1.48	6	0.99
CS4	0.93	0.01	0.94	0.45	1.39	9	0.99
CS5	1.05	0.02	1.07	0.3	1.37	12	0.99
CS6	1.16	0.02	1.17	0.2	1.37	14	0.99
CS7	0.14	0.09	0.23	0.1	0.33	16	0.71
CS8	0.11	0.08	0.19	0	0.19	18	0.60

EAST COAST EROSION: ECWATTLE

Cross Sections	Flow Rate: 0.56 cfs (1 pump)															
	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
	03.14.13	03.20.13	03.25.13	AVG	03.14.13	03.20.13	03.25.13	AVG	03.14.13	03.20.13	03.25.13	AVG	03.14.13	03.20.13	03.25.13	AVG
CS1	0.33	0.43	0.44	0.40	0.34	0.44	0.45	0.41	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS2	0.44	0.54	0.52	0.50	0.45	0.55	0.53	0.51	0.01	0.01	0.01	0.01	0.93	0.80	0.80	0.84
CS3	0.53	0.64	0.61	0.59	0.54	0.65	0.62	0.60	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS4	0.57	0.69	0.70	0.66	0.58	0.70	0.71	0.67	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS5	0.69	0.79	0.90	0.79	0.70	0.80	0.91	0.80	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS6	0.79	0.86	0.88	0.84	0.80	0.87	0.89	0.85	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS7	0.05	0.07	0.06	0.06	0.23	0.16	0.17	0.19	0.18	0.08	0.11	0.12	3.40	2.32	2.62	2.78
CS8	0.05	0.06	0.09	0.07	0.22	0.22	0.21	0.22	0.16	0.16	0.12	0.15	3.24	3.24	2.78	3.09

	Flow Rate: 0.56 cfs (1 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.40	0.01	0.41	0.9	1.31	0	0.99
CS2	0.50	0.01	0.51	0.75	1.26	3	0.99
CS3	0.59	0.01	0.60	0.6	1.20	6	0.99
CS4	0.66	0.01	0.67	0.45	1.12	9	0.99
CS5	0.79	0.01	0.80	0.3	1.10	12	0.99
CS6	0.84	0.01	0.85	0.2	1.05	14	0.99
CS7	0.06	0.12	0.18	0.1	0.28	16	0.57
CS8	0.07	0.15	0.21	0	0.21	18	0.31

Cross Sections	Flow Rate: 1.12 cfs (2 pump)															
	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
	03.14.13	03.20.13	03.25.13	AVG	03.14.13	03.20.13	03.25.13	AVG	03.14.13	03.20.13	03.25.13	AVG	03.14.13	03.20.13	03.25.13	AVG
CS1	0.49	0.65	0.64	0.59	0.50	0.66	0.65	0.60	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS2	0.57	0.73	0.76	0.69	0.58	0.74	0.77	0.70	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS3	0.66	0.84	0.84	0.78	0.67	0.85	0.85	0.79	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS4	0.73	0.92	0.92	0.86	0.74	0.93	0.93	0.87	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS5	0.82	1.00	1.06	0.96	0.84	1.01	1.07	0.97	0.02	0.01	0.01	0.01	1.13	0.80	0.80	0.91
CS6	0.90	1.04	1.13	1.02	0.92	1.05	1.14	1.04	0.02	0.01	0.01	0.01	1.13	0.80	0.80	0.91
CS7	0.07	0.12	0.08	0.09	0.51	0.30	0.39	0.40	0.44	0.18	0.31	0.31	5.34	3.40	4.49	4.41
CS8	0.10	0.12	0.14	0.12	0.30	0.25	0.25	0.27	0.20	0.13	0.11	0.15	3.62	2.89	2.70	3.07

	Flow Rate: 1.12 cfs (2 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.65	0.01	0.66	0.9	1.56	0	0.99
CS2	0.73	0.01	0.74	0.75	1.49	3	0.99
CS3	0.84	0.01	0.85	0.6	1.45	6	0.99
CS4	0.92	0.01	0.93	0.45	1.38	9	0.99
CS5	1.00	0.01	1.01	0.3	1.31	12	0.99
CS6	1.04	0.01	1.05	0.2	1.25	14	0.99
CS7	0.12	0.16	0.28	0.1	0.38	16	0.58
CS8	0.12	0.13	0.24	0	0.24	18	0.48

	Flow Rate: 1.68 cfs (3 pump)															
Cross	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
Sections	03.14.13	03.20.13	03.25.13	AVG	03.14.13	03.20.13	03.25.13	AVG	03.14.13	03.20.13	03.25.13	AVG	03.14.13	03.20.13	03.25.13	AVG
CS1	0.64	0.71	0.72	0.69	0.65	0.72	0.73	0.70	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS2	0.71	0.80	0.81	0.77	0.72	0.81	0.82	0.78	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS3	0.79	0.88	0.97	0.88	0.80	0.89	0.98	0.89	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS4	0.85	0.95	1.03	0.94	0.86	0.96	1.04	0.95	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS5	0.96	1.02	1.11	1.03	0.98	1.03	1.12	1.04	0.02	0.01	0.01	0.01	1.13	0.80	0.80	0.91
CS6	1.02	1.12	1.20	1.11	1.03	1.13	1.21	1.12	0.02	0.01	0.01	0.01	1.04	0.80	0.80	0.88
CS7	0.12	0.17	0.09	0.13	0.42	0.83	0.55	0.60	0.30	0.66	0.46	0.47	4.37	6.50	5.42	5.43
CS8	0.14	0.19	0.17	0.17	0.21	0.38	0.29	0.29	0.07	0.19	0.12	0.13	2.12	3.50	2.78	2.80

	Flow Rate: 1.68 cfs (3 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elev. (ft)	EGL (ft)	Dist. from CS1	HGL:EGL
CS1	0.64	0.01	0.65	0.9	1.55	0	0.99
CS2	0.76	0.01	0.77	0.75	1.52	3	0.99
CS3	0.84	0.01	0.85	0.6	1.45	6	0.99
CS4	0.92	0.01	0.93	0.45	1.38	9	0.99
CS5	1.06	0.01	1.07	0.3	1.37	12	0.99
CS6	1.13	0.01	1.14	0.2	1.34	14	0.99
CS7	0.08	0.26	0.34	0.1	0.44	16	0.40
CS8	0.14	0.11	0.25	0	0.25	18	0.55

EROSION TECH: WHEAT STRAW SEDIMENT LOGS

Cross	Flow Rate: 0.56 cfs (1 pump)															
	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
Sections	06.13.12	06.15.12	06.19.12	Average	06.13.12	06.15.12	06.19.12	Average	06.13.12	06.15.12	06.19.12	Average	06.13.12	06.15.12	06.19.12	Average
CS1	0.17	0.05	0.07	0.10	0.22	0.21	0.21	0.21	0.05	0.16	0.14	0.12	1.79	3.21	3.00	2.67
CS2	0.25	0.06	0.15	0.15	0.28	0.22	0.20	0.23	0.03	0.16	0.05	0.08	1.39	3.21	1.79	2.13
CS3	0.39	0.25	0.24	0.29	0.41	0.30	0.26	0.32	0.02	0.05	0.02	0.03	1.13	1.79	1.13	1.35
CS4	0.53	0.40	0.39	0.44	0.55	0.42	0.41	0.46	0.02	0.02	0.02	0.02	1.13	1.13	1.13	1.13
CS5	0.65	0.49	0.51	0.55	0.66	0.50	0.52	0.56	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS6	0.71	0.56	0.57	0.61	0.74	0.57	0.58	0.63	0.03	0.01	0.01	0.02	1.39	0.80	0.80	1.00
CS7	0.10	0.09	0.09	0.09	0.17	0.18	0.15	0.17	0.07	0.09	0.06	0.07	2.12	2.41	1.97	2.17
CS8	0.10	0.10	0.09	0.10	0.23	0.21	0.18	0.21	0.13	0.11	0.09	0.11	2.89	2.66	2.41	2.65

	Flow Rate: 0.56 cfs (1 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.10	0.11	0.21	0.9	1.11	0	0.90
CS2	0.15	0.07	0.22	0.75	0.97	3	0.93
CS3	0.29	0.03	0.32	0.6	0.92	6	0.97
CS4	0.44	0.02	0.46	0.45	0.91	9	0.98
CS5	0.55	0.01	0.56	0.3	0.86	12	0.99
CS6	0.61	0.02	0.63	0.2	0.83	14	0.98
CS7	0.09	0.07	0.17	0.1	0.27	16	0.73
CS8	0.10	0.11	0.21	0	0.21	18	0.47

Cross Sections	Flow Rate: 1.12 cfs (2 pump)															
	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
	06.13.12	06.15.12	06.19.12	Average	06.13.12	06.15.12	06.19.12	Average	06.13.12	06.15.12	06.19.12	Average	06.13.12	06.15.12	06.19.12	Average
CS1	0.39	0.20	0.28	0.29	0.44	0.33	0.36	0.38	0.05	0.13	0.08	0.09	1.79	2.89	2.27	2.32
CS2	0.50	0.32	0.38	0.40	0.53	0.35	0.41	0.43	0.03	0.03	0.03	0.03	1.39	1.39	1.39	1.39
CS3	0.64	0.52	0.50	0.55	0.65	0.56	0.53	0.58	0.01	0.04	0.03	0.03	0.80	1.60	1.39	1.27
CS4	0.77	0.64	0.63	0.68	0.78	0.66	0.64	0.69	0.01	0.02	0.01	0.01	0.80	1.13	0.80	0.91
CS5	0.90	0.74	0.75	0.80	0.91	0.75	0.76	0.81	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS6	0.96	0.81	0.83	0.87	0.98	0.84	0.84	0.89	0.02	0.03	0.01	0.02	1.13	1.39	0.80	1.11
CS7	0.13	0.11	0.10	0.11	0.20	0.26	0.19	0.22	0.07	0.15	0.09	0.10	2.12	3.11	2.41	2.55
CS8	0.14	0.11	0.13	0.13	0.25	0.22	0.29	0.25	0.11	0.11	0.16	0.13	2.66	2.66	3.21	2.84

	Flow Rate: 1.12 cfs (2 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.10	0.11	0.21	0.9	1.11	0	0.90
CS2	0.15	0.07	0.22	0.75	0.97	3	0.93
CS3	0.29	0.03	0.32	0.6	0.92	6	0.97
CS4	0.44	0.02	0.46	0.45	0.91	9	0.98
CS5	0.55	0.01	0.56	0.3	0.86	12	0.99
CS6	0.61	0.02	0.63	0.2	0.83	14	0.98
CS7	0.09	0.07	0.17	0.1	0.27	16	0.73
CS8	0.10	0.11	0.21	0	0.21	18	0.47

	Flow Rate: 1.68 cfs (3 pump)															
Cross	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
Sections	06.13.12	06.15.12	06.19.12	Average	06.13.12	06.15.12	06.19.12	Average	06.13.12	06.15.12	06.19.12	Average	06.13.12	06.15.12	06.19.12	Average
CS1	0.51	0.34	0.40	0.42	0.56	0.46	0.44	0.49	0.05	0.12	0.04	0.07	1.79	2.78	1.60	2.06
CS2	0.59	0.44	0.51	0.51	0.62	0.51	0.53	0.55	0.03	0.07	0.02	0.04	1.39	2.17	1.13	1.57
CS3	0.72	0.62	0.60	0.65	0.75	0.65	0.62	0.67	0.03	0.03	0.02	0.03	1.39	1.39	1.04	1.27
CS4	0.85	0.63	0.76	0.75	0.87	0.66	0.77	0.77	0.02	0.03	0.01	0.02	1.13	1.31	0.93	1.12
CS5	0.98	0.72	0.87	0.86	1.00	0.74	0.89	0.88	0.02	0.02	0.02	0.02	1.13	1.13	1.13	1.13
CS6	1.04	0.79	0.96	0.93	1.07	0.82	0.99	0.96	0.03	0.03	0.03	0.03	1.39	1.47	1.39	1.42
CS7	0.13	0.14	0.12	0.13	0.24	0.31	0.34	0.30	0.11	0.17	0.22	0.16	2.62	3.31	3.74	3.22
CS8	0.15	0.14	0.14	0.14	0.26	0.38	0.32	0.32	0.11	0.24	0.18	0.18	2.66	3.93	3.44	3.34

	Flow Rate: 1.68 cfs (3 pump)						
Cross Sections	**Water Depth (ft)**	$v^2/2g$ (ft)	s_f (ft)	**Bottom Elevation (ft)**	**EGL (ft)**	**Distance from CS1**	**HGL:EGL**
CS1	0.10	0.11	0.21	0.9	1.11	0	0.90
CS2	0.15	0.07	0.22	0.75	0.97	3	0.93
CS3	0.29	0.03	0.32	0.6	0.92	6	0.97
CS4	0.44	0.02	0.46	0.45	0.91	9	0.98
CS5	0.55	0.01	0.56	0.3	0.86	12	0.99
CS6	0.61	0.02	0.63	0.2	0.83	14	0.98
CS7	0.09	0.07	0.17	0.1	0.27	16	0.73
CS8	0.10	0.11	0.21	0	0.21	18	0.47

GEOHAY: GEOWATTLE

	Flow Rate: 0.56 cfs (1 pump)															
Cross	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
Sections	06.13.12	06.15.12	06.19.12	Average	06.13.12	06.15.12	06.19.12	Average	06.13.12	06.15.12	06.19.12	Average	06.13.12	06.15.12	06.19.12	Average
CS1	0.37	0.22	0.34	0.31	0.39	0.25	0.37	0.33	0.02	0.03	0.02	0.02	1.13	1.39	1.23	1.25
CS2	0.48	0.30	0.42	0.40	0.50	0.32	0.43	0.42	0.02	0.02	0.02	0.02	1.13	1.04	1.04	1.07
CS3	0.61	0.39	0.59	0.53	0.62	0.40	0.60	0.54	0.01	0.01	0.01	0.01	0.80	0.80	0.93	0.84
CS4	0.66	0.48	0.72	0.62	0.67	0.49	0.73	0.63	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS5	0.82	0.61	0.80	0.74	0.84	0.62	0.81	0.76	0.02	0.01	0.01	0.01	1.13	0.80	0.80	0.91
CS6	0.95	0.75	0.88	0.86	0.96	0.76	0.89	0.87	0.01	0.01	0.01	0.01	0.80	0.80	0.93	0.84
CS7	0.07	0.07	0.08	0.07	0.15	0.18	0.19	0.17	0.08	0.11	0.11	0.10	2.27	2.66	2.62	2.52
CS8	0.06	0.07	0.06	0.06	0.15	0.24	0.25	0.21	0.09	0.17	0.19	0.15	2.41	3.31	3.50	3.07

	Flow Rate: 0.56 cfs (1 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.31	0.02	0.33	0.9	1.23	0	0.98
CS2	0.40	0.02	0.42	0.75	1.17	3	0.98
CS3	0.53	0.01	0.54	0.6	1.14	6	0.99
CS4	0.62	0.01	0.63	0.45	1.08	9	0.99
CS5	0.74	0.01	0.76	0.3	1.06	12	0.99
CS6	0.86	0.01	0.87	0.2	1.07	14	0.99
CS7	0.07	0.10	0.17	0.1	0.27	16	0.64
CS8	0.06	0.15	0.21	0	0.21	18	0.30

	Flow Rate: 1.12 cfs (2 pump)															
Cross Sections	**Water Depth (ft)**				**Water Depth + Velocity Head (ft)**				**Velocity Head (ft)**				**Velocity (ft/sec)**			
	06.13.12	**06.15.12**	**06.19.12**	**Average**	**06.13.12**	**06.15.12**	**06.19.12**	**Average**	**06.13.12**	**06.15.12**	**06.19.12**	**Average**	**06.13.12**	**06.15.12**	**06.19.12**	**Average**
CS1	0.73	0.51	0.68	0.64	0.76	0.53	0.70	0.66	0.03	0.02	0.02	0.02	1.39	1.04	1.13	1.19
CS2	0.86	0.61	0.74	0.74	0.88	0.64	0.75	0.76	0.02	0.03	0.02	0.02	1.13	1.31	1.04	1.16
CS3	0.97	0.74	0.94	0.88	0.99	0.75	0.96	0.90	0.02	0.01	0.02	0.02	1.13	0.93	1.04	1.03
CS4	1.04	0.83	1.09	0.99	1.05	0.84	1.11	1.00	0.01	0.01	0.02	0.01	0.80	0.66	1.23	0.89
CS5	1.20	0.98	1.10	1.10	1.21	0.99	1.12	1.11	0.01	0.01	0.02	0.01	0.80	0.80	1.13	0.91
CS6	1.30	1.12	1.23	1.21	1.32	1.13	1.24	1.23	0.02	0.01	0.01	0.01	1.13	0.80	0.80	0.91
CS7	0.12	0.10	0.09	0.10	0.19	0.47	0.24	0.30	0.07	0.38	0.15	0.20	2.12	4.93	3.11	3.39
CS8	0.13	0.10	0.14	0.12	0.26	0.40	0.31	0.32	0.13	0.31	0.17	0.20	2.89	4.44	3.31	3.55

	Flow Rate: 0.56 cfs (1 pump)						
Cross Sections	**Water Depth (ft)**	$v^2/2g$ (ft)	s_f (ft)	**Bottom Elevation (ft)**	**EGL (ft)**	**Distance from CS1**	**HGL:EGL**
CS1	0.64	0.02	0.66	0.9	1.56	0	0.99
CS2	0.74	0.02	0.76	0.75	1.51	3	0.99
CS3	0.88	0.02	0.90	0.6	1.50	6	0.99
CS4	0.99	0.01	1.00	0.45	1.45	9	0.99
CS5	1.10	0.01	1.11	0.3	1.41	12	0.99
CS6	1.21	0.01	1.23	0.2	1.43	14	0.99
CS7	0.10	0.18	0.28	0.1	0.38	16	0.53
CS8	0.12	0.20	0.32	0	0.32	18	0.38

Cross Sections	Flow Rate: 1.68 cfs (3 pump)															
	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
	06.13.12	06.15.12	06.19.12	Average	06.13.12	06.15.12	06.19.12	Average	06.13.12	06.15.12	06.19.12	Average	06.13.12	06.15.12	06.19.12	Average
CS1	0.91	0.75	0.78	0.82	0.93	0.77	0.81	0.84	0.02	0.02	0.02	0.02	1.13	1.13	1.23	1.17
CS2	0.99	0.83	0.86	0.89	1.00	0.85	0.88	0.91	0.01	0.02	0.01	0.01	0.80	1.04	0.93	0.92
CS3	1.07	0.97	1.01	1.01	1.09	0.98	1.03	1.03	0.02	0.02	0.02	0.02	1.13	1.04	1.13	1.10
CS4	1.11	1.04	1.14	1.10	1.12	1.05	1.15	1.11	0.01	0.01	0.01	0.01	0.80	0.80	0.93	0.84
CS5	1.26	1.21	1.23	1.23	1.27	1.22	1.24	1.24	0.01	0.01	0.01	0.01	0.80	0.80	0.80	0.80
CS6	1.35	1.34	1.32	1.34	1.36	1.35	1.34	1.35	0.01	0.01	0.02	0.01	0.80	0.80	1.04	0.88
CS7	0.17	0.12	0.14	0.14	0.25	0.43	0.46	0.38	0.08	0.31	0.33	0.24	2.27	4.47	4.59	3.77
CS8	0.18	0.11	0.18	0.16	0.36	0.39	0.39	0.38	0.18	0.28	0.21	0.23	3.40	4.27	3.71	3.79

	Flow Rate: 0.56 cfs (1 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.82	0.02	0.84	0.9	1.74	0	0.99
CS2	0.89	0.01	0.91	0.75	1.66	3	0.99
CS3	1.01	0.02	1.03	0.6	1.63	6	0.99
CS4	1.10	0.01	1.11	0.45	1.56	9	0.99
CS5	1.23	0.01	1.24	0.3	1.54	12	0.99
CS6	1.34	0.01	1.35	0.2	1.55	14	0.99
CS7	0.14	0.22	0.36	0.1	0.46	16	0.52
CS8	0.16	0.22	0.38	0	0.38	18	0.41

WESTERN EXCELSIOR: ASPEN EXCEL LOGS DATA

Cross	Flow Rate: 0.56 cfs (1 pump)															
	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
Sections	02.20.13	03.01.13	03.04.13	AVG	02.20.13	03.01.13	03.04.13	AVG	02.20.13	03.01.13	03.04.13	AVG	02.20.13	03.01.13	03.04.13	AVG
CS1	0.01	0.05	0.07	0.04	0.20	0.31	0.25	0.25	0.19	0.26	0.18	0.21	3.47	4.07	3.44	3.66
CS2	0.01	0.05	0.03	0.03	0.20	0.27	0.23	0.23	0.19	0.22	0.20	0.20	3.47	3.74	3.62	3.61
CS3	0.07	0.10	0.06	0.08	0.16	0.16	0.19	0.17	0.09	0.06	0.12	0.09	2.41	2.02	2.82	2.42
CS4	0.13	0.16	0.19	0.16	0.15	0.21	0.21	0.19	0.02	0.05	0.02	0.03	1.23	1.73	1.13	1.36
CS5	0.23	0.36	0.33	0.31	0.26	0.37	0.34	0.32	0.03	0.01	0.01	0.02	1.31	0.80	0.80	0.97
CS6	0.37	0.43	0.44	0.42	0.38	0.44	0.46	0.43	0.01	0.01	0.02	0.01	0.80	0.80	1.04	0.88
CS7	0.15	0.08	0.05	0.10	0.18	0.27	0.20	0.22	0.03	0.18	0.15	0.12	1.39	3.44	3.11	2.64
CS8	0.20	0.08	0.10	0.13	0.23	0.17	0.25	0.21	0.03	0.08	0.15	0.09	1.31	2.32	3.11	2.25

	Flow Rate: 0.56 cfs (1 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.04	0.21	0.25	0.9	1.15	0	0.82
CS2	0.03	0.20	0.23	0.75	0.98	3	0.79
CS3	0.08	0.09	0.17	0.6	0.77	6	0.88
CS4	0.16	0.03	0.19	0.45	0.64	9	0.95
CS5	0.31	0.01	0.32	0.3	0.62	12	0.98
CS6	0.42	0.01	0.43	0.2	0.63	14	0.98
CS7	0.10	0.11	0.20	0.1	0.30	16	0.64
CS8	0.13	0.08	0.21	0	0.21	18	0.62

	Flow Rate: 1.12 cfs (2 pump)															
Cross	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
Sections	02.20.13	03.01.13	03.04.13	AVG	02.20.13	03.01.13	03.04.13	AVG	02.20.13	03.01.13	03.04.13	AVG	02.20.13	03.01.13	03.04.13	AVG
CS1	0.15	0.07	0.08	0.10	0.33	0.30	0.40	0.34	0.18	0.23	0.32	0.24	3.44	3.88	4.52	3.94
CS2	0.15	0.12	0.25	0.17	0.25	0.39	0.32	0.32	0.10	0.27	0.07	0.15	2.54	4.17	2.12	2.94
CS3	0.24	0.30	0.28	0.27	0.32	0.31	0.35	0.33	0.08	0.01	0.07	0.05	2.22	0.93	2.12	1.76
CS4	0.34	0.38	0.39	0.37	0.35	0.39	0.42	0.39	0.01	0.01	0.03	0.02	0.80	0.80	1.39	1.00
CS5	0.49	0.54	0.56	0.53	0.50	0.55	0.58	0.54	0.01	0.01	0.02	0.01	0.80	0.80	1.13	0.91
CS6	0.61	0.63	0.68	0.64	0.62	0.64	0.71	0.66	0.01	0.01	0.03	0.02	0.80	0.80	1.31	0.97
CS7	0.14	0.12	0.08	0.11	0.23	0.20	0.15	0.19	0.09	0.08	0.08	0.08	2.36	2.32	2.22	2.30
CS8	0.14	0.13	0.10	0.13	0.26	0.24	0.24	0.25	0.12	0.11	0.14	0.12	2.78	2.66	2.97	2.80

	Flow Rate: 1.12 cfs (2 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.10	0.24	0.34	0.9	1.24	0	0.81
CS2	0.17	0.13	0.31	0.75	1.06	3	0.87
CS3	0.27	0.05	0.32	0.6	0.92	6	0.95
CS4	0.37	0.02	0.39	0.45	0.84	9	0.98
CS5	0.53	0.01	0.54	0.3	0.84	12	0.98
CS6	0.64	0.01	0.65	0.2	0.85	14	0.98
CS7	0.11	0.08	0.19	0.1	0.29	16	0.72
CS8	0.13	0.12	0.25	0	0.25	18	0.51

Cross Sections	Flow Rate: 1.68 cfs (3 pump)															
	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
	02.20.13	03.01.13	03.04.13	AVG	02.20.13	03.01.13	03.04.13	AVG	02.20.13	03.01.13	03.04.13	AVG	02.20.13	03.01.13	03.04.13	AVG
CS1	0.25	0.14	0.25	0.21	0.37	0.41	0.49	0.42	0.12	0.27	0.25	0.21	2.74	4.17	3.99	3.63
CS2	0.40	0.31	0.38	0.36	0.49	0.43	0.55	0.49	0.09	0.12	0.17	0.13	2.36	2.78	3.31	2.82
CS3	0.48	0.42	0.45	0.45	0.54	0.48	0.48	0.50	0.06	0.06	0.03	0.05	1.91	1.97	1.39	1.76
CS4	0.47	0.49	0.53	0.50	0.48	0.51	0.58	0.53	0.01	0.02	0.06	0.03	0.80	1.13	1.91	1.28
CS5	0.63	0.61	0.66	0.63	0.64	0.63	0.67	0.65	0.01	0.02	0.01	0.01	0.80	1.04	0.80	0.88
CS6	0.71	0.74	0.81	0.75	0.72	0.82	0.82	0.79	0.01	0.08	0.01	0.04	0.93	2.27	0.93	1.37
CS7	0.13	0.14	0.13	0.14	0.31	0.34	0.32	0.32	0.18	0.20	0.18	0.19	3.40	3.62	3.44	3.49
CS8	0.12	0.17	0.19	0.16	0.28	0.32	0.38	0.33	0.16	0.15	0.19	0.17	3.21	3.14	3.53	3.29

	Flow Rate: 1.68 cfs (3 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elev. (ft)	EGL (ft)	Dist. from CS1	HGL:EGL
CS1	0.21	0.20	0.42	0.9	1.32	0	0.84
CS2	0.36	0.12	0.49	0.75	1.24	3	0.90
CS3	0.45	0.05	0.50	0.6	1.10	6	0.96
CS4	0.50	0.03	0.52	0.45	0.97	9	0.97
CS5	0.63	0.01	0.65	0.3	0.95	12	0.99
CS6	0.75	0.03	0.78	0.2	0.98	14	0.97
CS7	0.14	0.19	0.32	0.1	0.42	16	0.56
CS8	0.16	0.17	0.33	0	0.33	18	0.49

WESTERN EXCELSIOR: EXCEL STRAW LOGS DATA

	Flow Rate: 0.56 cfs (1 pump)															
Cross	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
Sections	12.02.11	01.25.12	02.01.12	Average	12.02.11	01.25.12	02.01.12	Average	12.02.11	01.25.12	02.01.12	Average	12.02.11	01.25.12	02.01.12	Average
CS1	0.21	0.05	0.11	0.12	0.25	0.13	0.19	0.19	0.04	0.08	0.07	0.06	1.60	2.22	2.17	2.00
CS2	0.30	0.13	0.22	0.22	0.32	0.16	0.24	0.24	0.02	0.03	0.02	0.02	1.23	1.31	1.13	1.22
CS3	0.39	0.25	0.34	0.33	0.41	0.27	0.35	0.34	0.02	0.02	0.01	0.02	1.13	1.04	0.80	0.99
CS4	0.45	0.26	0.38	0.36	0.46	0.29	0.39	0.38	0.02	0.02	0.01	0.02	1.04	1.23	0.93	1.06
CS5	0.55	0.37	0.44	0.45	0.57	0.39	0.46	0.47	0.02	0.02	0.02	0.02	1.04	1.13	1.04	1.07
CS6	0.61	0.44	0.52	0.52	0.62	0.47	0.54	0.55	0.01	0.03	0.03	0.02	0.93	1.47	1.31	1.23
CS7	0.06	0.05	0.05	0.05	0.14	0.14	0.14	0.14	0.08	0.10	0.09	0.09	2.22	2.50	2.41	2.37
CS8	0.04	0.04	0.07	0.05	0.25	0.21	0.23	0.23	0.21	0.17	0.16	0.18	3.68	3.34	3.24	3.42

	Flow Rate: 0.56 cfs (1 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.12	0.06	0.19	0.9	1.09	0	0.94
CS2	0.22	0.02	0.24	0.75	0.99	3	0.98
CS3	0.33	0.02	0.34	0.6	0.94	6	0.98
CS4	0.36	0.02	0.38	0.45	0.83	9	0.98
CS5	0.45	0.02	0.47	0.3	0.77	12	0.98
CS6	0.52	0.02	0.54	0.2	0.74	14	0.97
CS7	0.05	0.09	0.14	0.1	0.24	16	0.64
CS8	0.05	0.18	0.23	0	0.23	18	0.21

	Flow Rate: 1.12 cfs (2 pump)															
Cross	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
Sections	12.02.11	01.25.12	02.01.12	Average	12.02.11	01.25.12	02.01.12	Average	12.02.11	01.25.12	02.01.12	Average	12.02.11	01.25.12	02.01.12	Average
CS1	0.36	0.24	0.33	0.31	0.42	0.31	0.36	0.36	0.05	0.06	0.03	0.05	1.85	2.02	1.47	1.78
CS2	0.45	0.32	0.41	0.40	0.49	0.36	0.45	0.43	0.03	0.04	0.04	0.04	1.47	1.54	1.54	1.51
CS3	0.54	0.45	0.54	0.51	0.55	0.48	0.55	0.53	0.02	0.03	0.02	0.02	1.04	1.39	1.04	1.15
CS4	0.58	0.47	0.58	0.54	0.59	0.50	0.60	0.56	0.01	0.03	0.02	0.02	0.93	1.39	1.13	1.15
CS5	0.70	0.59	0.65	0.65	0.72	0.61	0.66	0.66	0.02	0.02	0.01	0.01	1.04	1.04	0.80	0.96
CS6	0.79	0.68	0.71	0.73	0.80	0.70	0.72	0.74	0.01	0.02	0.01	0.01	0.80	1.04	0.80	0.88
CS7	0.07	0.09	0.07	0.08	0.18	0.20	0.20	0.19	0.11	0.11	0.12	0.12	2.66	2.70	2.82	2.73
CS8	0.07	0.09	0.11	0.09	0.31	0.29	0.33	0.31	0.24	0.20	0.22	0.22	3.90	3.59	3.74	3.74

	Flow Rate: 1.12 cfs (2 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.31	0.05	0.36	0.9	1.26	0	0.96
CS2	0.40	0.04	0.43	0.75	1.18	3	0.97
CS3	0.51	0.02	0.53	0.6	1.13	6	0.98
CS4	0.54	0.02	0.56	0.45	1.01	9	0.98
CS5	0.65	0.01	0.66	0.3	0.96	12	0.99
CS6	0.73	0.01	0.74	0.2	0.94	14	0.99
CS7	0.08	0.12	0.19	0.1	0.29	16	0.61
CS8	0.09	0.22	0.31	0	0.31	18	0.30

	Flow Rate: 1.68 cfs (3 pump)															
Cross	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
Sections	12.02.11	01.25.12	02.01.12	Average	12.02.11	01.25.12	02.01.12	Average	12.02.11	01.25.12	02.01.12	Average	12.02.11	01.25.12	02.01.12	Average
CS1	0.50	0.36	0.46	0.44	0.52	0.43	0.51	0.49	0.03	0.07	0.05	0.05	1.31	2.17	1.79	1.76
CS2	0.57	0.46	0.54	0.52	0.61	0.52	0.56	0.56	0.04	0.06	0.02	0.04	1.60	1.91	1.13	1.55
CS3	0.65	0.55	0.66	0.62	0.67	0.57	0.69	0.64	0.02	0.02	0.03	0.02	1.04	1.13	1.31	1.16
CS4	0.69	0.56	0.70	0.65	0.70	0.58	0.72	0.67	0.01	0.03	0.02	0.02	0.93	1.31	1.04	1.09
CS5	0.81	0.68	0.78	0.76	0.83	0.69	0.79	0.77	0.02	0.01	0.01	0.02	1.13	0.93	0.93	1.00
CS6	0.90	0.76	0.84	0.83	0.93	0.78	0.87	0.86	0.02	0.03	0.02	0.02	1.23	1.31	1.23	1.25
CS7	0.09	0.10	0.10	0.10	0.52	0.33	0.42	0.42	0.43	0.23	0.31	0.32	5.26	3.82	4.49	4.53
CS8	0.10	0.12	0.12	0.11	0.56	0.39	0.38	0.44	0.46	0.27	0.26	0.33	5.42	4.20	4.07	4.56

	Flow Rate: 1.68 cfs (3 pump)						
Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.44	0.05	0.49	0.9	1.39	0	0.97
CS2	0.52	0.04	0.56	0.75	1.31	3	0.97
CS3	0.62	0.02	0.64	0.6	1.24	6	0.98
CS4	0.65	0.02	0.67	0.45	1.12	9	0.98
CS5	0.76	0.02	0.77	0.3	1.07	12	0.99
CS6	0.83	0.02	0.86	0.2	1.06	14	0.98
CS7	0.10	0.32	0.42	0.1	0.52	16	0.38
CS8	0.11	0.32	0.44	0	0.44	18	0.26

WINTERS EXCELSIOR: WINTERS WATTLES DATA

Cross Sections	Flow Rate: 0.56 cfs (1 pump)															
	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
	R1	R2	R3	Average	R1	R2	R3	Average	R1	R2	R3	Average	R1	R2	R3	Average
CS1	0.27	0.26	0.28	0.27	0.30	0.28	0.31	0.30	0.03	0.02	0.02	0.02	1.31	1.13	1.23	1.22
CS2	0.39	0.36	0.39	0.38	0.41	0.39	0.41	0.40	0.02	0.03	0.02	0.02	1.13	1.39	1.13	1.22
CS3	0.51	0.49	0.51	0.50	0.52	0.50	0.54	0.52	0.01	0.02	0.02	0.02	0.80	1.04	1.23	1.02
CS4	0.60	0.53	0.56	0.56	0.61	0.54	0.57	0.58	0.02	0.01	0.01	0.01	1.04	0.93	0.93	0.96
CS5	0.66	0.61	0.69	0.65	0.67	0.62	0.71	0.67	0.01	0.01	0.02	0.01	0.93	0.80	1.04	0.92
CS6	0.73	0.70	0.78	0.74	0.75	0.71	0.80	0.75	0.02	0.02	0.02	0.02	1.04	1.04	1.13	1.07
CS7	0.04	0.04	0.04	0.04	0.12	0.12	0.13	0.12	0.08	0.08	0.09	0.09	2.32	2.27	2.45	2.35
CS8	0.06	0.07	0.05	0.06	0.16	0.25	0.27	0.23	0.10	0.19	0.22	0.17	2.54	3.47	3.79	3.27

Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.27	0.02	0.30	0.9	1.20	0	0.98
CS2	0.38	0.02	0.40	0.75	1.15	3	0.98
CS3	0.50	0.02	0.52	0.6	1.12	6	0.99
CS4	0.56	0.01	0.58	0.45	1.03	9	0.99
CS5	0.65	0.01	0.67	0.3	0.97	12	0.99
CS6	0.74	0.02	0.75	0.2	0.95	14	0.98
CS7	0.04	0.09	0.12	0.1	0.22	16	0.62
CS8	0.06	0.17	0.22	0	0.22	18	0.26

Cross Sections	Flow Rate: 1.12 cfs (2 pump)															
	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
	R1	R2	R3	Average	R1	R2	R3	Average	R1	R2	R3	Average	R1	R2	R3	Average
CS1	0.52	0.57	0.56	0.55	0.56	0.60	0.60	0.59	0.04	0.03	0.04	0.04	1.54	1.39	1.67	1.53
CS2	0.64	0.69	0.67	0.67	0.67	0.72	0.69	0.69	0.02	0.03	0.03	0.03	1.23	1.39	1.31	1.31
CS3	0.80	0.80	0.79	0.80	0.82	0.83	0.82	0.82	0.02	0.02	0.03	0.02	1.04	1.23	1.39	1.22
CS4	0.87	0.86	0.81	0.85	0.88	0.88	0.83	0.86	0.02	0.02	0.02	0.02	1.04	1.13	1.13	1.10
CS5	0.96	0.95	0.93	0.95	0.97	0.97	0.95	0.96	0.01	0.01	0.02	0.02	0.93	0.93	1.13	1.00
CS6	1.03	1.03	1.00	1.02	1.04	1.04	1.03	1.04	0.01	0.01	0.02	0.02	0.93	0.80	1.23	0.98
CS7	0.08	0.05	0.06	0.07	0.27	0.29	0.34	0.30	0.19	0.23	0.27	0.23	3.47	3.88	4.20	3.85
CS8	0.11	0.09	0.08	0.10	0.23	0.30	0.38	0.30	0.12	0.21	0.30	0.21	2.74	3.65	4.40	3.59

Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.55	0.04	0.59	0.9	1.49	0	0.98
CS2	0.67	0.03	0.69	0.75	1.44	3	0.98
CS3	0.80	0.02	0.82	0.6	1.42	6	0.98
CS4	0.85	0.02	0.86	0.45	1.31	9	0.99
CS5	0.95	0.02	0.96	0.3	1.26	12	0.99
CS6	1.02	0.02	1.04	0.2	1.24	14	0.99
CS7	0.07	0.23	0.30	0.1	0.40	16	0.42
CS8	0.10	0.20	0.30	0	0.30	18	0.32

Cross Sections	Flow Rate: 1.68 cfs (3 pump)															
	Water Depth (ft)				Water Depth + Velocity Head (ft)				Velocity Head (ft)				Velocity (ft/sec)			
	R1	R2	R3	Average	R1	R2	R3	Average	R1	R2	R3	Average	R1	R2	R3	Average
CS1	0.65	0.66	0.65	0.65	0.67	0.68	0.68	0.68	0.03	0.02	0.02	0.02	1.31	1.23	1.23	1.25
CS2	0.72	0.72	0.74	0.73	0.74	0.74	0.77	0.75	0.02	0.02	0.03	0.02	1.13	1.13	1.31	1.19
CS3	0.84	0.79	0.86	0.83	0.86	0.82	0.88	0.85	0.02	0.03	0.01	0.02	1.23	1.31	0.93	1.15
CS4	0.93	0.84	0.89	0.89	0.95	0.87	0.90	0.91	0.02	0.03	0.01	0.02	1.13	1.31	0.93	1.12
CS5	1.00	0.95	1.00	0.98	1.02	0.97	1.01	1.00	0.02	0.02	0.01	0.02	1.13	1.23	0.80	1.05
CS6	1.07	1.03	1.06	1.05	1.11	1.05	1.07	1.08	0.04	0.02	0.02	0.02	1.54	1.13	1.04	1.24
CS7	0.13	0.07	0.13	0.11	0.40	0.33	0.30	0.34	0.26	0.26	0.17	0.23	4.12	4.09	3.34	3.85
CS8	0.14	0.12	0.11	0.12	0.39	0.50	0.43	0.44	0.26	0.38	0.32	0.32	4.07	4.97	4.54	4.52

Cross Sections	Water Depth (ft)	$v^2/2g$ (ft)	s_f (ft)	Bottom Elevation (ft)	EGL (ft)	Distance from CS1	HGL:EGL
CS1	0.65	0.02	0.68	0.9	1.58	0	0.98
CS2	0.73	0.02	0.75	0.75	1.50	3	0.99
CS3	0.83	0.02	0.85	0.6	1.45	6	0.99
CS4	0.89	0.02	0.91	0.45	1.36	9	0.99
CS5	0.98	0.02	1.00	0.3	1.30	12	0.99
CS6	1.05	0.02	1.08	0.2	1.28	14	0.98
CS7	0.11	0.23	0.34	0.1	0.44	16	0.48
CS8	0.12	0.32	0.44	0	0.44	18	0.28