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Design and Operation of U-Turns at Diamond Interchanges in Texas

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16. Abstract
U-turn lanes are commonly provided at diamond interchanges to reduce delays for U-turning traffic and for the interchange as a whole; however, there are currently many unknowns related to their design, operation, and use. This project provides the Texas Department of Transportation (TxDOT) with implementable guidelines for designing and operating U-turn lanes at diamond interchanges. Researchers identified and investigated several factors affecting U-turn lane use, determined the performance and limitations of U-turn lanes under various geometric and operational conditions, and determined the anticipated effectiveness of proposed solutions to U-turn operational issues. Researchers then developed and structured guidelines for inclusion in the TxDOT *Roadway Design Manual* and other manuals addressing access management, design, and operations of U-turn facilities. This investigative effort included a cross-sectional safety analysis of existing U-turn configurations that provided valuable insight into factors contributing to U-turn safety. Along with the safety analysis, researchers developed a self-calculating spreadsheet tool that can be used to predict U-turn safety performance under various conditions.

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DISCLAIMER

This research was performed in cooperation with the Texas Department of Transportation (TxDOT) and the Federal Highway Administration (FHWA). The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the FHWA or TxDOT. This report does not constitute a standard, specification, or regulation. This report is not intended for construction, bidding, or permit purposes. The engineer in charge of the project was Jonathan M. Tydlacka, P.E., Texas Registered Professional Engineer #103801.

The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

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TABLE OF CONTENTS

Page

List of Figures ..... **ix**
List of Tables ..... **xii**
Chapter 1. Introduction **1**
Project Overview 1
Contents of This Report 2
Chapter 2. Factors Affecting U-Turn Lane Use and Potential Solutions to Operational Issues **5**
Introduction 5
Literature Review 5
Assessment of TxDOT Practices 13
Factors Affecting U-Turns and Solutions to Operational Issues 27
Chapter 3. Characteristics of U-Turn Lanes under Various Conditions **29**
Introduction 29
Site Selections 29
Site Characteristics 30
Traffic Conditions 32
Signal Control 36
Chapter 4. Operational Effectiveness of Solutions **37**
Introduction 37
Evaluation Conditions 37
Evaluation Methodology 38
Simulation Evaluation Results 42
Field Testing of Selected Solutions 75
Chapter 5. Safety Evaluation of U-Turn Design **89**
Introduction 89
Overview of Safety Assessment Tasks 89
Database Development 95
Cross-Sectional Qualitative Analysis 107
Influential Varaibles for Final Models 115
Overview of Statistical Analysis 117
Conclusions 125
Chapter 6. Development of U-Turn Guidelines **127**
Introduction 127
Guidelines for U-Turns 127
Recommended Revisions to TxDOT *Roadway Design Manual* 129
References **131**
Appendix A. Questions Document for State-of-the-Practice Review **133**
Background 135
Document Used to Guide TxDOT State-of-the-Practice Information Gathering 135
TxDOT Project 0-6894: Guidelines for Design and Operations of U-Turns 135
Appendix B. Volume Data from Study Sites **139**
Appendix C. Base Data from Simulation **165**

Appendix D. Simulation Results from the Countermeasures 191
Appendix E. Research Forest Volume Data for Signal Timing Analysis 237
Appendix F. Description of Variables Used in Safety Analysis 243
Appendix G. Summary of Crash Data for Operational Study Sites 247
Site #1 Information (Site ID: 6894_1) 247
Site #2 Information (Site ID: 6894_2) 248
Site #3 Information (Site ID: 6894_3) 249
Site #4 Information (Site ID: 6894_4) 250
Site #5 Information (Site ID: 6894_5) 251
Site #6 Information (Site ID: 6894_6) 252
Site #7 Information (Site ID: 6894_7) 253
Site #8 Information (Site ID: 6894_8) 254
Site #9 Information (Site ID: 6894_9) 255
Site #10 Information (Site ID: 6894_10) 256
Site #11 Information (Site ID: 6894_11) 257
Site #12 Information (Site ID: 6894_12) 258
Site #13 Information (Site ID: 6894_13) 259
Site #14 Information (Site ID: 6894_14) 260
Site #15 Information (Site ID: 6894_15) 261
Site #16 Information (Site ID: 6894_16) 262
Site #17 Information (Site ID: 6894_17) 263
Site #18 Information (Site ID: 6894_18) 264
Site #19 Information (Site ID: 6894_19) 265
Site #20 Information (Site ID: 6894_20) 266
Site #21 Information (Site ID: 6894_21)—Removed from Safety Analysis (Atypical Configuration) 268
Site #22 Information (Site ID: 6894_22) 268
Site #23 Information (Site ID: 6894_23) 269
Site #24 Information (Site ID: 6894_24) 270
Site #25 Information (Site ID: 6894_25) 271
Site #26 Information (Site ID: 6894_26) 272
Appendix H. Supplemental Statistical Analysis 275
KAB Proportional Models 275
Safety Effects for U-Turn Signalized Sites 276
Scaling Variables 278
Frequency Analysis 279

LIST OF FIGURES

Page

Figure 1. Frontage Road U-Turn Spacing Diagram (*2*). ... 6
Figure 2. Five Categories of U-Turn Yield Treatments in 0-4986 (*8*). ... 9
Figure 3. Number of Sites Selected in Each District. ... 30
Figure 4. Turning Movement Counts at Two Sample Sites. ... 33
Figure 5. NB to SB U-Turn Departure Gap Time Distribution at I-45 @ Research Forest. ... 36
Figure 6. Field Study Sites—Interchange Volume vs. Interchange Performance. ... 47
Figure 7. Relationship between Interchange and U-Turn Performance. ... 48
Figure 8. Relationship between Interchange Volume and U-Turn Performance. ... 49
Figure 9. Relationship between U-Turn Volume and U-Turn Performance. ... 50
Figure 10. Relationship between Frontage Left-Turn Volume and U-Turn Performance. ... 51
Figure 11. Relationship between Frontage Left-Turn Queue and U-Turn Performance. ... 52
Figure 12. Relationship between Conflicting Volumes and U-Turn Performance. ... 53
Figure 13. NB to SB U-Turn Delay Varied by Length of Acceleration Lane at I-45 @ Research Forest. ... 58
Figure 14. SB to NB U-Turn Delay Varied by Length of Acceleration Lane and Distance to Nearest Driveway at I-45 @ Research Forest. ... 59
Figure 15. Before and After Condition at I-45 @ Rayford Road. ... 61
Figure 16. SB to NB U-turn Queue Results with Varied Driveway 1 Volumes Based on AM Scenario at I-45 @ Rayford/Sawdust. ... 63
Figure 17. SB to NB U-turn Queue Results Varied by Sum of U-turn, Right Turn, and U-turn to Dr #1 Volume Based on PM Scenario at I-45 @ Rayford/Sawdust. ... 64
Figure 18. I-45 @ Research Forest Interchange. ... 67
Figure 19. U-Turn Delay Varied by U-Turn Demand and Left-Turn (LT) Volume in Lane 1 (Ln1) on Frontage Road at Research Forest. ... 69
Figure 20. Example MUTCD RTOR and U-Turn Traffic Yield Signs. ... 75
Figure 21. I-410 @ Ingram Interchange. ... 77
Figure 22. I-45 @ Research Forest Interchange, The Woodlands, Texas. ... 78
Figure 23. Example MUTCD Lane Addition Signing. ... 79
Figure 24. I-410 @ Ingram SB to NB U-turn, Lane Striping Treatment. ... 80
Figure 25. U-Turn Departure Installation of Pylons. ... 80
Figure 26. Map of Research Forest Subsystem Considered for Retiming. ... 82
Figure 27. PASSER V-09 Representation of the Modeled System. ... 83
Figure 28. Time-Space Diagrams for Existing AM and PM Peak Timings. ... 84
Figure 29. Time-Space Diagrams for AM- and PM Peak Optimized Timing Plans for Option 2 Optimization Runs. ... 86
Figure 30. GIS Intersection Points for Freeways and Arterials. ... 91
Figure 31. Locating an Interchange with a U-Turn. ... 92
Figure 32. Diamond Interchanges with U-Turns. ... 93
Figure 33. Stage 1 Sample Interchanges. ... 94
Figure 34. Diamond Interchange Sample. ... 95
Figure 35. Example Interchange with a U-Turn on only One Side. ... 97
Figure 36. Turnaround Configuration and Influential Site Characteristics. ... 98

Figure 37. Right-Turn Entrance Options. 100
Figure 38. Right-Turn Exit Options. 101
Figure 39. Distribution of Cross-Street Right-Turn Treatment Zone Entrance Options. 102
Figure 40. Distribution of Cross-Street Right-Turn Exit Options. 103
Figure 41. U-Turn Leg 1 and Leg 2 Interior Spacing. 104
Figure 42. Effective Length of the Highway Used to Define Buffers around Study Sites. 105
Figure 43. Collision Diagram for Site #7 before Condition (2010 Example). 109
Figure 44. Collision Diagram for Site #7 after Condition (2013 Example). 110
Figure 45. Relationship between Distance to Closest Driveway and Average Posted Speed. 116
Figure 46. CURE Plots for the Total Crash Model. 119
Figure 47. CURE Plots for KAB Crashes Predictive Model. 122
Figure 48. Model Fit for KAB Crashes Predictive Model. 123
Figure 49. Abilene District—I-20 @ SH 351 (AM Peak Hour). 139
Figure 50. Abilene District—I-20 @ SH 351 (PM Peak Hour). 139
Figure 51. Bryan District—SH 6 @ Boonville (AM Peak Hour). 140
Figure 52. Bryan District—SH 6 @ Boonville (PM Peak Hour). 140
Figure 53. Bryan District—SH 6 @ Briarcrest (AM Peak Hour). 141
Figure 54. Bryan District—SH 6 @ Briarcrest (PM Peak Hour). 141
Figure 55. Bryan District—SH 6 @ Rock Prairie (AM Peak Hour). 142
Figure 56. Bryan District—SH 6 @ Rock Prairie (PM Peak Hour). 142
Figure 57. Bryan District—SH 6 @ SH 40 (AM Peak Hour). 143
Figure 58. Bryan District—SH 6 @ SH 40 (PM Peak Hour). 143
Figure 59. Bryan District—SH 6 @ University (AM Peak Hour). 144
Figure 60. Bryan District—SH 6 @ University (PM Peak Hour). 144
Figure 61. Bryan District—US 290 @ SH 36 (AM Peak Hour). 145
Figure 62. Bryan District—US 290 @ SH 36 (PM Peak Hour). 145
Figure 63. Corpus Christi District—SH 358 @ Greenwood (No AM Count). 146
Figure 64. Corpus Christi District—SH 358 @ Greenwood (PM Peak Hour). 146
Figure 65. Ft. Worth District—I-35W @ Alsbury (AM Peak Hour). 147
Figure 66. Ft. Worth District—I-35W @ Alsbury (PM Peak Hour). 147
Figure 67. Ft. Worth District—I-35W @ FM 1187 (AM Peak Hour). 148
Figure 68. Ft. Worth District—I-35W @ FM 1187 (PM Peak Hour). 148
Figure 69. Ft. Worth District—I-20 @ McCart (AM Peak Hour). 149
Figure 70. Ft. Worth District—I-20 @ McCart (PM Peak Hour). 149
Figure 71. Ft. Worth District—I-20 @ Hulen (AM Peak Hour). 150
Figure 72. Ft. Worth District—I-20 @ Hulen (PM Peak Hour). 150
Figure 73. Houston District—I-10 @ Bunker Hill Rd. (AM Peak Hour). 151
Figure 74. Houston District—I-10 @ Bunker Hill Rd. (PM Peak Hour). 151
Figure 75. Houston District—I-10 @ Gessner Rd. (AM Peak Hour). 152
Figure 76. Houston District—I-10 @ Gessner Rd. (PM Peak Hour). 152
Figure 77. Houston District—I-45 @ Rayford Rd/Sawdust Rd. (AM Peak Hour). 153
Figure 78. Houston District—I-45 @ Rayford Rd/Sawdust Rd. (PM Peak Hour). 153
Figure 79. Houston District—I-45 @ Research Forest Dr. (AM Peak Hour). 154
Figure 80. Houston District—I-45 @ Research Forest Dr. (PM Peak Hour). 154
Figure 81. Laredo District—I35 @ Mann (AM Peak Hour). 155

Figure 82. Laredo District—I35 @ Mann (PM Peak Hour). 155
Figure 83. Pharr District—I-2 @ FM 2220 (AM Peak Hour). 156
Figure 84. Pharr District—I-2 @ FM 2220 (PM Peak Hour). 156
Figure 85. Pharr District—I-2 @ SH 494 (AM Peak Hour). 157
Figure 86. Pharr District—I-2 @ SH 494 (PM Peak Hour). 157
Figure 87. San Angelo District—SH 306 @ US 67 (AM Peak Hour). 158
Figure 88. San Angelo District—SH 306 @ US 67 (PM Peak Hour). 158
Figure 89. San Antonio District—I-410 @ Callaghan (AM Peak Hour). 159
Figure 90. San Antonio District—I-410 @ Callaghan (PM Peak Hour). 159
Figure 91. San Antonio District—I-410 @ Ingram (AM Peak Hour). 160
Figure 92. San Antonio District—I-410 @ Ingram (PM Peak Hour). 160
Figure 93. Waco District—I-35 @ FM 286 (AM Peak Hour). 161
Figure 94. Waco District—I-35 @ FM 286 (PM Peak Hour). 161
Figure 95. Wichita Falls District—US 82 @ Kemp (AM Peak Hour). 162
Figure 96. Wichita Falls District—US 82 @ Kemp (PM Peak Hour). 162
Figure 97. Wichita Falls District—US 82 @ Lawrence (AM Peak Hour). 163
Figure 98. Wichita Falls District—US 82 @ Lawrence (PM Peak Hour). 163
Figure 99. Cross-Street AADT Compared to the Number of Crashes per Year 280
Figure 100. Raw Crash Data Plotted Against AADT Values. 281
Figure 101. CURE Plots for Second Refinement of the Total Crash Model. 286
Figure 102. Model Fit for the Total Crashes Model (Second Refinement—Site-Specific versus Total Site Population). 287

LIST OF TABLES

Page

Table 1. Frontage Road Connection Spacing Criteria (*2*).. 6
Table 2. General Information from TxDOT Districts... 14
Table 3. Additional Comments from TxDOT District Respondents. 17
Table 4. U-Turn Lanes Added in Last Five Years... 18
Table 5. Any Locations with Known Recurring Congestion Issues. 20
Table 6. Locations with Temporary Issues at U-Turn Lanes.. 21
Table 7. U-Turn Lanes Redesigned or Retrofitted to Improve Operations or Safety. 22
Table 8. Locations Currently Experiencing Issues. 23
Table 9. Issues with U-Turns over Freeway Underpasses. 23
Table 10. Any Box Diamonds Where U-Turn Designed Differently. 24
Table 11. Any Design Changes due to Proximity of Other Interchange/Intersection Forms. 24
Table 12. Potential Study Locations. 25
Table 13. OD Counts at I-10 @ Gessner Rd. for WB to EB U-Turn Departure Side. 34
Table 14. Lane Distribution at I-10 @ Gessner Rd. for EB to WB U-Turn Departure Side. 35
Table 15. SB to NB U-Turn Departure Delay and Stops at I-410 @ Ingram. 35
Table 16. Potential Countermeasures to Improve Operations at Sites with U-Turns. 39
Table 17. Example Calibration Targets. 41
Table 18. VISSIM Results Comparison—Set 1. 43
Table 19. VISSIM Results Comparisons—Set 2. 44
Table 20. VISSIM Results Comparisons—Set 3. 45
Table 21. Descriptive Statistics for the 25 U-Turn Study Sites. 46
Table 22. List of Eight Sites Chosen for Detailed Modeling... 54
Table 23. VISSIM Evaluation Results for Countermeasure of Adding a U-Turn Lane to Westbound at I-20 @ McCart AM Peak Hour 57
Table 24. VISSIM Evaluation Results for Countermeasures of Separation from Conflicted Traffic at Southbound U-Turn Departure End at I-45 @ Rayford Rd. 62
Table 25. VISSIM Evaluation Results for Countermeasure of Interior Left-Turn Operations at SH 6 @ Briarcrest Dr during AM Peak Hour 65
Table 26. VISSIM Countermeasures Results—Direct Vehicles to Alternate Receiving Lanes Performance Measures of Southbound U-turn Traffic at I-45 @ Research Forest Dr. 68
Table 27. VISSIM Countermeasures Results—No RTOR from Cross-Street Performance Measures at Houston District I-45 @ Research Forest Dr.. 70
Table 28. VISSIM Countermeasures Results—Eastbound Driveway Closure to U-Turn Performance Measures at I-10 @ Gessner Rd. 73
Table 29. Field Collected U-Turn Delay Data before and after Yield Sign Removal. 81
Table 30. Performance Measures for the Two Existing Timing Plans. 85
Table 31. Comparison of Performance Measures for Existing and Optimized Timings. 86
Table 32. Candidate Roadway Types in RHiNO. 90
Table 33. Number of Lanes on the Frontage Roads. 98
Table 34. Right-Turn Treatment Zone Entrances. 99

Table 35. Right-Turn Treatment Zone Exit Configurations. 102
Table 36. Characteristics for U-Turn Leg 1 and Leg 2. 104
Table 37. Number of Filtered Crashes Relative to Total Number of Annual Crashes. 106
Table 38. Crash Severity Summary at Operational (Task 4) Study Sites. 112
Table 39. Percent of Left-Turn Crashes at Operational Analysis Sites with and without U-Turns. 113
Table 40. Percent of Left-Turn Crashes Initiating on Frontage Road Contrasted to Other Left Turns. 114
Table 41. Summary of Data Characteristics. 116
Table 42. Simplified Predictive Model for Total Crashes (Signalized Sites No Yearly Factor). 118
Table 43. Predictive Models for KAB Crashes (Signalized Intersections). 121
Table 44. Influence of Site or Traffic Characteristics on Crashes. 125
Table 45. VISSIM Results Summary—Abilene District—I-20 @ SH 351. 165
Table 46. VISSIM Results Summary—Bryan District—SH 6 @ Boonville. 166
Table 47. VISSIM Results Summary—Bryan District—SH 6 @ Briarcrest. 167
Table 48. VISSIM Results Summary—Bryan District—SH 6 @ Rock Prairie. 168
Table 49. VISSIM Results Summary—Bryan District—SH 6 @ SH 40. 169
Table 50. VISSIM Results Summary—Bryan District—SH 6 @ University. 170
Table 51. VISSIM Results Summary—Bryan District—US 290 @ SH 36. 171
Table 52. VISSIM Results Summary—Corpus Christi District—SH 358 @ Greenwood. 172
Table 53. VISSIM Results Summary—Ft. Worth District—I-35W @ Alsbury. 173
Table 54. VISSIM Results Summary—Ft. Worth District—I-35W @ FM 1187. 174
Table 55. VISSIM Results Summary—Ft. Worth District—I-20 @ Hulen. 175
Table 56. VISSIM Results Summary—Ft. Worth District—I-20 @ McCart. 176
Table 57. VISSIM Results Summary—Houston District—I-10 @ Bunker Hill. 177
Table 58. VISSIM Results Summary—Houston District—I-10 @ Gessner. 178
Table 59. VISSIM Results Summary—Houston District—I-45 @ Rayford/ Sawdust. 179
Table 60. VISSIM Results Summary—Houston District—I-45 @ Research Forest. 180
Table 61. VISSIM Results Summary—Laredo District—I 35 @ Mann. 181
Table 62. VISSIM Results Summary—Pharr District—I-2 @ FM 2220. 182
Table 63. VISSIM Results Summary—Pharr District—I-2 @ SH 494. 183
Table 64. VISSIM Results Summary—San Angelo District—SH 306 @ US 67. 184
Table 65. VISSIM Results Summary—San Antonio District—I-410 @ Callaghan. 185
Table 66. VISSIM Results Summary—San Antonio District—I-410 @ Ingram. 186
Table 67. VISSIM Results Summary—Waco District—I-35 @ FM 286. 187
Table 68. VISSIM Results Summary—Wichita Falls District—US 82 @ Kemp. 188
Table 69. VISSIM Results Summary—Wichita Falls District—US 82 @ Lawrence. 189
Table 70. VISSIM Countermeasures Results—Extending Left-Turn and U-Turn Bays Performance Measures of AM Peak Hour at I-10 @ Gessner Rd. 191
Table 71. VISSIM Countermeasures Results– Extending Left-Turn and U-Turn Bays Performance Measures of PM Peak Hour at I-10 @ Gessner Rd. 192
Table 72. VISSIM Countermeasures Results Summary: I-20 @ Hulen AM Peak Hour—Extend U-Turn Bay. 193
Table 73. VISSIM Countermeasures Results Summary: I-20 @ Hulen PM Peak Hour—Extend U-Turn Bay. 194

Table 74. VISSIM Countermeasures Results Summary: I-410 @ Ingram AM Peak Hour—Extend U-Turn Bay.......... 195
Table 75. VISSIM Countermeasures Results Summary: I-410 @ Ingram PM Peak Hour—Extend U-Turn Bay.......... 196
Table 76. VISSIM Countermeasures Results Summary: I-410 @ Ingram AM Peak Hour—Extend U-Turn Bay (Increased Travel Demand).......... 197
Table 77. VISSIM Countermeasures Results Summary: I-410 @ Ingram PM Peak Hour—Extend U-Turn Bay (Increased Travel Demand).......... 198
Table 78. VISSIM Countermeasures Results Summary: I-410 @ Ingram AM Peak Hour—Dual U-Turn Lane.......... 199
Table 79. VISSIM Countermeasures Results Summary: I-410 @ Ingram PM Peak Hour—Dual U-Turn Lane.......... 200
Table 80. U-Turn Departure Side Results: I-410 @ Ingram Base Scenario and Dual U-Turn Lane Improvement. 200
Table 81. VISSIM Countermeasures Results Summary: I-20 @ McCart AM Peak Hour—Add U-Turn (Westbound).......... 201
Table 82. VISSIM Countermeasures Results Summary: I-20 @ McCart PM Peak Hour—Add U-Turn (Westbound).......... 202
Table 83. VISSIM Countermeasures Results– Adding Northbound U-Turn Lane: Performance Measures of AM Peak Hour at SH 6 @ Briarcrest Dr. 203
Table 84. VISSIM Countermeasures Results—Adding U-Turn Lanes for Departure: Performance Measures at I-10 @ Gessner Rd. 204
Table 85. VISSIM Countermeasures Results—Adding U-Turn Lanes for Departure: Performance Measures at I-10 @ Bunker Hill Rd.......... 205
Table 86. VISSIM Countermeasures Results—Separation from Conflicted Traffic: Performance Measures of Southbound U-Turn Departure End at I-45 @ Rayford Rd.......... 206
Table 87. VISSIM Countermeasures Results—Separation from Conflicted Traffic: Performance Measures of Westbound Right Turn at I-45 @ Rayford Rd.......... 206
Table 88. VISSIM Countermeasures Results—Separation from Conflicted Traffic: Performance Measures of Northbound Through at I-45 @ Rayford Rd. 207
Table 89. VISSIM Countermeasures Results—Separation from Conflicted Traffic: Performance Measures of Eastbound Left Turn at I-45 @ Rayford Rd. 207
Table 90. VISSIM Countermeasures Results—Interior Left-Turn Operations: Performance Measures of AM Peak Hour at SH 6 @ Briarcrest Dr. 208
Table 91. VISSIM Countermeasures Results Summary: I-410 @ Ingram AM Peak Hour—Signalized Control U-Turn. 209
Table 92. VISSIM Countermeasures Results Summary: I-410 @ Ingram PM Peak Hour—Signalized Control U-Turn. 210
Table 93. VISSIM Countermeasures Results Summary: I-410 @ Ingram AM Peak Hour—Added Lane Sign for U-Turn Lane.......... 211
Table 94. VISSIM Countermeasures Results Summary: I-410 @ Ingram PM Peak Hour—Added Lane Sign for U-Turn Lane.......... 212
Table 95. U-Turn Departure Side Results: I-410 @ Ingram Base Scenario and Added Lane Sign for U-Turn Lane Improvement.......... 213

Table 96. VISSIM Countermeasures Results—Direct Vehicles to Alternate Receiving Lanes Performance Measures of Southbound U-Turn Traffic at I-45 @ Research Forest Dr. ... 213
Table 97. VISSIM Countermeasures Results—No RTOR from Cross-Street Performance Measures at Houston District I-45 @ Research Forest Dr. ... 214
Table 98. VISSIM Countermeasures Results Summary: I-20 @ McCart AM Peak Hour—No RTOR from Cross Street (Southbound Only). ... 215
Table 99. VISSIM Countermeasures Results Summary: I-20 @ McCart PM Peak Hour—No RTOR from Cross Street (Southbound Only). ... 216
Table 100. U-Turn Departure Side Results: I-20 @ McCart Base Scenario and No RTOR from Cross-Street Improvement. ... 217
Table 101. VISSIM Countermeasures Results Summary: I-20 @ Hulen AM Peak Hour—No RTOR from Cross Street. ... 218
Table 102. VISSIM Countermeasures Results Summary: I-20 @ Hulen PM Peak Hour—No RTOR from Cross Street. ... 219
Table 103. U-Turn Departure Side Results: I-20 @ Hulen Base Scenario and No RTOR from Cross-Street Improvement. ... 220
Table 104. VISSIM Countermeasures Results Summary: I-410 @ Ingram AM Peak Hour—No RTOR from Cross Street. ... 221
Table 105. VISSIM Countermeasures Results Summary: I-410 @ Ingram PM Peak Hour—No RTOR from Cross Street. ... 222
Table 106. U-Turn Departure Side Results: I-410 @ Ingram Base Scenario and No RTOR from Cross-Street Improvement. ... 223
Table 107. VISSIM Countermeasures Results—No RTOR Except from Right Lane Sign Performance Measures at I-10 @ Gessner Rd. ... 224
Table 108. VISSIM Countermeasures Results—Eastbound Driveway Closure to U-Turn Performance Measures at I-10 @ Gessner Rd. ... 225
Table 109. VISSIM Countermeasures Results—No RTOR from Cross-Street Measure of Effectiveness of at I-45 @ Research Forest Dr. ... 226
Table 110. VISSIM Countermeasures Results Summary: I-20 @ McCart AM Peak Hour—Driveway Closure (Westbound First Driveway). ... 227
Table 111. VISSIM Countermeasures Results Summary: I-20 @ McCart PM Peak Hour—Driveway Closure (Westbound First Driveway). ... 228
Table 112. U-Turn Departure Side Results: I-20 @ McCart Base Scenario and Driveway Closure Improvement. ... 229
Table 113. VISSIM Countermeasures Results Summary: I-20 @ Hulen AM Peak Hour—Driveway Closure (Westbound Only). ... 230
Table 114. VISSIM Countermeasures Results Summary: I-20 @ Hulen PM Peak Hour—Driveway Closure (Westbound Only). ... 232
Table 115. U-Turn Departure Side Results: I-20 @ Hulen Base Scenario and Driveway Closure Improvement. ... 234
Table 116. VISSIM Countermeasures Results Summary: I-20 @ Hulen AM Peak Hour—RTOR Yield to U-Turn Traffic. ... 235
Table 117. VISSIM Countermeasures Results Summary: I-20 @ Hulen PM Peak Hour—RTOR Yield to U-Turn Traffic. ... 236

Table 118. U-Turn Departure Side Results: I-20 @ Hulen Base Scenario and RTOR Yield to U-Turn Traffic Improvement. 236
Table 119. Data from Six Pines Intersection. 237
Table 120. Data from Holly Hill Intersection. 238
Table 121. Data from Pinecroft Intersection. 239
Table 122. Data from I-45—Northbound Frontage Road Intersection. 240
Table 123. Data from I-45—Southbound Frontage Road Intersection. 241
Table 124. Data from David Memorial Intersection. 242
Table 125. Variable Descriptions. 243
Table 126. Site #1—Summary of Site Conditions. 247
Table 127. Site #1—Summary of Crash Severity. 247
Table 128. Site #1—Summary of Left-Turn Crashes. 247
Table 129. Site #1—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 248
Table 130. Site #2—Summary of Site Conditions. 248
Table 131. Site #2—Summary of Crash Severity. 248
Table 132. Site #2—Summary of Left-Turn Crashes. 248
Table 133. Site #2—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 249
Table 134. Site #3—Summary of Site Conditions. 249
Table 135. Site #3—Summary of Crash Severity. 249
Table 136. Site #3—Summary of Left-Turn Crashes. 249
Table 137. Site #3—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 250
Table 138. Site #4—Summary of Site Conditions. 250
Table 139. Site #4—Summary of Crash Severity. 250
Table 140. Site #4—Summary of Left-Turn Crashes. 250
Table 141. Site #4—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 251
Table 142. Site #5—Summary of Site Conditions. 251
Table 143. Site #5—Summary of Crash Severity. 251
Table 144. Site #5—Summary of Left-Turn Crashes. 252
Table 145. Site #5—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 252
Table 146. Site #6—Summary of Site Conditions. 252
Table 147. Site #6—Summary of Crash Severity. 252
Table 148. Site #6—Summary of Left-Turn Crashes. 253
Table 149. Site #6—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 253
Table 150. Site #7—Summary of Site Conditions. 253
Table 151. Site #7—Summary of Crash Severity. 253
Table 152. Site #7—Summary of Left-Turn Crashes. 254
Table 153. Site #7—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 254
Table 154. Site #8—Summary of Site Conditions. 254
Table 155. Site #8—Summary of Crash Severity. 254

Table 156. Site #8—Summary of Left-Turn Crashes. 255
Table 157. Site #8—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 255
Table 158. Site #9—Summary of Site Conditions. 255
Table 159. Site #9—Summary of Crash Severity. 255
Table 160. Site #9—Summary of Left-Turn Crashes. 256
Table 161. Site #9—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 256
Table 162. Site #10—Summary of Site Conditions. 256
Table 163. Site #10—Summary of Crash Severity. 256
Table 164. Site #10—Summary of Left-Turn Crashes. 257
Table 165. Site #10—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 257
Table 166. Site #11—Summary of Site Conditions. 257
Table 167. Site #11—Summary of Crash Severity. 258
Table 168. Site #11—Summary of Left-Turn Crashes. 258
Table 169. Site #11—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 258
Table 170. Site #12—Summary of Site Conditions. 258
Table 171. Site #12—Summary of Crash Severity. 259
Table 172. Site #12—Summary of Left-Turn Crashes. 259
Table 173. Site #12—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 259
Table 174. Site #13—Summary of Site Conditions. 259
Table 175. Site #13—Summary of Crash Severity. 260
Table 176. Site #13—Summary of Left-Turn Crashes. 260
Table 177. Site #13—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 260
Table 178. Site #14—Summary of Site Conditions. 260
Table 179. Site #14—Summary of Crash Severity. 261
Table 180. Site #14—Summary of Left-Turn Crashes. 261
Table 181. Site #14—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 261
Table 182. Site #15—Summary of Site Conditions. 261
Table 183. Site #15—Summary of Crash Severity. 262
Table 184. Site #15—Summary of Left-Turn Crashes. 262
Table 185. Site #15—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 262
Table 186. Site #16—Summary of Site Conditions. 262
Table 187. Site #16—Summary of Crash Severity. 263
Table 188. Site #16—Summary of Left-Turn Crashes. 263
Table 189. Site #16—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 263
Table 190. Site #17—Summary of Site Conditions. 263
Table 191. Site #17—Summary of Crash Severity. 264
Table 192. Site #17—Summary of Left-Turn Crashes. 264

Table 193. Site #17—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 264
Table 194. Site #18—Summary of Site Conditions. 264
Table 195. Site #18—Summary of Crash Severity. 265
Table 196. Site #18—Summary of Left-Turn Crashes. 265
Table 197. Site #18—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 265
Table 198. Site #19—Summary of Site Conditions. 265
Table 199. Site #19—Summary of Crash Severity. 266
Table 200. Site #19—Summary of Left-Turn Crashes. 266
Table 201. Site #19—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 266
Table 202. Site #20—Summary of Site Conditions. 266
Table 203. Site #20—Summary of Crash Severity. 267
Table 204. Site #20—Summary of Left-Turn Crashes. 267
Table 205. Site #20—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 267
Table 206. Site #21—Summary of Site Conditions. 268
Table 207. Site #21—Summary of Crash Severity. 268
Table 208. Site #22—Summary of Site Conditions. 268
Table 209. Site #22—Summary of Crash Severity. 269
Table 210. Site #22—Summary of Left-Turn Crashes. 269
Table 211. Site #22—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 269
Table 212. Site #23—Summary of Site Conditions. 269
Table 213. Site #23—Summary of Crash Severity. 270
Table 214. Site #23—Summary of Left-Turn Crashes. 270
Table 215. Site #23—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 270
Table 216. Site #24—Summary of Site Conditions. 270
Table 217. Site #24—Summary of Crash Severity. 271
Table 218. Site #24—Summary of Left-Turn Crashes. 271
Table 219. Site #24—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 271
Table 220. Site #25—Summary of Site Conditions. 271
Table 221. Site #25—Summary of Crash Severity. 272
Table 222. Site #25—Summary of Left-Turn Crashes. 272
Table 223. Site #25—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 272
Table 224. Site #26—Summary of Site Conditions. 272
Table 225. Site #26—Summary of Crash Severity. 273
Table 226. Site #26—Summary of Left-Turn Crashes. 273
Table 227. Site #26—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated. 273
Table 228. Proportion of KAB Left-Turn Crashes of Frontage Road Left Turns (All Sites). 275

Table 229. Proportion of KAB Crashes among All Intersection Crashes (All Sites). 276
Table 230. KAB Left-Turn Crashes of Frontage Road Left Turns (U-Turn, Signalized Sites). 277
Table 231. Standard Deviations Needed to Derive the Effects of Scaled Values. 279
Table 232. Predictive Model for Total Crashes (Signalized Sites with Yearly Factor). 282
Table 233. Predictive Model for Total Crashes (Signalized Sites but without a Yearly Factor). 283
Table 234. Predictive Model for Total Crashes (Signalized Sites with Yearly Factor). 285

CHAPTER 1. INTRODUCTION

PROJECT OVERVIEW

Frontage road (FR) U-turn movements support diamond interchange operations and local circulation within the corridor. Signal timing plans at most closely spaced interchanges in Texas are of the TTI four-phase variety, which progresses all traffic movements through the interchange—without stopping within the interchange interior—with the exception of the tail end of the frontage road left-turn movement that turns left again at the other frontage road (i.e., a U-turn maneuver through the signalized interchange).

In providing motorists the opportunity to cross a freeway without passing through the signalized portion of the interchange, overall interchange delay and trip times are reduced for corridor users locally circulating within the frontage road portions of the freeway corridor and for motorists on one side of the freeway who wish to continue their trip in the opposing direction. Since both interchange delay and trip times can be high—especially during peak traffic hours in urbanized areas—the delay savings brought about by U-turn lanes can have a substantial impact on improving operations.

U-turn lanes at interchanges can have a positive effect on corridor operations during incidents as well as during everyday operations. Because freeway interchanges can easily become overcongested by excessive re-routed traffic demand during construction work-zone lane closures on the freeway mainlanes or under freeway incident conditions, U-turn lanes can serve as relief routes that allow traffic to reroute without the additional delay incurred at a congested interchange. Additionally, U-turn lanes are often featured as part of the route serving re-directed traffic within traffic control plans during freeway mainlane and frontage road reconstruction.

U-turn lanes are commonly provided at diamond interchanges to reduce delay for U-turning traffic and for the interchange as a whole; however, there are currently many unknowns related to their design, operation, and use. While serving in any of the roles above, U-turn lanes have the potential to become overcongested, either due to demand far in excess of typical daily traffic volumes or due to external factors that limit the free flow of traffic from one frontage road to the opposing-direction frontage road.

Recently, some design-build contractors have inquired about the necessity of including U-turn lanes in their design-build contracts. Their notion is that the cost of adding U-turn lanes to a diamond interchange design (specifically the design of the bridge spans) is quite large compared to the perceived benefit of the U-turn lanes, which has not been easily quantified using previous research. However, the value of U-turn lanes and their relative value (compared to construction costs) can be determined from the results of this research project. Knowing the delay-reducing capabilities of U-turns as well as what treatments can be made to maximize their efficiency will greatly aid decision-makers in properly evaluating not only the design and operation of U-turn lanes, but also their necessity in diamond interchange design and their actual value.

Project 0-6894 was tasked with identifying and investigating factors affecting U-turn lane use, determining the capacity of U-turn lanes under various geometric and operational conditions, and determining the anticipated effectiveness of proposed solutions to U-turn operational issues. This project also provides a cross-sectional safety analysis of existing U-turn configurations at diamond interchanges. The results of this research can be used to design or improve the effectiveness of U-turn lanes, resulting in more efficient traffic flow at diamond interchanges.

Project 0-6894 provides TxDOT with implementable guidelines for designing and operating U-turn lanes at diamond interchanges. These guidelines are formatted for inclusion in the *Roadway Design Manual* and other manuals dealing with access management, design, and operations of facilities. A key product of the safety analysis is a self-calculating spreadsheet tool that can be used to predict U-turn safety performance under various conditions.

CONTENTS OF THIS REPORT

This report describes the activities taken by researchers to complete the tasks prescribed as part of Project 0-6894. The report consists of six chapters and eight appendices, as follows:

- Chapter 1 contains this introductory chapter.
- Chapter 2 contains descriptions of the activities performed to determine the factors affecting U-turn lane use and potential solutions to operational issues with U-turns at diamond interchanges. The activities included a literature review, state-of-the-practice assessment, and creation of an initial list of factors and potential solutions.
- Chapter 3 contains descriptions of the processes used for study site selection and data collection for the study sites.
- Chapter 4 summarizes the activities used in creating baseline VISSIM models of the study sites, creating more detailed models, and modeling many different countermeasure solutions to design and operational issues with U-turns. Chapter 4 also summarizes the results and findings of these modeling efforts. This chapter also contains the results for the two field site evaluations.
- Chapter 5 summarizes the activities performed in creating a safety evaluation of U-turns and developing a statistical equation for producing a predictive safety model spreadsheet.
- Chapter 6 contains the proposed guidelines for implementing U-turn lanes, along with supporting information from the research.
- Appendix A contains the questions document used during the state of the practice, as described in Chapter 2.

- Appendix B contains the volume count data from all study sites, as described in Chapter 3.
- Appendix C contains the base model simulation results for all study sites, as described in Chapter 4.
- Appendix D contains the simulation results from the evaluation of countermeasures, as described in Chapter 4.
- Appendix E contains the traffic volume data from the signal timing field evaluation corridor of Research Forest, including the I-45 @ Research Forest field study site, as described in Chapter 4.
- Appendix F contains a description of the variables used in the safety analysis, as described in Chapter 5.
- Appendix G summarizes the crash data for all of the operational study sites, as described in Chapter 5.
- Appendix H summarizes the safety supplemental statistical analysis, as described in Chapter 5.

CHAPTER 2. FACTORS AFFECTING U-TURN LANE USE AND POTENTIAL SOLUTIONS TO OPERATIONAL ISSUES

INTRODUCTION

This chapter describes the activities performed in Task 2 of this project. The objectives of this task were:

- To perform a literature review to identify potential factors and solutions and to determine their relevance to the research project.
- To perform a fact-gathering effort by contacting a representative from each TxDOT district.
- To identify the factors affecting U-turn lane use and potential solutions to operational issues.

LITERATURE REVIEW

Existing Guidance on Designing and Operating U-Turn Lanes at Diamond Interchanges

U-turn lanes (sometimes called turnaround lanes) at diamond interchanges are fairly common in Texas but are infrequently found elsewhere. These lanes provide an opportunity for drivers on a one-way frontage road to connect directly to the one-way frontage road running in the opposite direction on the other side of a freeway without having to pass through traffic signals at the diamond interchange, reducing user delay and frustration.

In addition to reducing delay for U-turning traffic, U-turn lanes free up capacity for all other traffic passing through signalized approaches of the interchange. However, as traffic volumes increase at the interchange, overall interchange delay increases, and U-turn lanes can experience excessive delay and queuing, especially in areas of heavy development along frontage roads. Recently, some design-build contractors have begun recommending designs that eliminate U-turn lanes to reduce construction cost.

The *Roadway Design Manual* (RDM) contains little guidance for the specification and design of U-turns (*1*). Users need to refer to more general criteria designated for frontage road and median turn lanes for design criteria. The *Access Management Manual* gives the minimum connection spacing criteria for frontage roads and explains how the access connection spacing in the proximity of frontage road U-turn lanes will be measured (*2*). Guidance on passing lane length and spacing is based primarily on the average daily traffic (ADT) of the roadway, as shown in Figure 2-3 and Table 2-1 of the *Access Management Manual*, reproduced here as Figure 1 and Table 1.

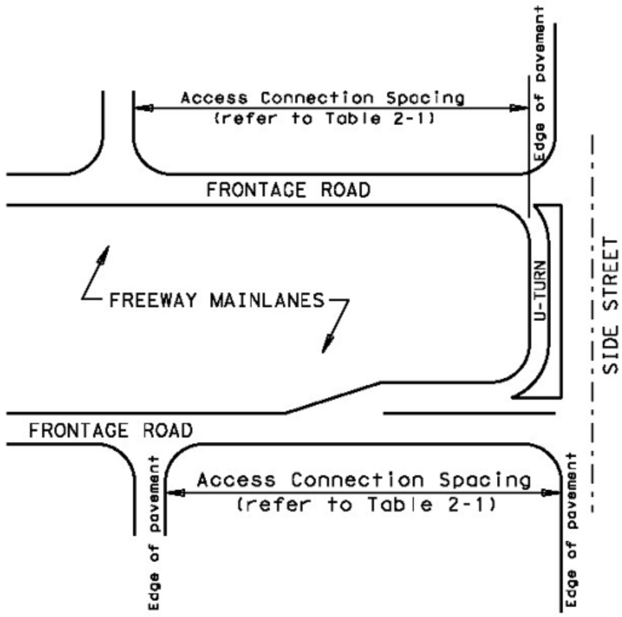


Figure 1. Frontage Road U-Turn Spacing Diagram (*2*).

Table 1. Frontage Road Connection Spacing Criteria (*2*).

Minimum Connection Spacing Criteria for Frontage Roads [1, 2]		
	Minimum Connection Spacing (ft)	
Posted Speed (mph)	**One-Way Frontage Roads**	**Two-Way Frontage Roads**
≤ 30	200	200
35	250	300
40	305	360
45	360	435
≥ 50	425	510

[1] Distances are for passenger cars on level grade. These distances may be adjusted for downgrades and/or significant truck traffic. Where present or projected traffic operations indicate specific needs, consideration may be given to intersection sight distance and operational gap acceptance measurement adjustments.
[2] When these values are not attainable, refer to the variance process as described in Chapter 2, Section 5.

In general, traffic engineers lack design and operational guidelines regarding when U-turns are needed, where they should be placed, how they should be designed, what their delay-reducing capabilities are, and what the safety benefits are. On the surface, it appears to be a district-driven policy based on engineering judgment as to when U-turns are constructed on a facility.

Benefits of U-Turn Lanes

In the 1960s, TTI researchers investigated the effects of the U-turn movement on delay and intersection capacity, particularly at diamond interchanges. Wilson et al. conducted a study of five sites in Houston and in the Fort Worth area and found that U-turn traffic at interchanges that contain no separate U-turn lanes is a source of delay to the system (*3*). Not only are the U-turn vehicles delayed, but they can potentially affect the vehicles on all of the other approaches as well. Researchers concluded that U-turn traffic should be adequately accommodated through the design of special U-turn lanes at all diamond-type interchanges.

In recent years, a study by Liu et al. estimated the effects of U-turning vehicles on signalized intersection capacity by using data collected at three signalized intersections in Tampa Bay, Florida (*4*). They found that U-turning vehicles adversely affect the capacities of signalized intersections, and the effect increases with the increase in the percentage of U-turning vehicles in the left-turn lane. When the capacity of a signalized intersection is estimated, it is essential to account for the capacity reduction due to the presence of U-turning vehicles, especially when the percentage of U-turning vehicles on the approach is relatively high (> 40 percent). The effect can be quantified by applying the adjustment factors developed in this study by Liu et al.

Carter et al. studied operational and safety effects of U-turns at signalized intersections using regression analysis (*5*). Their analysis suggests a 1.8 percent saturation flow-rate loss in the left-turn lane for every 10 percent increase in U-turn percentage and an additional 1.5 percent loss for every 10 percent increase in U-turns if the U-turning movement was opposed by protected right-turn overlap from the cross street. The safety analysis of the study also found that while most of the study sites did not have any collisions involving U-turns in the 3-year study period, sites with double left-turn lanes, protected right-turn overlap, or high left-turn and conflicting right-turn traffic volumes were found to have a significantly greater number of U-turn related collisions.

Rodriguez et al. also investigated the potential fuel savings that can be realized from the provision of U-turn lanes at diamond interchanges (*6*). Researchers conducted the study by using the vehicles emission simulation module of the software TEXAS (Traffic Experimental Analytical Simulation). Six diamond interchanges from Austin and El Paso, Texas (with and without U-turn lanes), were selected as case studies for their research. The results indicated that the amount of fuel consumed by U-turning vehicles using the U-turn lane is significantly less (60 to 80 percent less) than that used by turning vehicles going through the intersection of a diamond interchange.

Safety of U-Turns

Very little research exists for the safety performance of U-turn lanes at diamond interchanges in Texas. The literature search revealed that while it has been the intuitive perception that U-turn lanes in the diamond interchange provide safer conditions by allowing vehicles to bypass the two traffic signals at the intersection without mixing with the other traffic movements, not much research has been conducted directly on the safety of U-turns at signalized diamond interchanges. Extensive research has been devoted to the safety of U-turns at unsignalized intersections, such as median openings. The National Cooperative Highway Research Program (NCHRP) Project G17-21 documented a thorough review of the safety and operational effects of various median opening designs (*7*). Researchers then compared the median opening crash and conflict rates and found that crashes related to U-turn and left-turn maneuvers (which do not distinguish clearly between each other at unsignalized median openings) occur very infrequently, and there is no indication that U-turns at unsignalized median openings constitute a major safety concern. The study estimated the average accident rates per median opening movement (U-turn plus left-turn maneuvers) for specific median opening types in both urban and rural arterial corridors. No satisfactory regression relationships relating median opening accident frequency to the volume of U-turn and left-turn maneuvers through the median opening could be developed.

Previous Research in Texas

Previous TTI work (Research Project 2-8-61-24) investigated the U-turn movement of frontage road traffic to determine its effect on the delay produced at signalized intersections and to determine minimum design criteria required to facilitate this movement at freeway interchanges (*3*). Design features that are considered important to the proper functioning of the interchange in relation to the U-turn movement have been studied. Those design features include side slopes, bridge span, U-turn lane, lateral and vertical clearances, various travel paths on U-turn lanes, and U-turn access lanes.

TxDOT-sponsored research project 0-4986 assessed the effectiveness of the wide variety of frontage road exit ramp and U-turn yield treatments that exist in Texas (*8*). Researchers collected field data at a number of sites around the state of Texas that represent the five categories of current U-turn yield treatments, as shown in practice (see Figure 2).

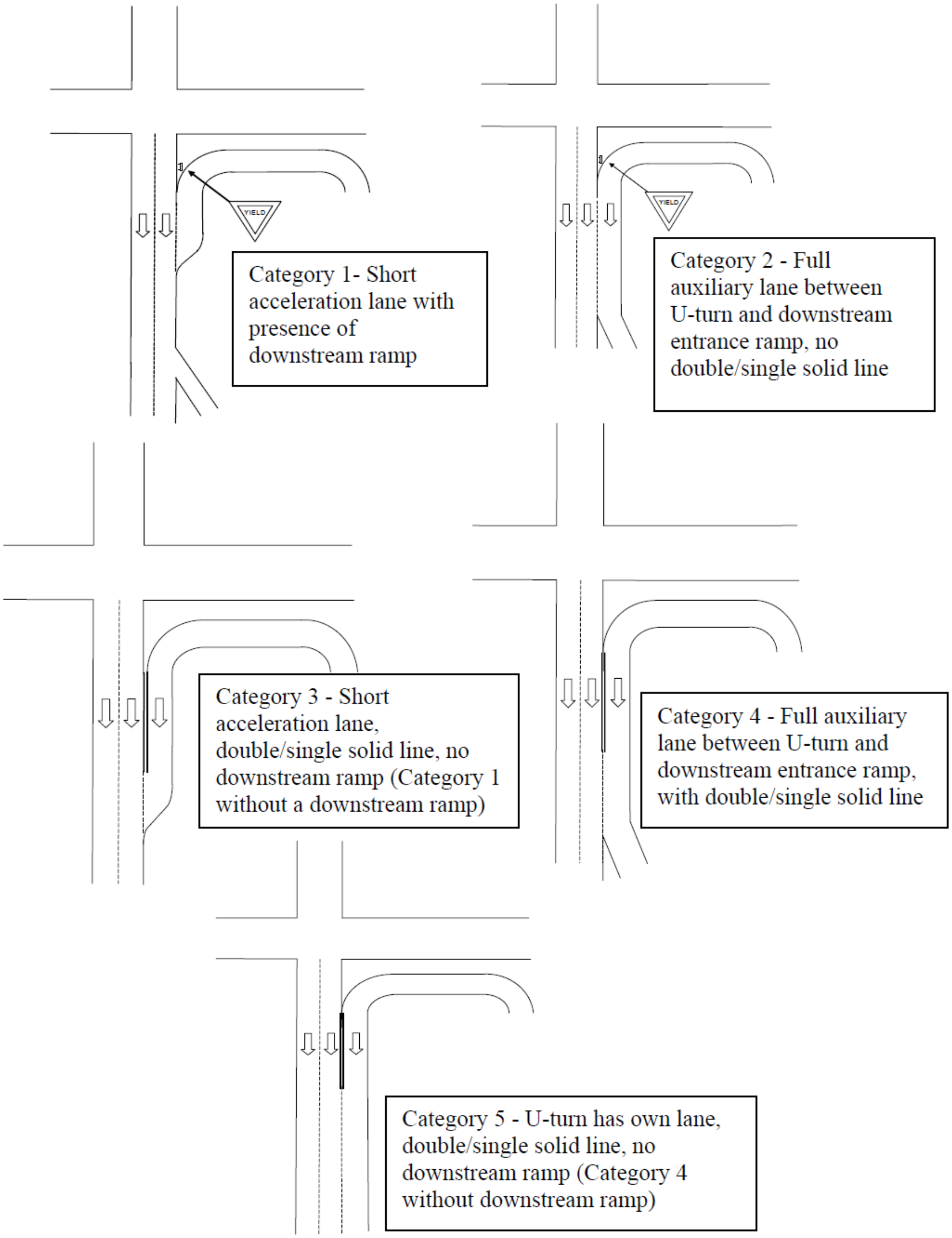


Figure 2. Five Categories of U-Turn Yield Treatments in 0-4986 (*8*).

To assess the plethora of prevailing operating characteristics (e.g., variances in speeds, volumes, driveway densities), researchers used simulation modeling procedures to compensate for the impracticality of the data collection effort that would be required for every possible combination thereof. Several key operational and geometric features of each case study site were carefully replicated and analyzed to produce a calibrated model for each case study condition.

The study concluded that, with no downstream entrance ramp, Category 5 appears to provide the best overall performance; provision of the continuous lane will result in better operation and safety (but the U-turn flows may not justify the addition of a lane); with a downstream entrance ramp, Category 4 seems to provide the best overall performance; however, Categories 2 and 4 are very close. The provision of an added lane unsurprisingly results in improved efficiency and safety; the addition of a Yield sign does not appear to improve safety, although the case of no Yield sign without an acceleration lane was not considered (*8*).

Several previous studies have examined the different elements in the diamond interchange as part of a larger effort to find strategies for improving traffic operations at signalized diamond interchanges. In a 1992 TTI study (01-31-92-1148), Herrick et al. developed procedures for identifying, evaluating, and selecting the optimal design and signal control strategy for five types of signalized diamond interchanges for Texas design conditions under both under-saturated and oversaturated traffic (*9*). Those diamond interchanges include conventional tight urban diamond, single-point urban diamond, split diamond, three-level box diamond, and three-level stacked diamonds. In a 2000 TTI study (7-4913), Chaudhary et al. developed guidelines for the optimal operation of isolated diamond interchanges as well as the coordination of diamond interchanges with adjacent signals on the arterial (*10*). Guidelines/models were developed and tested using computer simulation and then applied to two facilities located in the Corpus Christi and Pharr districts. In TxDOT project 0-6106, Nelson et al. conducted state-of-the-practice surveys, focus groups, and driver surveys to develop test signs used for field deployment and evaluation regarding lane assignment on frontage roads and cross streets (*11*). The research provided recommendations on when to apply non-standard signing to more clearly convey lane assignment to drivers approaching more atypical intersections.

Guidance on Left-Turn Lanes

In typical conditions, the *Highway Capacity Manual* (HCM) treats U-turns as left turns for estimating effective saturation flow rate (*12*). In many cases, U-turn lanes face conditions similar to those of left-turn lanes. NCHRP Project 03-102 expanded on AASHTO guidance for auxiliary lanes at intersections, particularly regarding bypass lanes, channelized right-turn lanes, deceleration and taper length, design and capacity of multiple left-turn lanes, and alternative intersection designs (*13*). In NCHRP Project 03-91, researchers developed a process for determining whether a left-turn accommodation is justified at an unsignalized intersection and, if so, the types of accommodations that are appropriate (*14*). The process considers safety, operational efficiency, and construction costs, and the researchers developed the design guidance for typical left-turn accommodations. In the report, they also described the likely benefits and impacts of accommodations.

TxDOT project 0-5998 investigated the impact of traffic congestion on signalized operations and developed guidelines on how to operate congested signal systems (*15*). In this project, researchers conducted VISSIM-based computer simulation to study the impacts of queue spillback in the vicinity of left-turn-bay entrances (including blocking of through vehicles by queue spillback from the left-turn bay and blocking of the left-turn bay by through vehicles) for:

- A range of bay lengths.
- One- versus two-lane left-turn bays.
- A range of distributions of left-turn and through vehicles in the leftmost lane on the intersection approach upstream of the bay entrance.
- Four left-turn phasing sequences.
- Actuated versus fixed-time control.
- Signal cycle lengths.

These factors resulted in numerous unique geometric plus traffic scenarios. All simulations consisted of fully loaded traffic demand conditions to achieve congested traffic conditions. For each scenario, researchers conducted five replications of simulation and averaged the results before making inferences. From this analysis, researchers found that:

- The worst scenario occurs when there is equal distribution of left and through vehicles in the lane feeding traffic to the left-turn bay.
- When blocking occurs, increasing cycle length decreases capacity.
- With optimal cycle length and phasing sequence:
    - A 500-ft single-lane is sufficient to provide the maximum capacity, which is 95 percent of the ideal capacity.
    - A 400-ft dual-lane bay is sufficient to provide up to 99 percent of ideal capacity.

However, the geometry and operations of signalized diamond interchanges are significantly different from standard multiphase intersections. Therefore, these results cannot be directly applied to diamond interchanges with U-turn lanes. Nonetheless, similar analyses can be conducted to directly or indirectly study the impacts of various factors on the capacity of U-turn lanes together with other movements at a diamond interchange. These factors include:

- Interchange phasing sequence (e.g., three-phase, four-phase, and non-standard).
- Distribution of U-turn, left-turn, and through traffic.
- Level of traffic demand.
- Distance of U-turn entrance from the stop bar.
- Design of the U-turn in terms of total storage and storage parallel to the main lanes on the FR.
- Number of lanes and turn bays on the FR approach to the interchange.

- Design of the U-turn lane on the exit side (direct entrance or acceleration lane with delineation).
- Amount of traffic weaving that may impact the operation at the interchange. Weaving is a function of origin-destination (OD) patterns and interchange design (that is, diamond versus X interchange).
- U-turn control on the exit side combined with sequence of phases and control (e.g., right turn on red, protected right-turn, protected versus permissive left turn).
- Speed differential between exiting U-turn vehicles and conflicting traffic from the intersections.
- Exit-side traffic weaving between U-turn vehicles and conflicting traffic.

Other U-Turn Practice

In recent years, a number of innovative intersection designs, often involving U-turn lanes, have been researched and implemented to reduce delay. These designs provide alternative ways to better accommodate the through and turning traffic.

The Median U-turn (MUT) intersection design, also called “ThrUTurn,” guides all traffic, except right-turning vehicles, through the main intersection. The traffic desiring to turn left does so through a U-turn opening in the median beyond the main intersection. The City of Plano, Texas, installed a variation of a MUT design on SH 289 and Legacy Drive. It was reported that the new design provides 20 to 50 percent greater capacity.

A Diverging Diamond Interchange (DDI) (also known as a Double Crossover Diamond, DCD) accommodates left-turning movements onto arterials and limited-access highways while eliminating the need for a left-turn signal phase at signalized ramp terminal intersections. On the cross street, the traffic moves to the left side of the roadway between the signalized ramp intersections. This allows drivers of vehicles on the cross street who want to turn left onto the ramps the chance to continue to the ramps without conflicting with opposing through traffic and without stopping. El Paso built Texas’s first such interchange at the intersection of Loop 375 and Spur 601. Another DDI was recently constructed at RM 1431 and I-35 in Round Rock, Texas.

A superstreet is a divided highway with intersections in which the minor cross-street traffic is prohibited from going straight through or turning left. The minor cross-street traffic must turn right and then access a MUT to proceed to the desired direction. Two superstreet corridors in San Antonio, Loop 1604 West and US 281 North, experienced reduced travel time and increased speed after the new design operation.

San Marcos, Texas, has implemented the Displaced Left-Turn Intersection (with signalized U-turn lanes that require U-turn queue storage space) at interchanges I-35 at SH 80 and I-35 at SH 82. The southbound to northbound U-turn at SH 82 has two lanes to help accommodate the displaced left-turn volume.

ASSESSMENT OF TXDOT PRACTICES

Introduction

For this task, researchers collected information about TxDOT district practices related to the planning, design, and operation of U-turn lanes at diamond interchanges (i.e., turnaround lanes). To facilitate the information-gathering process, researchers developed a list of questions (provided in Appendix A of this report) to ask each respondent. The questionnaire document included a list of related factors possibly affecting U-turn demand and capacity and potential solutions for improving efficiency; these factors were identified by researchers based on their expertise in the subject area and on the literature review. Next, the researchers contacted staff in the TxDOT districts via telephone and email to solicit responses to the questions in the document. The researchers also asked TxDOT staff to review the list of related factors. In many cases, researchers emailed this document to the identified staff in each district and followed up with a telephone call. Collectively, these selected TxDOT staff members had familiarity/expertise in planning, design, and operations or in a combination of these areas. Researchers received responses over the phone and/or in a written form using a copy of the above-mentioned document sent to them via email.

TTI attempted to make contact with all TxDOT districts but was unable to acquire responses from TxDOT personnel in the following districts:

- Beaumont District.
- Dallas District.
- Odessa District.
- Paris District.
- Tyler District.
- Waco District.

Of the 20 districts from which researchers received responses, two districts stated that they did not have relevant sites. TTI found that the Brownwood District does not have any diamond interchanges, and the Childress District does not have any diamond interchanges with U-turn lanes. The following sections summarize the responses received from the remaining 18 districts. Note that the state-of-practice information for the Waco District presented below was provided by the immediate past director of transportation operations for the district, who served in this capacity for many years.

General Information

Table 2 provides general information about districts' use of U-turn lanes at diamond interchanges. The general information can be summed up as follows:

- In most districts, the majority (90 percent or more) of interchanges with U-turn lanes are located in urban areas.
- In five districts (Abilene, Laredo, Lubbock, Pharr, and San Angelo), all or most U-turn lanes are at tight diamonds, which have interior spacing of 450 ft or less.
- The Atlanta District has several X-interchanges with U-turn lanes.

Table 2. General Information from TxDOT Districts.

District	Respondent*	Location	Other comments
Abilene	Pat Mckennon	95% are urban	Most tight diamonds.
Amarillo	Heath Bozeman	95% are urban	
Atlanta	Rebecca Wells (O)	Most are urban	About a dozen X-interchanges with U-turns along I-30 in Texarkana.
Austin	Keith Taylor (D); Robert Wheeler (O)	More than 90% are urban	Prefer to provide U-turns in urban areas.
Bryan	Mike Jedlicka (O)	Mostly urban	40–50% of interchanges have U-turns. Half in BCS, three in Brenham, and one in Huntsville.
Corpus Christi	Ismael Soto (O)	75–80% urban	
El Paso	Edgar Fino (O)	95% urban	
Fort Worth	Tejas Soni (P)	80–90%	
Houston	Pam Elmer	More than 90% are urban	
Laredo	Danny Magee	Mostly urban	All tight diamonds < 450'. 25–30 interchanges have U-turn lanes. A few on SL 79 outside Del Rio could be considered rural.
Lubbock	Shelly Haris	100% urban	At tight diamonds.
Lufkin	Kelly Morris	70% are urban	
Pharr	Jesse Leal (O)	95% urban	All tight diamonds < 450'.
San Angelo	Thomas Johnson	All urban	All U-turns on tight diamonds.
San Antonio	Clayton Ripps	95% urban	
Waco**	Larry Colclasure (O)	90% urban	
Wichita Falls	Travis Herrell	100% urban	
Yoakum	Amanda Fling (D)	100% urban	

* Letters in parentheses in 2nd column refer to design (D), planning (P), and operations (O).
** Waco response is from a retired TxDOT staff member with long service with the district.

Documents and Guidelines Used during Planning and Design

Respondents provided the following responses related to planning and design:

- Most respondents indicated that they are not aware of any specific guidelines or policies related to the planning and design of U-turn lanes at diamond interchanges.
- There is little detailed traffic data available during the planning and design stages.
- Districts use one or more of the following documents/tools for design, which is typically done by consultants:

- TxDOT RDM (Chapter 3, Section 6).
  - *AASHTO Green Book* and turning templates.
  - *Access Management Manual*.
  - Autoturn.
  - Microstation (3D templates, MSTurn to simulate design vehicle, GeoPak to help design U-turns).
  - HCM, PASSER, Synchro to determine level of service (LOS) and impact of queues on U-turn entrance.
  - *Texas Manual on Uniform Traffic Control Devices* (TMUTCD).
  - Standard U-turn curb detail (Houston District).
- Data/information used in planning and design of U-turn lanes:
  - ADT and percent trucks.
  - Left-turn/U-turn volumes.
  - Congestion level.
  - 20- and 30-year projections using metropolitan planning organization (MPO) or TxDOT Transportation Planning and Programming office data.
  - Level of existing and/or proposed development (i.e., business density) in the vicinity of interchange.
  - Urban versus rural area.
  - Potential for the interchange to become signalized.
- Table 3 provides respondents' comments to the questions. A few additional comments are listed below:
  - It is important to address queuing/blocking on the frontage road approach.
  - A diamond interchange operating in the 4-phase (tight diamond) signal cycle operates better with U-turns.
  - Advance lane assignment signage can help drivers better position themselves as they approach the interchange.
  - U-turn lane design is part of overall interchange design.
  - Bridge span, number of support columns, and the tradeoff between serving U-turn traffic versus serving more through vehicles are important factors.
  - U-turns can be added to alleviate congestion if left-turn traffic is high.
  - Driver expectancy is a factor. Drivers going through the first signal expect the second signal to be green, and it is not uncommon for them to run the second signal if it is red.
  - Frontage road approaches could be widened to provide more lanes.
  - Of all the countermeasures identified in the questionnaire, dual-lane U-turn is the only one noted as potentially problematic by the respondents.
  - Site conditions, such as railroad tracks on one side of the freeway, may limit the suitability of U-turn lanes at that site.
  - Operation of U-turns at diamond interchanges is generally an afterthought.

Table 4 lists U-turns reportedly added by districts and the reasons for doing so. Table 5 provides information about locations reported to have recurring congestion problems. Table 6 identifies locations with temporary issues at U-turn lanes. Table 7 identifies reported information about U-turn lanes redesigned or retrofitted to improve operations or safety. Table 8 lists reported locations currently facing issues. Only the San Antonio District (Table 9) identified any issues with U-turns over freeway underpasses. Table 10 describes responses by district staff who consider U-turn lane design at box diamonds differently than conventional diamonds. Only the San Antonio contact provided a response on how alternate intersection/interchange designs might have any impact on U-turn design (Table 11). Finally, Table 12 identifies potential study locations for which information was provided by respondents. The information in each table is shown exactly as provided by the respondents, including abbreviations and other shorthand notations. Abbreviations in these tables that are used elsewhere in the report include northbound (NB), southbound (SB), eastbound (EB), and westbound (WB).

Table 3. Additional Comments from TxDOT District Respondents.

District	Comments
Austin	The Austin District tries to provide U-turn lanes in urban areas when possible.
Bryan	SH 6 ramp configurations at all locations (many of which are complete) are being changed from diamond to X to move high-speed freeway weaving to lower-speed frontage road (FR) weaving. Metropolitan Planning Organization (MPO) data were used for this decision and to improve operations and safety.
Corpus Christi	The Corpus Christi District tries to build U-turn lanes if there is development on both sides of the freeway. If current volumes do not necessitate a U-turn, the District will leave span length available so the U-turn can be built with limited expense in the future. Vertical clearance is an important but often overlooked issue when it comes to U-turn lanes. A sign with vertical clearance information can be located on the turnaround sign to inform drivers. Closing U-turn lanes for construction can easily clog interchange operation in an urban area. In these situations, the District may have to change signal timing at the interchange to avoid trapping U-turning traffic in the interior of the interchange (using 4-phase timing). Consider volumes for determining minimum turn lane and accelerations lane lengths. However, detailed traffic data is usually not available at the design stage. There are some locations in the district where turning templates should have been used to better accommodate trucks.
Fort Worth	The Fort Worth District does not provide U-turn lanes at interchanges with two-way FRs. U-turns are considered as part of two-way FR conversion to one-way, which creates the need to provide better access to adjacent properties. The district tries to provide a U-turn every two miles and uses main lane design vehicles for designing the curve. The standard width of a U-turn lane is 14 ft. The district ensures that there are no drainage issues, available crash history at the location or at similar nearby locations has been considered, there is adequate sight distance for vehicles exiting the U-turn lane, and speed differential between U-turn and on coming through traffic on the exit side are considered. Also, the use of an acceleration lane is not common. The district also provides U-turn lanes upstream of crossing railroad tracks.
Laredo	The Laredo District always tries to install U-turn lanes for diamond interchanges. Constraints on project construction budget would be the only reason not to install them.
Lubbock	U-turns are only constructed if the FR is one-way. An example is US 62 at South Loop 193 where one side is one-way and the other is two-way and there is a U-turn for only one movement. The bridge is wide enough to provide a U-turn in the other direction.
Lufkin	At one location, one left-turn lane was changed to two left-turn lanes and this change may have helped minimize queue blockage of the U-turn entry.
Pharr	Generally try to install them if the geometry allows.
San Angelo	Larger spacing typically associated with low volume conditions.
San Antonio	U-turn is a given in urban areas. Interchanges are designed for U-turns even if their construction is deferred. For skewed interchanges, there is an issue (with no guidance) on whether to build the U-turns parallel to the cross street (i.e., also skewed) or perpendicular to the frontage road.
Yoakum	All new overpasses with frontage road will have U-turns.

Table 4. U-Turn Lanes Added in Last Five Years.

District	Locations and Reasons
Abilene	I-20 at Loop 322 for movement and future development and I-20 at SH 351 for alleviating congestion caused by a new WalMart at the north corner.
Amarillo	I-40 at SL 335 (Lakeside). The turnaround where done as temporary measure to help with freight traffic that utilizes the fuel facilities at that intersection. A procurement is currently being developed to do a full interchange.
Atlanta	In Texarkana, U-turns were added as part of I-30 reconstruction, which also converted two-way frontage roads to one-way. In Mount Pleasant, U-turns are included in the construction of a new by pass.
Austin	Several along I-35. SH 71 at Loop 50 and several other locations in Bastrop. SH 71 at Riverside in Austin. US 290 Manor Expressway (CRRMA Project), US 183N Toll Road. In general, the purpose was to improve operations.
Austin	Yes, for example Hwy 71 at Riverside. To improve throughput on the side street. Also, SH 29 at I-35, U-turns were added to provide improved access.
Bryan	SH 6 @ Rock Prairie, U-turn was added to improve operations. SH 6 at W.D. Fitch interchange was built with U-turn lanes. At this location, East-side has a barrier to prevent Southbound SH6 U-turn traffic to make sudden lane-changes to access adjacent development. This was done to prevent any potential safety issues.
Corpus Christi	None, but considering 2 locations. First is a diamond with an intersection ¾ mile away that will be converted to RI/RO. U-turn at interchange will improve local circulation since access will be restricted at the regular intersection. Second location is a U-turn retrofit to handle an anticipated very large increase in traffic (U-turn included) as a result of industrial expansion in the SH 361 and SH 35 areas.
El Paso	Several recent or planned locations: I-10 Collector Distributor Project. U-turns planned at I-10 at SH 20 (Mesa) and I-10 at Sunland Park. Construction of Spur-601. U-turns built at Spur-601 at Chaffee, Spur-601 at Global Reach, Spur-601 at Constitution. Construction of LP 375 Mainlanes—U-turns built at LP 375 at Northwestern, LP 375 at Resler, LP 375 at Paseo Del Norte, LP 375 at US-54, LP 375 at Kenworthy, LP 375 at Rushing, LP 375 at Alcan, LP 375 at BU-54 (Dyer), LP 375 at (FM 2529) McCombs. All projects were done for operational improvements.
Fort Worth	U-turns adding at FM 5 and I-20 as part of an interchange improvement project. This location has a skewed angle, alignment shift, and speed vertical grade issues, which may require a design exception. In the design process observed U-turn at an adjacent location (Ranch House Rd at I-20), which has truck-caused side-swipe marks on retaining walls. U-turns also added/being-added at several locations due to new development. These locations include: Bryant Irvin at I-20, Basswood Dr. at I-35W and Six Flag Dr. at I-30.

Table 4. U-Turn Lanes Added in Last Five Years (Continued).

District	Locations and Reasons
Houston	The following projects and permit provided for the addition of U-turns to facilitate projected traffic (including 18 wheeler) due to development at locations: (1) CSJ: 0508-07-286: Location: Spur 330 at Decker Dr. Description: Increase the U-turn lane storage. (2) CSJ: 0508-01-349: Location: I-10 at John Martin- Description: U-turn from I-10 Frontage Rd WB to EB. (3) I-10 @ Sjolander—U-turn added by permit (SE Harris Area Office).
Laredo	Yes, Loop 20 at SH 359 and at MacPherson. Projects were done primarily as congestion relief projects converting very busy intersections to interchanges. U-turns were a part of that and help improve efficiency but were not the main purpose.
Lubbock	Yes, for improving operations.
Pharr	Yes, turnarounds recently constructed but not for specific reasons having to do with U-turning needs. Part of larger freeway expansion projects.
San Angelo	One that was added to alleviate traffic at main interchange; Loop 306 and US 67.
San Antonio	Yes, US 281 at SH 46 had turnarounds added to accommodate the traffic from new development in the area. Pulling the U-turn traffic from the interchange's signals was intended to alleviate expected congestion.
Waco	• I-35 at FM 286 in Hillsboro, improve traffic flow near the outlet mall. • I-35 at FM 2114 in West, added as a part of expansion of I-35. • I-35 at Big Elm Rd north of Temple, added as a part of expansion of I-35. • I-35 at Old Blevins Rd north of Temple, added as a part of expansion of I-35. • I-35 at Eddy Dr. in Bruceville-Eddy, added as a part of expansion of I-35. • I-35 at Telephone Rd south of Lorena, added as a part of expansion of I-35. • I-35 at FM 2847 in Lorena, added as a part of expansion of I-35. • I-35 at FM 2847 north end of Lorena, added as a part of expansion of I-35. • I-35 at FM 3148 north of Lorena, added as a part of expansion of I-35. • I-35 at MLK in Waco, added as a part of expansion of I-35. • I-35 at New Road in Waco, added as a part of expansion of I-35.
Wichita Falls	None were added recently, but we do have one planned for US 82/Grand Avenue in Gainesville. Scheduled letting date is February 2016. Reason—Operational.
Yoakum	No. All existing ones are in Victoria County on LP 463 (US 87 Railroad [1 side]), Mallette Drive, US 77 (Navarro), John Stockbauer, Salem, Mockingbird, and Airline Overpasses. Also on US 59 at LP 463 Overpass (1 side) and at US 59 at US 87 Railroad Overpass (1 side). They were all done as part of original design project "Construct Overpass."

Table 5. Any Locations with Known Recurring Congestion Issues.

District	Locations and Descriptions
Austin	Recurring congestion is generally the issue in Urban areas. I-35 at 123 in San Marcos has a skewed geometry because of which 18-wheelers cannot use U-turn lanes, so they use signalized movements. As a result, protected left had to be provided at the 2nd intersection to accommodate them.
Bryan	SH 6 at University and SH 6 at Briarcrest, both of which do not have U-turns. U-turn lanes can help at these locations.
Corpus Christi	Frontage traffic blocking access to U-turn lane is an issue in heavily developed areas. Receiving frontage road can also be so busy that U-turn traffic cannot exit the lane, esp. if no U-turn acceleration lane is provided.
El Paso	I-10 at FM 659—Congestion, high volume interchange plus serving as detour for current construction on I-10 at LP 375.
Fort Worth	Lake Shore Drive at I-20.
Laredo	Roughly three quarters of U-turn lanes in the district face some form of operational issue. The most significant issue is that U-turns typically don't have their own lane at the receiving/downstream frontage road. U-turn traffic also has to compete/weave with traffic wanting to get to the downstream freeway entrance ramp, as well as merging across the frontage road to get to driveways. Queues at the interchange signal can also block access to the U-turn lane. Also important to note that not all U-turn lanes were originally designed (turn radii) to accommodate 18-wheelers. All should have been designed this way.
Lufkin	SL 287 at Tulane, Lufkin and SL 287 at US 69 South, Lufkin.
Pharr	Yes. Queues building in the leftmost frontage lane block the opening to the U-turn lane. There are no capacity problems for the U-turns in most cases once vehicles can access the turnaround.
San Angelo	Yes, because there isn't adequate storage at the interchange to allow traffic to access U-turn. This is primarily an issue with older interchanges and U-turn design practices from 20+ years ago.
San Antonio	Yes, there are locations where access from driveways to a U-turn entry is problematic, as is the case where traffic from a U-turn lane wants to weave quickly to the right to access driveways. Curbs have been used to reduce these movements due to safety concerns.
Wichita Falls	US 82 westbound at two locations—Kemp Street and Lawrence Street.

Table 6. Locations with Temporary Issues at U-Turn Lanes.

District	Temporary issues at U-turns
Abilene	Maybe during construction.
Amarillo	Yes.
Austin	I-35 DDI project 1431–Access to Turnaround was closed during phasing that significantly affected traffic movements due to high demand for movement.
Corpus Christi	Yes. Such locations could be improved with longer U-turn access lanes and a U-turn acceleration lane.
Laredo	None.
Lubbock	Only if there is a wreck in the lane.
Lufkin	SL 224 at US 59, Nacogdoches.
Pharr	None.
San Angelo	None.
San Antonio	U-turns have been part of detour routes, but there are usually no problems as long as the U-turn entry lane is long enough that vehicles wanting to U-turn are not blocked by left-turning queues.

Table 7. U-Turn Lanes Redesigned or Retrofitted to Improve Operations or Safety.

District	Operations	Safety
Abilene	At US 83 and FM 89, Pylons were installed to prevent rapid lane change from U-turn acceleration lane to a driveway.	
Amarillo	Yes.	Yes.
Atlanta	Generally both factors were at play in the I-30 reconstruction and frontage road (FR) conversion to one way. The District did not want people and businesses affected by FR conversion to one-way due to limited accessibility. U-turns allow people to access businesses without going through the signal.	
Austin	At one location in San Marcos, U-turn was removed to provide more room for main lanes. On SH80 @ I-35, redesign to DDI added U-turn.	
Corpus Christi	Added curb delineators, eventually transitioning to a linear concrete curb (lower maintenance) to limit rapid lane changes to frontage driveways. Past design practices did not use truck turning templates when designing U-turn lanes; this is now routinely done. Some change locations with revised U-turn radii are along the urbanized portions of SH 358	
El Paso	I-10 at Lee Trevino. Retrofitted.	
Laredo	Delineator curbs have been used at several locations (I-35 at Calton, I-35 at Mann) to reduce cut-across traffic along the frontage road from the U-turn lane to driveways. Making these curbs permanent at a few locations.	
Lubbock	Two-way frontage road to one-way, I-27 at US 270 in Plainview.	
Lufkin	SL 224 at US 59, Nacogdoches, to improve both operation and safety. US 59 at US 190, Livingston, to improve both operation and safety.	
Pharr	At I-69E/SH 48 (Boca Chica), queues were blocking access to the lane. Widened and lengthened the U-turn approach to improve access. Also, many U-turn lanes around the district used to have a yield condition at the receiving frontage road; most were converted to an acceleration lane (last 10 years) to improve U-turn operations.	
San Angelo	No.	
San Antonio	Some elevated U-turns (i.e., those on bridge structure) have required reconstruction to improve turn radii. One example is Loop 1604 NB to SB U-turn at Culebra.	

Table 8. Locations Currently Experiencing Issues.

District	Operations	Safety
Amarillo	Yes.	No.
Bryan	Operational issues at some BCS locations during peak periods.	
Corpus Christi		Yes, primarily those U-turn locations where the exit control is a yield sign (i.e., no acceleration lane). These sites present higher-than-expected rear-end crash frequency. SH 358 at Greenwood is an example.
Laredo	I-35 at Calton, Mann and Del Mar. Off-ramp traffic weaves to make right turns at the diamond interchange, and slows frontage traffic (which must yield) that must weave across this traffic to reach the U-turn lane.	
Lubbock	Some interchanges on SH 289 between US 87 and SH 327 may have occasional issues with accommodating trucks. No room for wider bridge.	
Lufkin	SL 287 at Tulane, Lufkin and SL 287 at US 69 South, Lufkin.	
Pharr	Interchanges along I-2 in western McAllen (such as FM 2220 and SH 494) are experiencing heavy demand due to development and large spacing between interchanges. U-turns blocked by queues.	
San Angelo	Sites where gore extensions have been added; paddles added but not low profile barriers due to possible safety concerns.	
San Antonio	I-410 at Callaghan. On the WB to EB U-turn downstream side, traffic wants to weave across to a gas station driveway.	

Table 9. Issues with U-Turns over Freeway Underpasses.

District	Comment
Lufkin	One interchange in the district had a design flaw where the bridge for the U-turn was not wide enough to accommodate truck turns. It is being redesigned by the consultant.
San Antonio	Yes. Tangent lane width is not as wide as for U-turn lanes at freeway overpasses, and turn radii are often exaggerated. Also, entry deceleration and exit acceleration lane are typically not as long, since these things affect retaining walls, etc. U-turns are more likely to be deferred at these locations since construction cost is higher.

Table 10. Any Box Diamonds Where U-Turn Designed Differently.

District	Any	Designed Differently?
Corpus Christi	Closest is SH 286 at Laredo/Agnes.	This site should have been designed for trucks (turning templates) when it was built many years ago, but it was not. Operation gets "choked" due to limited access lane length and tight turn radii.
Laredo	Yes.	I-35 at Loop 20. Nothing is unique at this location in terms of U-turn design. There are U-turn lanes on all four approaches.
Pharr	Yes.	I-69E at Tyler/Harrison and I-69E at Spur 54 in Harlingen. Nothing unusual at these sites.
San Antonio	U-turns were retrofitted and/or improved in these cases, but the basic design is the same as regular diamonds.	

Table 11. Any Design Changes due to Proximity of Other Interchange/Intersection Forms.

District	Design Changes
San Antonio	This is an issue with divergent diamond interchanges and displaced left turns, since the U-turn lane can be signalized in the future. Also, separate turnarounds and displaced left turn lanes may be needed if volumes are high enough to cause congestion.

Table 12. Potential Study Locations.

District	Suggested Study Locations
Abilene	I-20 at SH 351 and US 83 at FM 89. At the second location, NB 1-way FR ties into a two-way surface street, which intersects with FM 89 to form one of the intersections of the diamond. NTCIP, VIVDS, city operates and can provide more details about phasing and if VIVDS can be accessed.
Amarillo	I-40 at SL 335 (Soncy). Being constructed currently to address issues. Originally was not going to do a turnaround on the west side of the intersection, but it is being change ordered in.
Atlanta	I-30 at FM 559 and I-30 at SH 93. No known issues. 3-phase. NTCIP compliant controller. Video detection and will permit use of recording equipment. Will also allow other data recording equipment in the cabinet.
Austin	If suitable sites were to be identified, District would allow data collection. However, policy requires that district staff be present at the cabinet when researchers go there. Existing control is 3- and 4-phase, U-turn yield at exist and RTOR allowed. District uses Econolite ASC3s. Mostly (90%) video and some legacy loops. Leaning toward radar. Have about 10 Matrix and Advance sensors. Advance sensors are used to provide extension. No F2C com. Have Ethernet radios, but not working at this time.
Bryan	University and Briarcrest mentioned earlier. Existing control in the district is 3-phase or 4-phase, U-turn yield at exit and RTOR allowed. Siemens M50 controllers. Mostly video detection. No C2F comm. Will allow use of these or other sites for field studies including use of video or data recording equipment.
Corpus Christi	US 77 south of Robstown will soon be designed as an I-69 freeway section (currently a divided highway). This location has a RR on one side of the future freeway corridor, raising questions about the need for U-turn lanes. Perhaps they should just be provided in one direction? Examine volumes to determine the need, but there are no firm criteria. Tight Diamond. Most are 4-phase, a few 3-phase. CC transitioning to Matrix (radar), some PTZ being installed for status monitoring and some VIVDS around. District has some traffic counts, all available counts are car/truck classified.
El Paso	I-10 at FM 1281(Horizon)—Diamond interchange, diamond ramps, bypass lanes, turnarounds both directions on I-10, 2 through lanes on FM 1281 with dedicated left- and right turn bays, 2 through lane approach on I-10 with dedicated turnaround, left and right turn bays. This site has heavy congestion (high truck volumes), truck stops on both sides of interstate, close adjacent signalized intersections North and South of I-10 on FM 1281. It is 4 phase, U-turns yield at exit, right on red allowed. Naztec TS-2 cabinet and controller. Streetwise. VIVDS, C2F and F2F comm using spread spectrum radio. No traffic data. Would allow data recording equipment in the cabinet.
Houston	I-10 at Bunker Hill; I-10 at Gessner; and I-45 North Research Forest Dr.

Table 12. Potential Study Locations (Continued).

District	Suggested Study Locations
Laredo	Nothing necessarily unique about U-turn lanes in Laredo, but a typical example with Laredo operations concerns is I-35 at Mann Road, which is a tight diamond with yield control for U-turn. Has a relatively new Naztec controller, 4-phase operations. VIVDS at most interchanges. Video from these can be displayed in STRATIS TMC, but this feature is down at present.
Lubbock	US 84 at FM 2528 Interchange (under construction). Skewed roadway.
Lufkin	1. US 190 at 59. This is a 3 phase Diamond W/ U turns and Yield signs Siemens Controller and Cabinet Firmware 3.34 Not NTCIP Compatible. Iteris Vivids detection system. No Volume /classification Data. Should have room in cabinet. 2. US 69 South at SL287/ 59. 4 phase Diamond W/ U turns And Yield signs. Siemens Controller and cabinet Firmware 3.33 not NTCIP Compatible. Iteris Vivids detection system. No Volume/ Classification Data. Should have room in cabinet. 3. US 59 at SL224. 4 phase diamond with one U turn and Yield sign. This intersection is under construction and unknown what detection will be until contractor provides. This will be a new cabinet Siemens. No Volume/ classification data. Note: All these locations firmware can be updated to NTCIP compatible if needed. These are all Siemens M40 but we can update to M50.
Pharr	Congested locations include I-2 at FM 2220 and SH 494. Long queues block access to U-turn lanes. U-turn demand present due to wide spacing between interchanges. Tight diamond. Usually acceleration lane Naztec NCTIP. 4-Phase. Most interchanges have loops. VIVDS used at a few sites (perhaps Shary Road—SH 494), but video is not brought back to district HQ. For about half of locations, have existing TMC counts with classification.
San Angelo	Loop 306 at US 67; took a while for people to get used to using it. Volume levels are good now; newer design so long entrance lanes provided. Most are tight. Siemens/Eagle M52. All diamonds use 3-phase timing. Video for stopbar and advance detection. Video is not brought back to central, but should be able to record at the cabinet. For most, no field data.
San Antonio	EB to WB I-410 at Callaghan. Heavy LT to Medical Center off of frontage, and access to U-turn lane is blocked by LT queue. I-410 at Ingram, WB to EB U-turn is heavy. Tight diamonds with yield and/or acceleration lanes. City runs most and uses 2070s. Loops 50%, VIVDS 45%, and Radar 5%. Some existing data, but no classification.
Wichita Falls	US 82 (Kell Freeway) westbound at Kemp Street and Lawrence Street. See response for details of geometric characteristics. Interior distance for both 325'. Both have U-turn lanes. Problem: Truck Queues preventing use of U-turn lanes, primarily relative to westbound ramps. Timing is non-diamond with overlaps. TS-1 Cabinets—Econolite ver. 2.54. VIVDS at Kemp and Loops & radar at Lawrence. Communication uses 900 Mhz Motorola Radio. Will allow recoding equipment in cabinet if room exists.
Yoakum	With the limited amount of U-turns in our district, we feel that researchers should conduct their research in a district that deals with more U-turn issues.

FACTORS AFFECTING U-TURNS AND SOLUTIONS TO OPERATIONAL ISSUES

After receiving feedback from the districts, researchers were able to supplement the original list of factors affecting demand and capacity as well as the list of potential solutions for improving U-turn efficiency. In particular, several interesting factors were revealed during the questionnaire process. Many district personnel believed that acceleration lanes help greatly with both the operation and safety of U-turn lanes as opposed to the typical Yield sign without acceleration lane. Also, several districts have effectively used barriers (including curbs and pylons) to prevent vehicles from weaving from the U-turn lane to nearby driveways. Furthermore, truck accommodation in U-turns seems to be a major concern for many districts. Some districts face problems because previously designed U-turns cannot accommodate trucks, which lead to trucks stacking up in the left-turn lane, thus blocking access to the U-turn lane. One district even had to rebuild a U-turn lane because a U-turn could not handle trucks. Other districts tend to have issues with how to handle skewed intersections and problems with trucks using those U-turns. Finally, many districts agree that tight diamonds with 4-phase signal operation work well in serving the interchange traffic and minimizing blockage of the U-turn lane. After reviewing all of this information, researchers prepared the following revised lists of factors and solutions.

Factors Possibly Affecting U-Turn Demand

Six factors were identified as potentially affecting U-turn demand:

- Lane use/assignment.
- Nearby development intensity.
- Proximity and number of nearby driveways.
- Ramp configuration (diamond or “X”).
- Distance to downstream entrance ramp.
- Interchange spacing (i.e., distance between consecutive U-turns along an FR).

Factors Possibly Affecting U-Turn Capacity

Six factors were identified as potentially affecting U-turn capacity:

- Traffic volumes and patterns:
  - Truck percentages.
  - Lane utilization.
  - Volumes at approach to U-turn.
  - Volumes on FR receiving the U-turn.
- Interchange geometrics:
  - Tight diamond versus traditional/rural diamond.
  - Lane widths, storage bay lengths, acceleration lane lengths (if present at all).
  - Can trucks use U-turn (proper templates used?).

- For skewed cross streets:
  - U-turn perpendicular to the frontage.
  - Skewed like the cross street.
    - Trucks may not be able to use skewed U-turns.
- U-turn traffic control:
  - Yield sign.
  - Yield pavement markers.
  - No Yield sign/other.
- Interchange signal phasing:
  - Four-phase.
  - Three-phase.
  - Other.
- Right-turn demand from the cross street.
- Driveway access near the interchange.

Potential Solutions and Techniques for Improving U-Turn Efficiency

Eight solutions were identified as having potential to improve U-turn efficiency:

- Modifications to signal timing plans to reduce queue length and facilitate access to lanes or bays at the start of each U-turn.
- Modifications to signal timing plans to facilitate access to FR lanes at the end of each U-turn and/or signalized control of the U-turn approach.
- RTOR restrictions on cross street to reduce the conflicts between U-turning and right-turning traffic.
- U-turn bay extensions or added lane(s) to facilitate entry to the U-turn lane.
- Additional lanes to handle the left-turn movement (either by adding a lane or creating a shared left if one did not previously exist), potentially minimizing queue blockage of U-turn lane by left-turning vehicles.
- Two-lane U-turn lanes for added capacity to serve unusually high traffic demand.
- Access controls (barriers, pylons, concrete curb) and/or driveway closure proximate to the interchange U-turn lane.
- Access controls for either the U-turn lane or the right-turn lane from the arterial to remove the conflict between these two movements.

CHAPTER 3. CHARACTERISTICS OF U-TURN LANES UNDER VARIOUS CONDITIONS

INTRODUCTION

This chapter summarizes the activities performed in Task 3 of this project. The objectives of this task are:

- To collect traffic and site characteristics data on a selection of interchanges with U-turn lanes.
- To use these data and previously existing data to conduct simulation studies to establish baseline performance measures for interchanges.
- To document these performance measures for U-turn lanes operating under several different scenarios.

SITE SELECTIONS

As with most traffic operations-related evaluation activities, the data resources needed for analysis of Texas freeway interchange sites, as related to the function and impacts of U-turn lanes, included roadway geometrics, traffic volume, and traffic signal timing data. The first of these items—roadway data—was largely collected as researchers and TxDOT staff identified study sites for the current project and collected information about each of those sites. The primary means of observing and recording the roadway data was using the online mapping application Google Earth™, with researchers recording such details as the number of lanes and lane use for all lanes on each approach to both of the intersections within each study interchange. Supplemental details were measured using utilities contained within Google Earth™ and recorded by research staff; these details included the width of the interchange, the length of storage bays on all interchange approach legs, and the lengths of the various weaving sections between driveways and vehicles exiting or entering U-turn lanes.

In consultation with TxDOT district staff, researchers identified a set of 34 potential study sites in 14 districts. The researchers then carried out aerial surveys of these sites using Google Earth™. The objective of these surveys was to select a minimum of 25 sites for detailed analysis. This section describes the results of aerial view site surveys.

Figure 3 shows the geographic diversity of 26 selected sites. Furthermore, it shows the number of sites selected in each district. Researchers based study site selection on several factors, including diversity of geometric characteristics, land development, and traffic demand level.

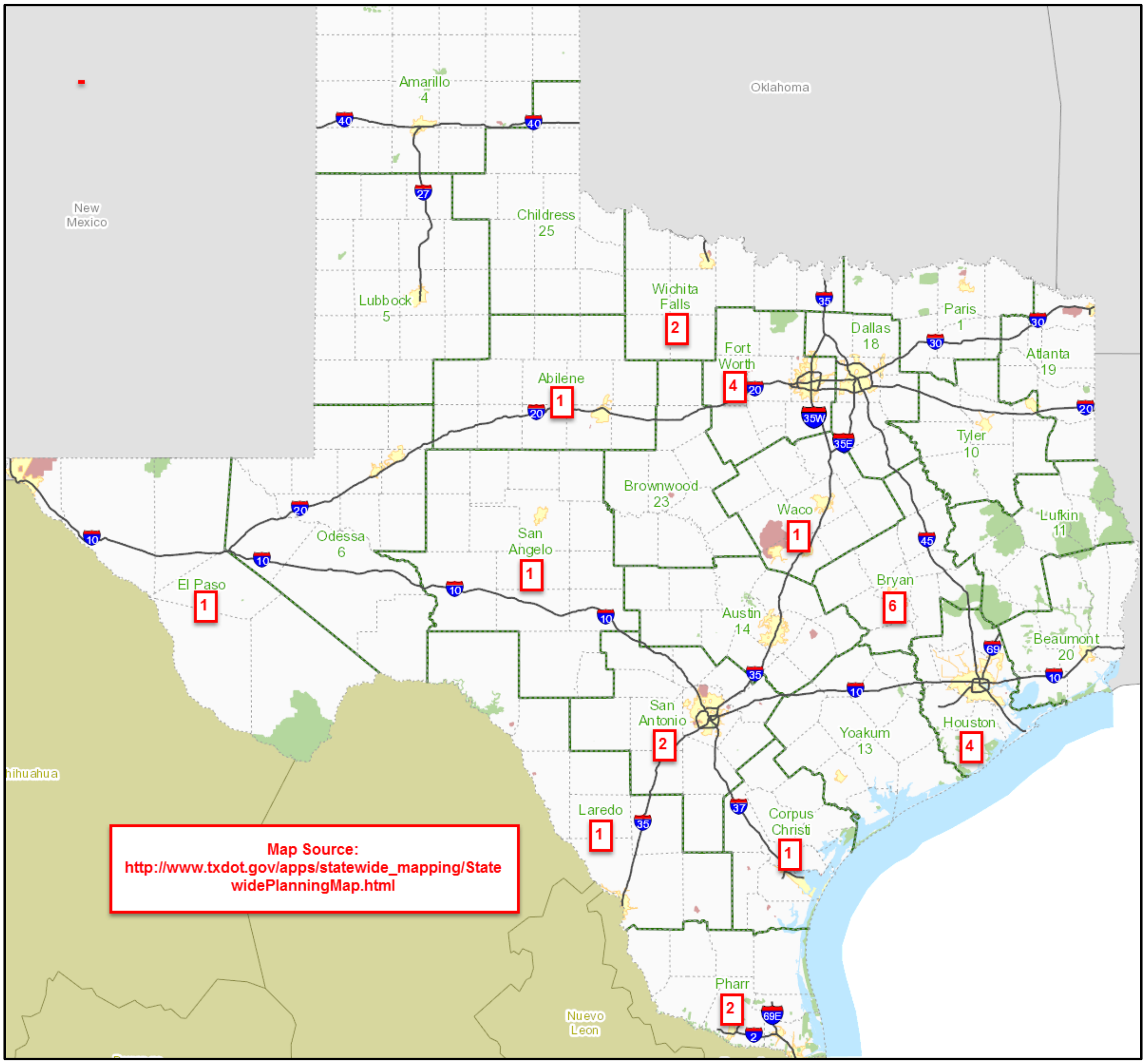


Figure 3. Number of Sites Selected in Each District.

SITE CHARACTERISTICS

The 26 selected sites exhibit the following range of characteristics:

- Freeway over or under the cross street:
  - There were 18 sites with the freeway over the cross street.
  - Eight sites had the freeway under the cross street.
- Diamond versus X configuration:
  - Four sites were diamond interchanges.
  - 12 of the sites were X-interchanges, including all sites in Bryan–College Station that were recently converted from diamond to X-interchanges.

- 10 sites were mixed configurations in which both adjacent ramps on one side were either exit or entrance ramps, or there was a hybrid geometry where one side had a diamond configuration and the other side had an X configuration.

- Presence of U-turn lane:
  - A U-turn lane was on both sides at 17 sites.
  - There was no U-turn lane at four sites; all of these were locations where the freeway passed under the cross street.
  - There were five sites where the U-turn lane was present only on one side; at two of these sites, the freeway passed under the cross street.
- Length of U-turn lane from the stop bar:
  - Average length was 263 ft.
  - Maximum length was 477 ft.
  - Minimum length was 24 ft.
  - Median length was 241 ft.
  - Mode length was 200 ft.
- Maximum width of U-turn lane at three locations:
  - Beginning: Overall range of 12 to 52 ft, but most were between 23 and 36 ft.
  - Middle: Range of 13 to 27 ft.
  - End: Overall range of 14 to 45 ft, but most were between 22 and 26 ft.
- Interior spacing, from the stop line on one side to the stop line on the other side:
  - Range was from 137 to 1313 ft.
- Interior lane configurations include:
  - Left-turn bays (6 sites).
  - Continuous left-turn lanes (19 sites).
  - Continuous left-turn lanes that extend upstream of the intersection.
- Approach-lane configurations on the FR and arterial approaches:
  - Left-turn and right-turn bays.
  - Exclusive lanes.
  - Shared lanes.
- Qualitative measurements of land development:
  - Low intensity.
  - Medium intensity.
  - High intensity.
  - Balanced development.
  - Imbalanced development.
- Number of nearby driveways:
  - From the interchange to a half-mile upstream, the number of driveways on the FR ranged from 0 to 14.
  - From the interchange to a half-mile downstream, the number of driveways on the FR varied from 0 to 21, with the exception of one site that had 32 driveways.

- Distance from U-turn departure to next driveway:
  - The distance to the next downstream driveway on the FR ranged from 0 ft to 796 ft.
- Geometry of departure from U-turn lane:
  - Yield (11 sites).
  - Taper.
  - Added lane.
- Distance from U-turn departure to downstream entrance ramp:
  - The distance along the FR to the downstream entrance ramp varied from 567 to 6427 ft.
- None of the sites have a downstream metered on-ramp.
- Type of interchange signal operations:
  - Standard three-phase.
  - Standard four-phase.
  - Non-diamond mode.

TRAFFIC CONDITIONS

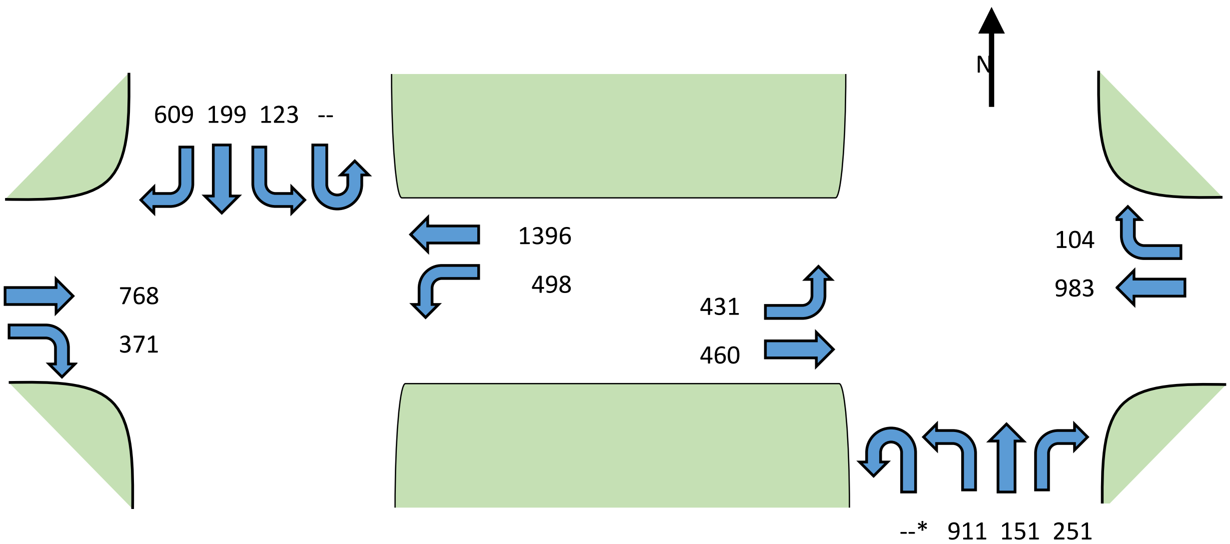
Traffic count data are the primary input requirement for the majority of analyses performed on roadway facilities. Such information provides not only a condition assessment in terms of roadway lane or intersection utilization but also the means to evaluate the performance of roadway facilities. Traffic data collected for this project included turning movement counts, OD counts, and U-turn departure performance data.

Because it was necessary to collect detailed turning movement counts for each approach within each interchange studied for U-turn operations, researchers chose video recording as the means of collecting data at each of the 26 statewide interchange study sites. Cameras were installed to cover the area surrounding the junction between the U-turn departure and the FR, and they recorded for a continuous duration of at least 24 hours. The video recording enabled analysts to observe and count traffic for all study interchanges as well as created a permanent record of traffic behavior that could be later reviewed for additional information, such as gap acceptance while drivers were departing turnaround lanes.

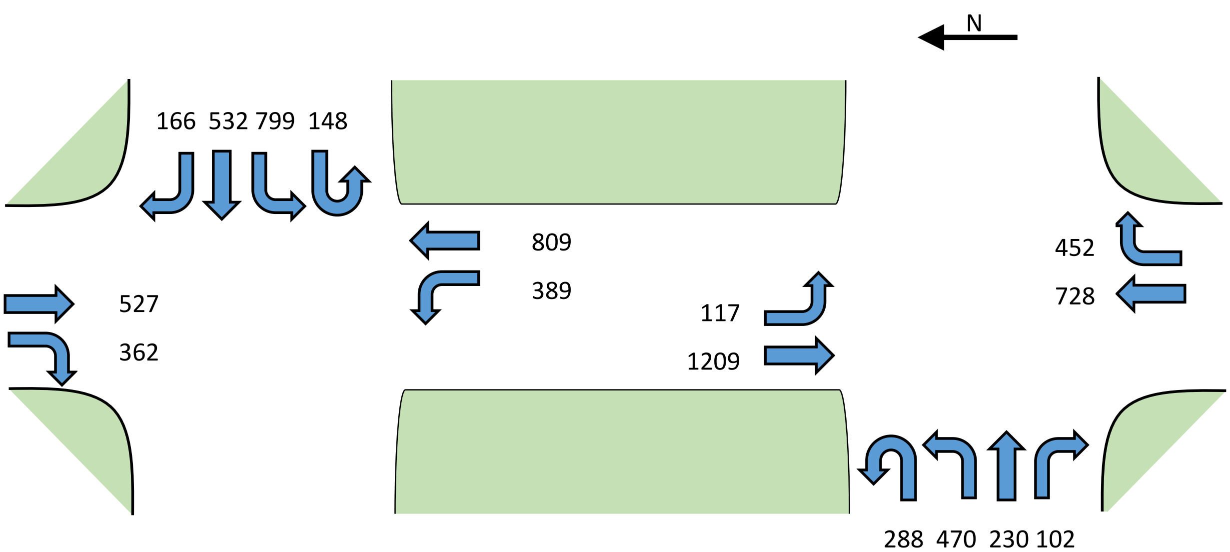
Analysts reduced, or viewed and recorded, data from each intersection to generate 15-minute counts of vehicles turning left, going through, and turning right from each approach to each interchange. U-turn counts were also obtained for the FR approaches for each interchange. As is typical in traffic engineering analysis, researchers aggregated the 15-minute counts into peak hourly counts for analysis, using the variation in traffic volume within each hour to more realistically account for real-world traffic volume fluctuations using a calculated value known as the peak hour factor.

Traffic Volume Count Data

Analysts recorded traffic count data for each of the 26 sites selected from across the state. These data consist of peak-hour volume counts for both the morning and afternoon peak hours, as aggregated from 15-minute count data. Figure 4 displays the recorded count data for a sample of the statewide interchanges studied. Appendix B has the complete list of count data for sites studied.



a. Bryan District—SH 6 @ Briarcrest (AM Peak Hour).
*Note: no direct U-turn volume count available due to lack of U-turn lane at this site



b. San Antonio District—I-410 @ Callaghan (AM Peak Hour).

Figure 4. Turning Movement Counts at Two Sample Sites.

Origin-Destination Counts

OD data are another important traffic input for evaluation of U-turn design and operations. The destination (e.g., downstream freeway entrance ramp, adjacent driveway) of the vehicles on the departure side of a U-turn will affect the ability of U-turn traffic to merge into traffic on the FR. For example, at a U-turn with Yield control and high volumes of FR traffic, an increase in U-turn traffic intending to access the nearest driveway will cause greater delay in the U-turn lane. Furthermore, on the approach to a U-turn lane, traffic from the upstream freeway exit ramp intending to make a right turn onto the cross street may create a queue in the left lane next to the U-turn lane if high volumes of FR vehicles make it difficult for ramp vehicles to change lanes to the right. For these reasons, researchers collected OD data for the U-turn departure side and the approach side. For the departure side, OD counts were recorded from different movements (e.g., the interior left turn, FR through, cross-street right turn, and the U-turn) to the downstream driveways and freeway entry ramp. For some sites with tight spacing on the departure side, traffic distributions in the FR receiving lanes were also counted. Table 13 and Table 14 show OD counts and the lane distribution counts, respectively, for the departure side at I-10 and Gessner Rd. For the U-turn approaches, percentages of traffic coming from the freeway exit ramp, if presented, were collected.

Table 13. OD Counts at I-10 @ Gessner Rd. for WB to EB U-Turn Departure Side.

Origin	Peak Hour	Destination				Total
		Freeway Entry	Frontage Road	Driveway 1	Driveway 2	
U-Turn	AM	82	84	20	3	190
	PM	107	149	46	35	337
Left Turn	AM	607	210	14	2	833
	PM	424	195	31	17	667
Through	AM	90	856	51	8	1005
	PM	184	451	43	45	722
Right Turn	AM	216	62	0	1	278
	PM	207	76	0	0	283

Table 14. Lane Distribution at I-10 @ Gessner Rd. for EB to WB U-Turn Departure Side.

Origin	Peak Hour	Lane 1		Lane 2		Lane 3		Total
		Count	Percent	Count	Percent	Count	Percent	
U-Turn	AM	208	67.3%	75	24.2%	26	8.5%	309
	PM	260	81.2%	40	12.4%	20	6.3%	320
Left Turn	AM	216	53.5%	36	9.0%	151	37.4%	403
	PM	421	46.7%	100	11.2%	379	42.1%	900
Through	AM	107	21.0%	214	42.2%	187	36.8%	508
	PM	337	36.1%	296	31.7%	301	32.2%	934
Right Turn	AM	101	26.9%	148	39.3%	127	33.8%	376
	PM	58	19.2%	69	22.6%	177	58.2%	304

U-Turn Departure Performance

U-turn performance data at the departure side were collected by recording detailed arrival time stamps from videos. The performance data included U-turn delay time, U-turn number of stops, and gap time between successive FR vehicles. These data provided critical information for evaluating the existing conditions of the study sites. Along with the gap data, the delay and stop data were used in the process of simulation evaluation for calibrating modeling parameters. They were also directly observed performance measures used for the evaluation of countermeasures implemented in the field. Table 15 shows the U-turn delay and number of stops of NB to SB U-turn departure side at I-410 @ Ingram for 15-minute intervals in AM and PM peak hours collected in July 2016. The AM peak had fewer U-turn vehicles than the PM peak, and delay and stops were generally lower in the morning than the evening.

Table 15. SB to NB U-Turn Departure Delay and Stops at I-410 @ Ingram.

	Period	No. U-turn Vehicles	Total Delay (sec)	Avg. Delay (sec/veh)	Total Stops	Avg. Stops
AM	7:00–7:15	27	12	0.4	10	0.37
	7:15–7:30	30	25	0.8	15	0.50
	7:30–7:45	48	41	0.9	22	0.46
	7:45–8:00	38	38	1.0	13	0.34
PM	17:00–17:15	81	114	1.4	42	0.52
	17:15–17:30	81	84	1.0	37	0.46
	17:30–17:45	88	77	0.9	50	0.57
	17:45–18:00	84	80	1.0	42	0.50

The collected gap data were only used directly in calibrating simulation models in this research, but they can be used as the key input data for the estimation of U-turn capacities by following the HCM (*16*) procedure. Figure 5 displays the distribution of FR gap times accepted and rejected by U-turn traffic at the NB to SB departure side at I-45 @ Research Forest. In this example, the gap threshold where more vehicles accepted the gap than rejected it was 8 seconds.

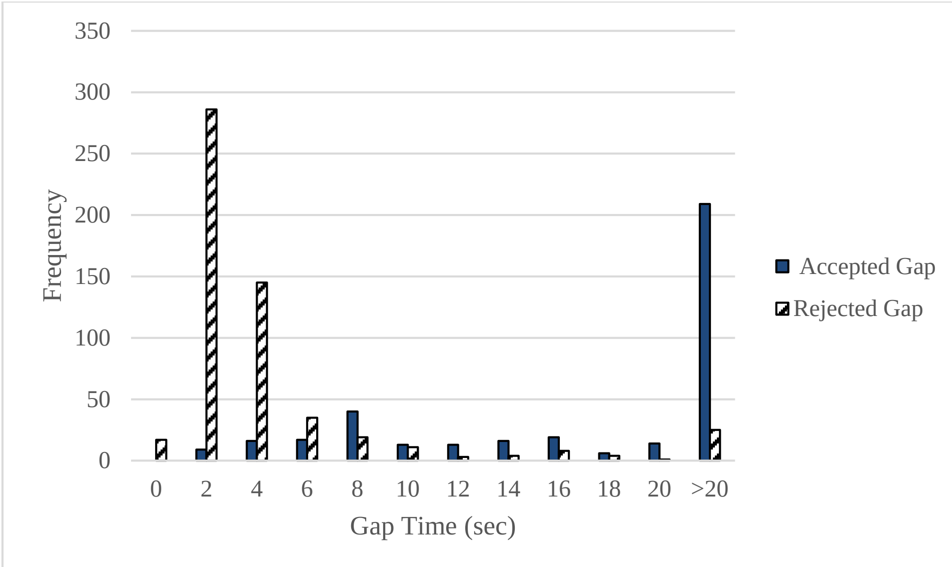


Figure 5. NB to SB U-Turn Departure Gap Time Distribution at I-45 @ Research Forest.

SIGNAL CONTROL

Following roadway geometric and traffic volume data, the final requirement for traffic operations evaluation is traffic signal settings. These data typically consist of signal cycle length, minimum and maximum phase times, clearance intervals, phasing strategy (i.e., Texas 3- or 4-phase operation), and, if the signal is coordinated with adjacent intersections, phase timing splits and timing reference offset. Such data are necessary for each signal timing plan stored within the intersection's controller unit, and there are typically at least four plans stored for each controller (AM peak, PM peak, daytime off-peak, and nighttime).

While video recording of intersections can be used to calculate signal timing settings, this process is time consuming and can suffer from inaccuracies if signals are operated in an actuated mode (where signal timing is variable, based on demand). Rather, researchers coordinated with TxDOT or municipal staff responsible for each of the study interchanges and obtained a hard copy of the controller programming/timing sheets for each interchange operated by a traffic signal (i.e., the vast majority of the study sites). Using the cycle, phase times/split, clearance intervals, and offset information on these sheets, researchers were able to enter the same signal settings used at each field site into the traffic analysis and simulation models developed for research investigation of each interchange.

CHAPTER 4. OPERATIONAL EFFECTIVENESS OF SOLUTIONS

INTRODUCTION

This chapter summarizes the activities performed in Task 3 and Task 4 of this project. The objectives of this task were:

- To apply solutions to operational issues identified at previously selected study sites.
- To evaluate the effectiveness of these solutions using VISSIM-based computer simulation by comparing performance measures for these simulations to the measures obtained in Task 3.
- To conduct field testing of identified solutions at two study sites and compare results before and after implementation.

EVALUATION CONDITIONS

Researchers evaluated U-turn operation at the statewide study sites under the existing conditions and, if determined insufficient, under conditions with selected treatments. Among the 26 study sites, the researchers were unable to obtain signal controller timing sheets for one of the signalized intersections. Therefore, 25 sites were evaluated under the existing conditions. Later, eight of these sites were re-evaluated in detail with individual countermeasures applied under computer simulation environment and/or field implementation conditions.

Base Conditions

For each study site, researchers established the operational condition with the existing characteristics as the base condition. Evaluation of the existing conditions at study sites considered two scenarios. The first scenario was with geometry, traffic, and signal control conditions remaining the same as in the field to reflect the performance of the study sites at the existing demand level. The other scheme was with the volume input varied within a reasonable range to better evaluate the study sites' performance in accommodating short-term and/or long-term changes in traffic demand. For this reason, researchers selected the following variations in traffic demand levels:

- Actual recorded (base) volumes.
- All base volumes increased by 25 percent.
- All base volumes decreased by 25 percent.
- Only U-turn base volumes increased by 25 percent.
- Only U-turn base volumes decreased by 25 percent.

Evaluation of the existing conditions provided the basis for selecting sites with inadequate performance for further application or implementation of countermeasures in terms of design and/or operation.

Conditions with Potential Countermeasures

For sites deemed in need of U-turn improvement after the evaluation of the base conditions, researchers considered individual countermeasures to be evaluated in computer simulation and/or field implementation. Researchers started with the list of possible countermeasures previously identified in Task 2 and considered a few additional potential options as well. Due to the large number of possible countermeasures and the high costs of field implementation necessary to effectively evaluate each countermeasure, a limited number of potential countermeasures were chosen for simulation and/or field evaluation. Table 16 outlines the countermeasures considered and denotes those tested through detailed simulation modeling or field implementation.

EVALUATION METHODOLOGY

Operational evaluation of U-turn lanes in this research consisted of simulation evaluation and field evaluation of existing conditions and potential solutions to improve existing conditions.

Simulation Evaluation

Traffic simulation tools provide visualizations of traffic flow on the transportation system, and such visualizations are readily adaptable for side-by-side comparison of current and proposed conditions. In addition, computing power makes it feasible to analyze a vast array of designs of different transportation alternatives to assess and maximize operational characteristics. Simulation is most useful when modeling multiple facility types where congestion is often an issue. The more complex the situation and the more detailed the results desired, the greater the advantage that simulation can have compared to theoretical methods.

The 25 statewide study sites were simulated using the microscopic simulation software PTV VISSIM, Version 8. VISSIM (a German acronym meaning “traffic in towns—simulation”) was developed to model urban traffic operations on a microscopic level based on time step and driver behavior. The program can analyze traffic conditions under any specified constraints, such as lane configuration, traffic composition, traffic signals, transit stops, and weaving behaviors, thus making it a useful tool for the evaluation of various alternatives based on transportation engineering and planning measures of effectiveness. VISSIM outputs different measures of effectiveness such as average delay, queue length, speed, and vehicle emissions that can then be used as a basis for a comparison of alternatives. The study methodology of this project complies with the principles described in FHWA’s *Traffic Analysis Toolbox III: Guidelines for Applying Traffic Microsimulation Software* (*17*).

Table 16. Potential Countermeasures to Improve Operations at Sites with U-Turns.

Countermeasure	Simulation	Field
Extending approach turn bays (for left turn, U-turn, and/or right turn).	X	
Two-lane U-turn lanes (for sites with high U-turn volume that are geometrically capable of accommodating the additional receiving lane on the destination frontage road).	X	
Adding a U-turn lane (for sites currently without a U-turn lane).	X	
Adding departure acceleration lanes.	X	
Access controls (pavement marking, raised curb, or flexible pylon) for the U-turn departure or the cross-street right turn to reduce or remove conflicts at the U-turn merge.	X	X
Signal timing adjustments to reduce queue length/delay.	X	
Signal control changes to allow for U-turns to access the FR with better gaps.	X	
Signalized control of the U-turn.	X	
Adding/removing Yield signs (R1-2) or Added Lane sign (W4-3) or the Entering Roadway Added Lane sign (W4-6) on the U-turn.	X	X
Altering left-turn "cat tracks" (dotted lines) to direct vehicles to alternate receiving lanes on frontage roads.	X	
No RTOR on cross streets to reduce conflicts between RT and UT traffic (R10-11 series signs).	X	
No RTOR Except from Right Lane sign (R10-11c) for cross streets.	X	
Access controls/driveway closures.	X	
Signage on the U-turn departure or cross-street right turn to reduce or remove their conflicts. Examples are the RTOR Must Yield to U-Turn sign (R10-30) and U-Turn Yield to Right Turn (R10-16).	X	

Model Development

The process of developing VISSIM simulation models for interchanges included developing the geometry models of roadway networks with traffic and operation input and calibrating the model parameters to match real field conditions. The inputs for the VISSIM models include three major components: roadway geometrics, traffic volumes, and signal settings. Each of these is described in detail below.

Field observations and aerial maps were used to obtain accurate geometric parameters, which are major factors affecting vehicle behavior in the model. VISSIM uses the concept of links and connectors to define the roadway network, and the links break only when necessary in cases such as the addition or subtraction of a lane due to lane drops/additions or at an intersection of roadways (e.g., on/off ramp or intersection). For each link, modelers specified details such as the number of lanes, link type, lane width, gradient, and other factors. In VISSIM, a connector is used to join links.

Traffic volume data include roadway segment (link) volume, turning movement counts at intersections, vehicle classification mix, and traffic route. In this study, the turning movement count data incorporated into the models were collected in the field between fall 2015 and spring 2016 and were used for developing the existing condition models. These turning movement values enabled the use of the traffic route choice function within the VISSIM model, and vehicles in the model utilized the arrays of routes to traverse different link sets.

Existing conditions analysis involved the coding of traffic signal phasing and timing. Researchers obtained site-specific signal control information for each site from TxDOT or the responsible municipal agency. This traffic signal information was then coded into the VISSIM models to simulate the operation of existing signalized interchanges.

Model Calibration

Two basic sets of parameters are implemented within VISSIM to control the movement of individual vehicles in the network. These are the car following and the lane changing models. VISSIM uses the models based on the continuous work of Wiedemann (*18*, *19*). The overall behavior of the model can be changed considerably by increasing or decreasing the parameters within the models. Other than changing those behavior model parameters, the local behavior parameters of gap acceptance in driver yielding situations are also important in this study. In VISSIM, yield priority rules, gap acceptance time, and headway can be changed to match real-world conditions. In this project, researchers selected several sets of vehicle trajectories from different sites and recorded the headway gap time of the headway of each trajectory. The final results provided a range within which the driver behavior model parameters were adjusted.

The calibration for a microsimulation study ultimately requires comparing simulated data with field-observed traffic data. Because the field observations vary from day to day due to the stochastic nature of traffic, the calibration objective is to reproduce the typical real-world traffic variation in the simulation. The calibration efforts are focused on the use of observed data to calibrate the most critical parameters in the VISSIM simulator.

For any simulation study, the calibration procedure is crucial. The objective of model calibration is to obtain the best match possible between model performance estimates and the field measurements of performance. The analyst needs to know when to stop the calibration effort, and this is the purpose of adopting calibration targets for the model. Calibration targets are developed based on the minimum performance requirements for the microsimulation model, taking into consideration the available data resources. The targets will vary according to the purpose for which the microsimulation model is being developed and the resources available to the analyst. Table 17 provides an example of calibration targets that meet the guidelines established in FHWA's *Traffic Analysis Toolbox III: Guidelines for Applying Traffic Microsimulation Software* (*17*).

Table 17. Example Calibration Targets.

Calibration Criteria	Calibration Acceptance Targets
Hourly volume, model versus observed	Within 100 vehicles per hour (vph) (for volumes less than 700 vph)
	Within 15% (for volumes between 700 and 2,700 vph)
	Within 400 vph (for volumes greater than 2,700 vph)
GEH statistic	Less than five for individual link flows
Travel Time for certain routes	Within 15%
Queue lengths	To analyst's satisfaction based on field observations

Performance Measures

The results of the field studies and computer simulation are used as the basis for the operations performance analysis. The goal of this analysis is to identify if and/or when a certain type of application may be more beneficial for operations.

In the process of the evaluation of alternatives using microsimulation model results, the selection and the interpretation of performance measure results is vital. For the detailed simulation analysis of U-turn operations, researchers required the following benchmarks for the measures of performance:

- The measure is able to reflect the changes of the different treatments.
- The measure is independent from other measures.
- The data collection can be accomplished in VISSIM.

Following these criteria, the measures of performance listed below were collected during the simulation:

- Measures for individual approaches (movements) of the interchange; these measures represent the performance within a given movement such as U-turns and left turns:
  - Number of vehicles performing the given movement.
  - Delay time (in seconds) that it takes to complete the movement.
  - Stop delay (in seconds) during the movement through the interchange.
  - Number of stops during the movement.
  - Average queue length (in feet) of the given movement.
  - Maximum queue length (in feet).
- System measures; all vehicles released into the interchange were recorded for their performance, including:
  - Total system travel time and delay (in vehicle hours).
  - Average speed for vehicles in the entire system (mph).
  - Average delay time per vehicle in the entire system (in seconds).
  - Average number of stops of each vehicle in system.

To account for the stochastic nature of both the traffic volumes and the traffic simulation results, each simulation model was run for at least seven complete runs (replications), each with a random and unique stochastic set of traffic demands based on a random seed which VISSIM utilizes.

Field Evaluations

Field evaluations were performed at two sites for selected treatments using before/after studies. Traffic condition data, including U-turn delay and stop data, traffic volume data, and U-turn gap data, were collected in the field using video recordings before countermeasures were implemented and collected again at least four weeks after the countermeasures were implemented. These performance parameters mirrored those collected during the detailed simulation portion of the research investigation under Task 4 and gave researchers the means to compare each before and after condition for delay and driver behavior impacts. As an example, the removal of the Yield signs may demonstrate reduced delay (based on the queue waiting time data collected from video) under the same volume conditions since drivers would have no signing-related indication that a downstream situation exists that requires them to have to observe and respond to conflicting traffic.

SIMULATION EVALUATION RESULTS

In Task 3, researchers identified 25 study sites and collected traffic data for each of these sites. Next, researchers used VISSIM simulation modeling to establish baseline performance measures for each study site and to document these performance measures for U-turn lanes operating under various scenarios. The performance measures of greatest significance are volume, queue length, average delay, and stops. In Task 4, the goal was to select several of the sites for more detailed modeling and to apply various solutions and test those solutions with VISSIM simulation modeling. During the process of creating more detailed models, researchers further calibrated the base models for selected sites so that they reflected the most accurate data available, such as the traffic lane distribution data and the gap data. The base results are provided in Appendix C. Additional simulations completed after applying individual countermeasures helped demonstrate the impacts on U-turn operation at the sites selected for detailed modeling. This analysis helped in the development of the guidelines for some of the design and operational aspects for U-turns. These simulation results from the countermeasures modeling are provided in Appendix D.

Base Modeling Results

As researchers expected, the modeling results show how the site characteristics at a particular interchange do indeed influence the performance of that interchange. The sites examined had a range of values for many of the recorded site characteristics, and these differences produced results that were examined in the analysis phase of the research.

General Site Characteristics

While many comparisons and analyses were completed on the modeling results, some particularly interesting results stand out. In comparing three sites, we can see how different geometric characteristics seem to contribute to greater queue lengths for the U-turn lane. Table 18 shows some example characteristics and results for comparison from this set of three sites.

Table 18. VISSIM Results Comparison—Set 1.

Site	**San Antonio District I-410 @ Callaghan**	**Pharr District I-2 @ SH 494**	**San Angelo District US 67 @ Loop 306**
Approach, Peak Period	Eastbound, PM	Westbound, PM	Southbound, AM
U-turn Volume (veh/hour)	392	666	247
U-turn Bay?	Yes	Yes	Yes
U-turn Widths (ft) (Begin, Mid, End)	25, 20, 22	32, 26, 36	20, 17, 21
Driveway Density (# driveways ½ mi upstream)	13	4	1
U-turn departure Geometry	Taper & merge 65 ft to end of taper	Taper & merge 150 ft to taper end	Taper & merge 149 ft to taper end
Average Queue Length (ft)	201	42	10
Max. Queue Length (ft)	779	527	123

All three of these site approaches have relatively high U-turn hourly volume during the referenced peak hour, and all of them have U-turn bays on the approach. The site with the most U-turn volume (I-2 WB @ SH 494 during PM peak) does not have the greatest queue length values. Reasons for this may include the lower driveway density on the approach, the wider U-turn lane, and the longer distance to the end of the taper at the U-turn departure acceleration lane. Signal timing features, such as cycle length, can influence queue length; however, for each of these sites, cycle length was similar. Comparable relationships are noted for the site in San Angelo.

The site at I-410 EB @ Callaghan in San Antonio is shown to have very large queue lengths, up to 779 ft. Key factors that seem to cause this larger queue length are the greater number of driveways on the approach and the much shorter length to taper at the U-turn departure. Naturally, the U-turn demand volume is also a factor.

Similar comparisons and relationships between the site characteristics and interchange performance exist for many other sites and involve a number of geometric design factors as well as development intensity and traffic volume demand. Researchers have examined these model results for additional sites to determine the factors and relationships that are most important and most influential in affecting interchange performance. Key characteristics of these two sets of sites are described in Table 19 and Table 20.

Table 19. VISSIM Results Comparisons—Set 2.

Site	Houston District I-10 @ Gessner	Houston District I-10 @ Bunker Hill	Houston District I-45 @ Rayford/ Sawdust	Houston District I-45 @ Research Forest
Approach, Peak Period	Westbound, PM	Eastbound, PM	Southbound, AM	Southbound, PM
U-turn Volume	358	609	709	514
U-turn Bay?	Yes	Yes	Yes	Yes
U-turn Widths (ft) (Begin, Mid, End)	29, 21, 29	31, 23, 31	27, 14, 26	26, 25, 24
Driveway Density (# driveways ½ mi upstream)	9	12	11	14
U-turn departure Geometry	Yield	Add Lane	Add Lane	Yield
Average Queue Length (ft)	1544	732	24	39
Max. Queue Length (ft)	1665	1532	160	283

The four sites in the Houston District shown in Table 19 all have U-turn bays and have varying U-turn volumes; however, the queue lengths are not proportional to the volumes. The two sites on I-45 have much lower queue lengths than the two sites on I-10, and the site with the lowest peak U-turn volume has the highest queue lengths. It is likely that the yield control on the Gessner site has some effect on the queue length, but that by itself does not explain the relationship since the Research Forest site also has yield control but much shorter queues. Driveway densities upstream of the U-turn bays are very similar, so the effect of that variable seems to be minimal among these four sites.

Table 20. VISSIM Results Comparisons—Set 3.

Site	Ft. Worth District I-20 @ Hulen	Ft. Worth District I-20 @ McCart	Bryan District SH 6 @ Briarcrest	San Antonio District I-410 @ Ingram
Approach, Peak Period	Eastbound, AM	Eastbound, AM	Northbound, AM	Southbound, PM
U-turn Volume	199	345	94	241
U-turn Bay?	Yes	Yes	No	Yes
U-turn Widths (ft) (Begin, Mid, End)	23, 18, 19	22, 18, 18	N/A	34, 15, 23
Driveway Density (# driveways ½ mi upstream)	6	14	2	4
U-turn departure Geometry	Stop	Yield	N/A	Add Lane
Average Queue Length (ft)	28	26	81	22
Max. Queue Length (ft)	183	206	342	243

The four sites in Table 20 all have lower U-turn volumes than the four sites in Table 19. As expected, their queue lengths are relatively low, though not necessarily as low as the I-45 sites in Table 19. The SH 6 @ Briarcrest site does not have a U-turn bay, which helps to explain the longer queue length. The remaining three sites have somewhat similar characteristics and resulting queue lengths.

An examination of the average and maximum queue lengths of U-turn traffic in comparison with the storage space and the adjacent left-turn queuing conditions at the U-turn approach side reveals that the following sites and conditions may be operating close to or over the U-turn capacity:

- I-20 @ Hulen: WB to EB U-turn during with all volume increased by 25 percent in PM.
- I-410 @ Ingram: U-turn in both directions with all volume increased by 25 percent in PM.
- I-410 @ Callaghan: PM base condition.
- US 82 @ Lawrence: PM base scenario.
- I-10 @ Gessner: WB to EB U-turn during PM base condition.
- I-10 @ Bunker Hill: EB to WB U-turn during PM base condition.
- I-45 @ Research Forest: SB to NB U-turn with U-turn volume increased by 25 percent during PM peak hour.
- I-45 @ Rayford/Sawdust: NB to SB U-turn in both AM and PM conditions and SB to NB U-turn during PM.

Traffic Characteristics

Hand-in-hand with site characteristics in determining the need for and features of U-turn lanes are the potential traffic demands for those lanes. Further, traffic demand for most of the other movements within an interchange can also have direct and indirect impacts on U-turn operation. A number of research analyses were conducted using the field data collected at each of the 25 project study sites to identify relationships between traffic demand levels, the interchange geometric features designed to meet those demands, and the collective impacts of demand, capacity, and facility design on U-turn performance within interchanges.

Researchers generated several descriptive statistics for the overall dataset used to analyze U-turn performance at the 25 field sites. Demand data, such as interchange peak-hour volume and directional U-turn peak-hour volume, were directly measured and recorded by analysts in the field. U-turn geometric data, such as the length of U-turn bays, were measured using geographical information system (GIS) tools and up-to-date aerial imagery. Performance data, such as peak-hour U-turn delay and interchange delay, could not be readily and cost-effectively measured in the field. These measures were derived from VISSIM traffic simulation models of the exact geometric, traffic control (signal operations, etc.), and demand conditions at each interchange. General statistics about the range of field sites used in the current research are found in Table 21.

Table 21. Descriptive Statistics for the 25 U-Turn Study Sites.

Feature	**Units**	**Average**	**85^{th} Percentile**
Interchange Peak-Hour Volume	vph	4053	5989
Interchange Peak-Hour Delay	sec/veh	37.0	49.4
U-Turn Peak-Hour Volume	vph	207	386
U-Turn Peak-Hour Delay	sec/veh	17.2	52.3
U-Turn Bay Length	ft	265.6	369.0
Frontage Left-turn Peak-Hour Volume	vph	388	698
Frontage Left-Turn Average Queue Length	ft	87.4	296.6
Frontage Left-Turn Maximum Queue Length	ft	126.9	416.0

Overall Volumes

Perhaps the most fundamental relationships defining the need for U-turn lanes are those based on demand, both for the overall interchange and the U-turn movement itself. Figure 6 presents the relationship between demand (peak hour volume) and average vehicular delay for all 25 of the field sites investigated in this research investigation. Expected trends, such as increasing delay with increasing volume, are directly and readily apparent. It is also clear that the field sites selected for the study represent a broad range of the conditions found in rural and urban areas across Texas; the peak-hour volumes entering the interchanges range from just over 500 vph to almost 8000 vph.

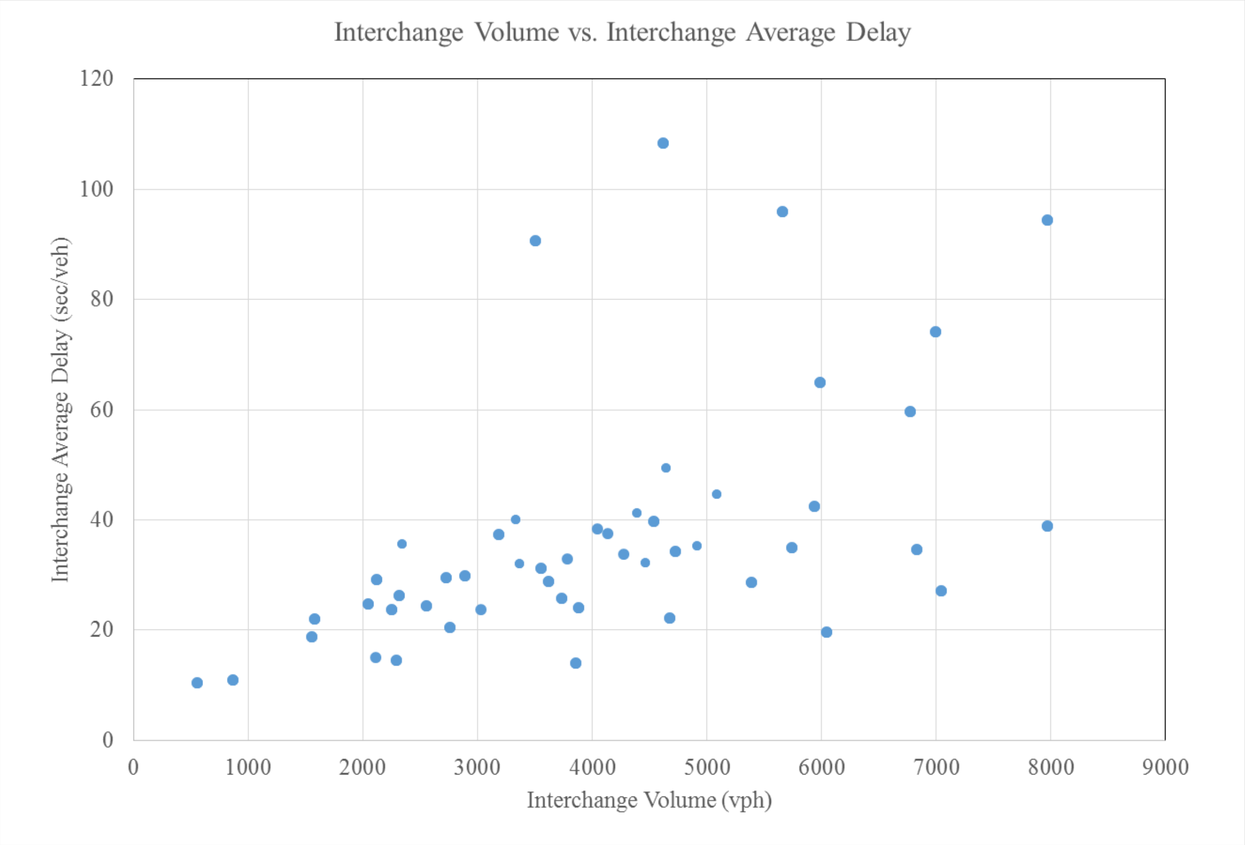


Figure 6. Field Study Sites—Interchange Volume vs. Interchange Performance.

In addition to higher delays associated with higher volumes, it is also clear that some interchanges experience high levels of delay in the peak hour. The HCM (*16*) identifies 80 seconds per vehicle (sec/veh) as the threshold for LOS F or overcongested/overcapacity conditions at signalized intersections. Four of the 25 study sites fall into the highest level of congestion specified in the manual, and an additional three field sites are approaching this congestion level. In reviewing these relationships, note that congestion can occur at even moderate traffic demand levels; interchanges with fewer approach lanes and/or those interchanges lacking U-turn lanes would be substantially more susceptible to congestion under these conditions.

A final observation from Figure 6 is that the average delays represent all movements within the interchange in aggregate; however, U-turn lanes may not experience the same level of delay as other interchange users, depending on interchange design. As shown in Figure 7, there is no consistent correlation between interchange delay and U-turn delay. High U-turn delays are observed to occur when overall interchange delays are moderate to very high, but the same levels of interchange delay also show very low U-turn delays (at other interchanges in the study site dataset). The lack of a predictable relationship in performance between the interchange and U-turn lane(s) suggests that the need for U-turn lanes, and their ultimate performance if installed/present, is dependent on complex interactions between demand, geometric design, and traffic control.

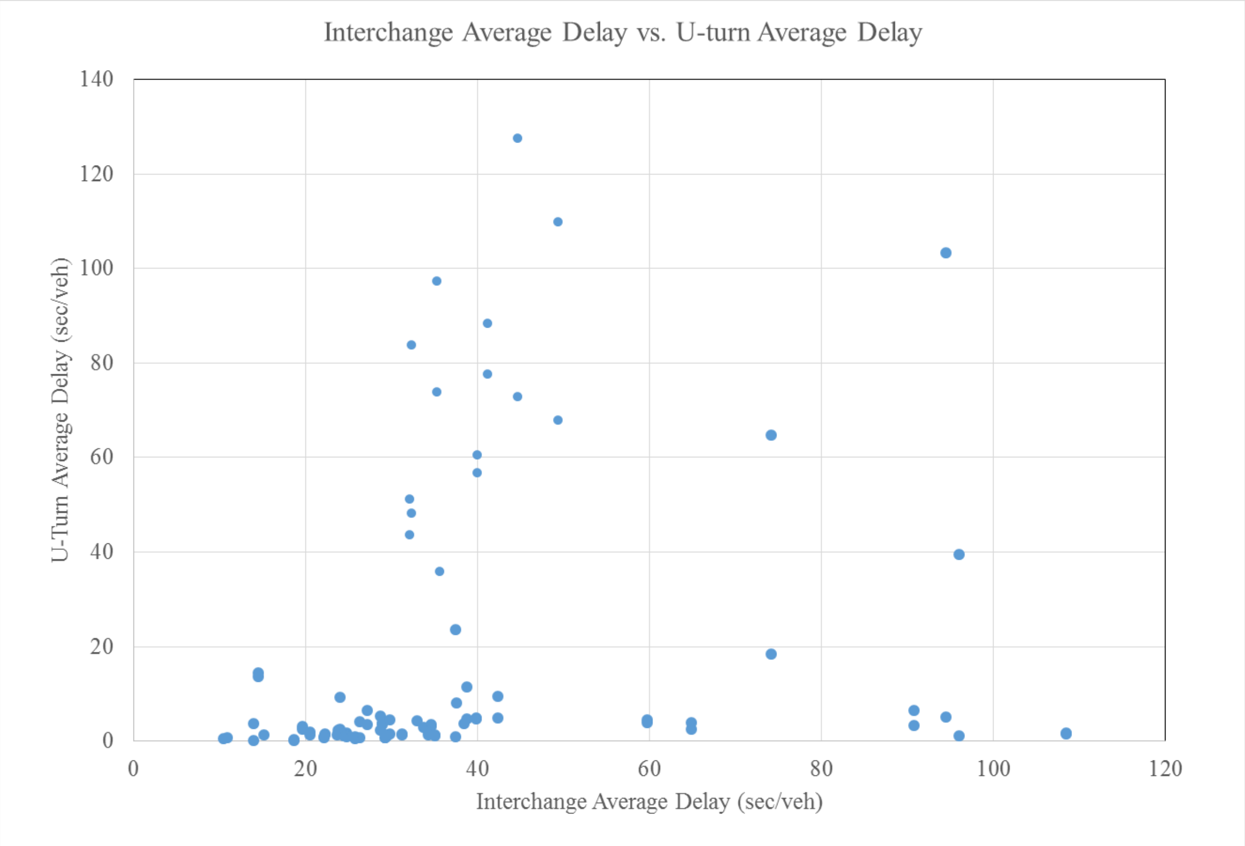


Figure 7. Relationship between Interchange and U-Turn Performance.

The analysis supporting Figure 8 was undertaken to further explore the relationship between interchange overall demand and U-turn performance. If all data points in the figure are considered, the rough relationship is (as expected) an increase in U-turn delay with increasing interchange volume. However, this relationship can be characterized as weak since high interchange volume is tied to sites with both low and very high U-turn delays.

By breaking the dataset into subsets with (blue data points) and without (orange data points) U-turn lanes, substantial findings are produced. Sites without U-turn lanes consistently have substantially higher U-turn delay than sites with U-turn lanes, and for this subset of the field study, a distinct linear (or nearly linear) relationship exists between interchange volume and U-turn delay. Best-fit linear trendlines are included in Figure 8 for each of the subsets. Using HCM LOS C (a typical performance target level for design) criteria for delay at 35 sec/veh, a general guideline emerges that suggests U-turn lanes become integral to interchange design at a peak-hour volume of 2000 entering vph, or roughly 20,000 entering ADT (assuming a generic K-factor of 0.10).

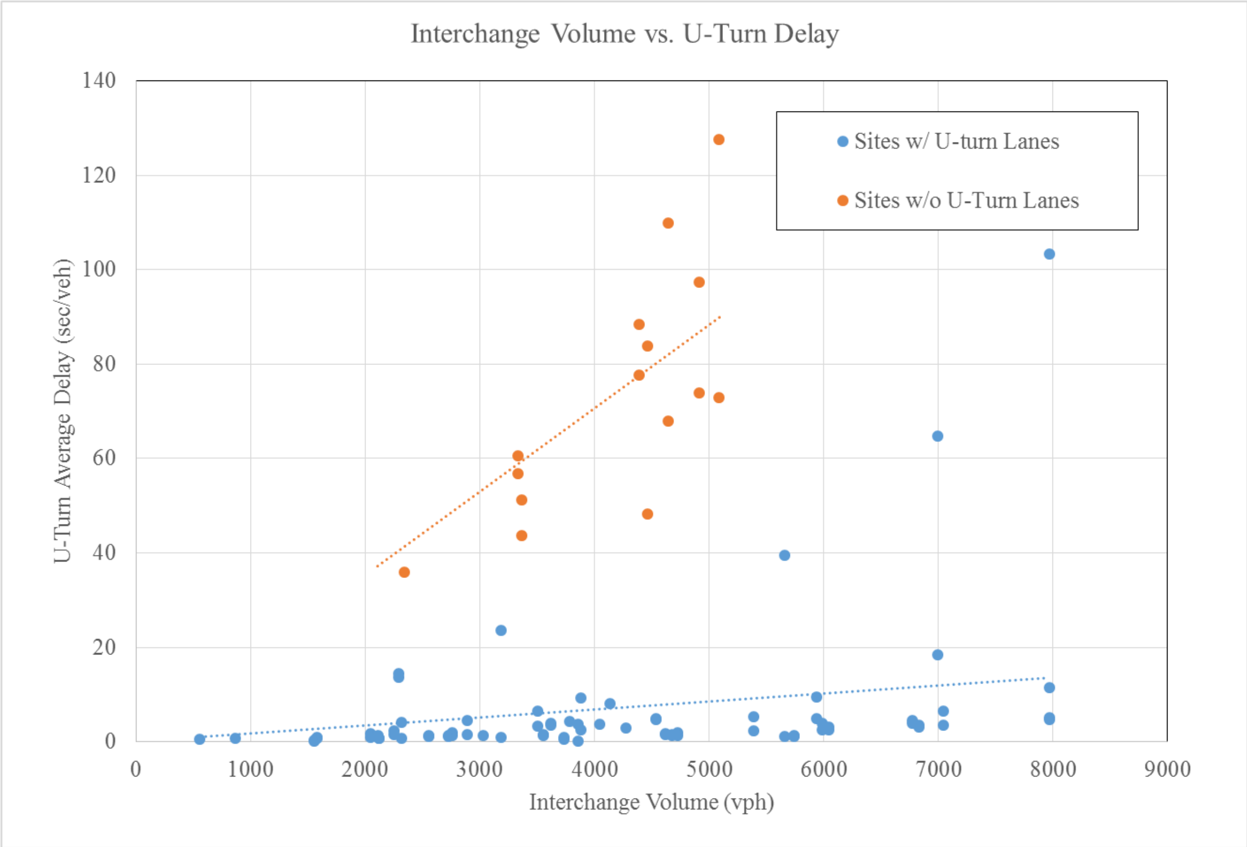


Figure 8. Relationship between Interchange Volume and U-Turn Performance.

Researchers explored a final demand relationship between U-turn peak-hour volume and U-turn performance. The full dataset was again broken into subsets for locations with and without U-turn lanes, as depicted in Figure 9. Sites without U-turn lanes are consistently lower-volume sites and feature high delay relative to sites with U-turn lanes. Average delays were calculated for sites with (6.2 sec/veh) and without (73.0 sec/veh) U-turn lanes; sites with U-turn lanes had only 8.5 percent of the delay of their counterparts without U-turn lanes (i.e., a delay reduction of 91.5 percent). The conclusions drawn from this analysis are that:

- U-turn lanes are an effective treatment for reducing delay to turnaround movements within urban interchanges.
- Delay-based justification for constructing U-turn lanes exists at even low U-turn volumes.
- Well-designed U-turn lanes (i.e., with adequate approach bays and departure lanes) can service high U-turn demand at low levels of U-turn delay.

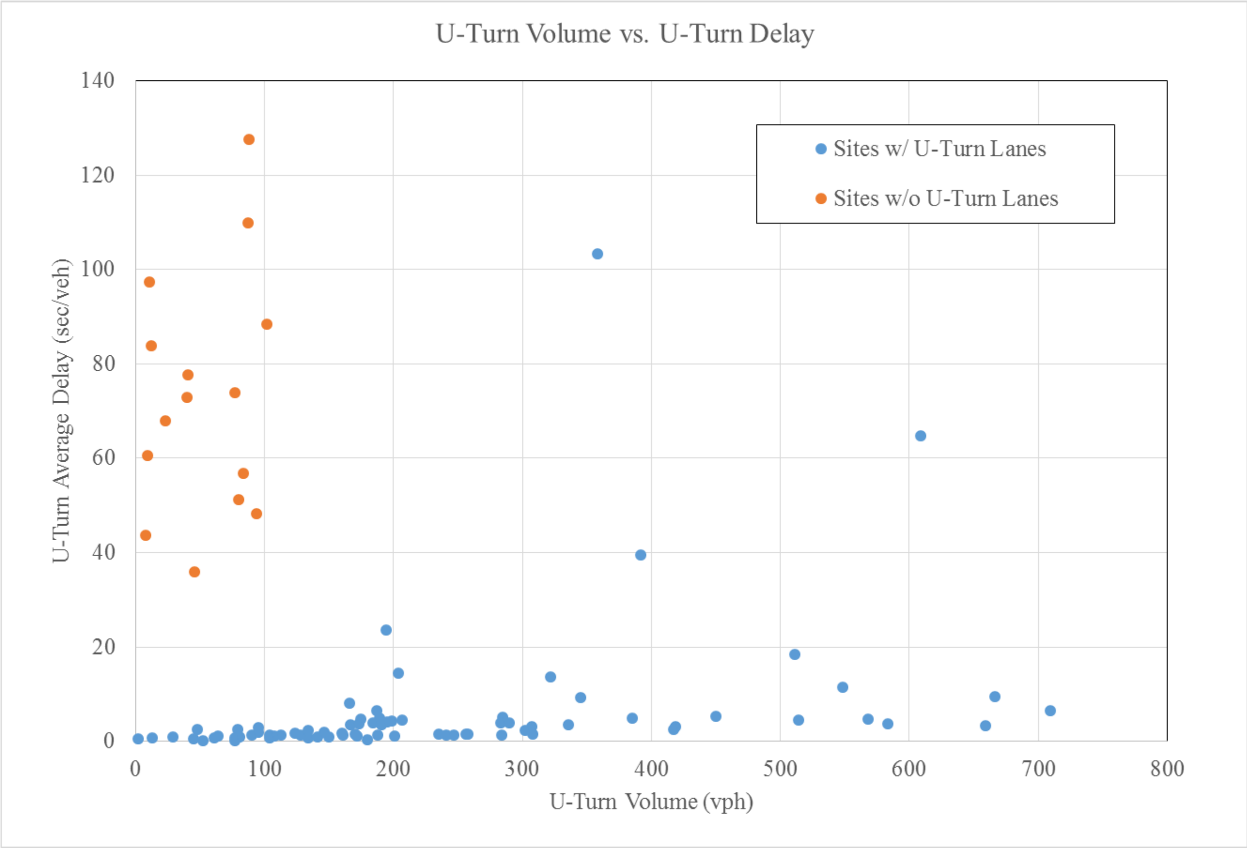


Figure 9. Relationship between U-Turn Volume and U-Turn Performance.

U-Turn Volume versus Left-Turn Volume on Approach Frontage Road

The relationship between U-turn volume and left-turn volume along an FR approach to an interchange is important based both on demand and geometric design. For instance, high left-turn demand may result in queues along the FR that block U-turning vehicles from accessing the U-turn bay. Conversely, high FR volumes on the departure side of a U-turn may cause queuing into the approach/entering side of the U-turn lane, slowing both U-turning and left-turning vehicles on the approach to the interchange signal and possibly even resulting in a U-turn queue that spills back into a lane servicing both U-turns and left-turning vehicles.

Figure 10 displays the relationship between FR left-turn peak-hour volume and U-turn movement delay through the interchange. While the overall trend is increasing U-turn delay as the volume of left-turning vehicles on the same FR approach increases, this relationship is considered weak since high left-turn volumes are associated with both low and high U-turn delays. A very low correlation coefficient, R^2, of 0.0834 was calculated between left-turn volume and U-turn average delay. However, a much stronger relationship ($R^2 = 0.5346$) exists between the left-turn queue length on the frontage approach and the quantity of U-turn average delay, as shown in Figure 11. This relationship suggests that the length of the U-turn bay, the amount of storage for left-turning vehicles, and the efficiency of interchange signal operations in limiting the left-turning queue all play a role in efficient U-turn operations.

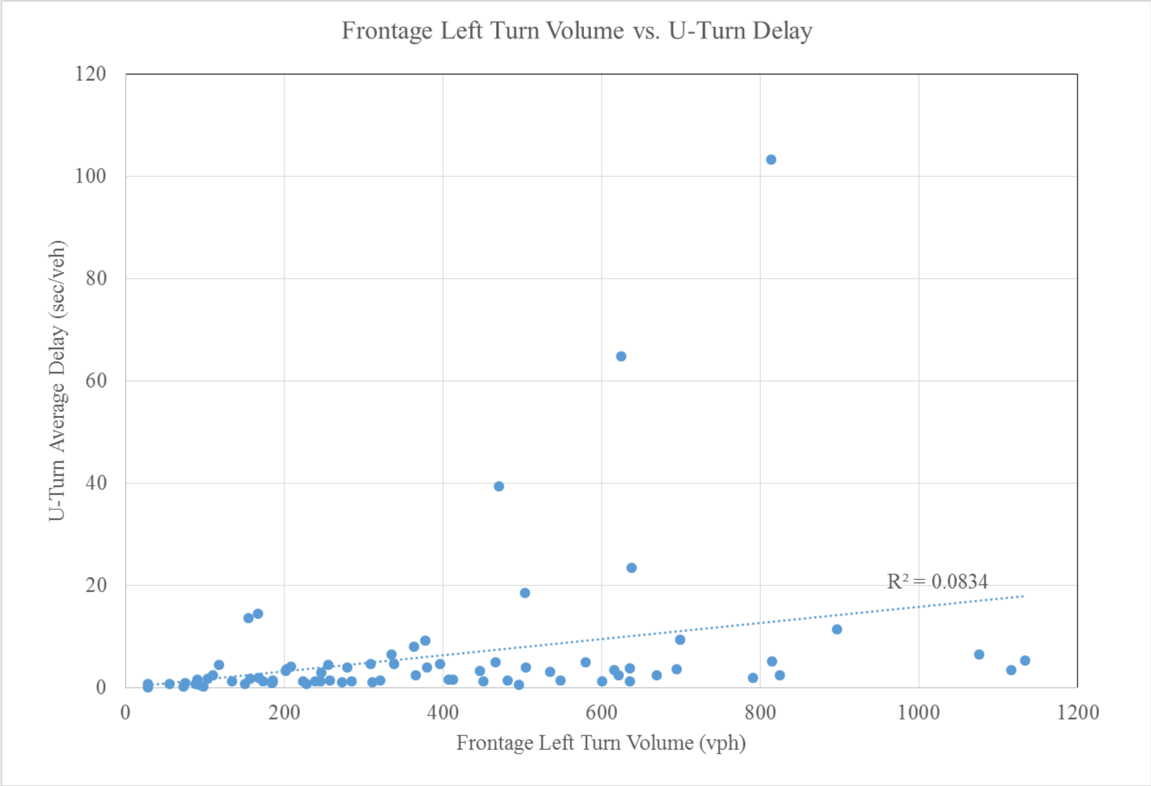


Figure 10. Relationship between Frontage Left-Turn Volume and U-Turn Performance.

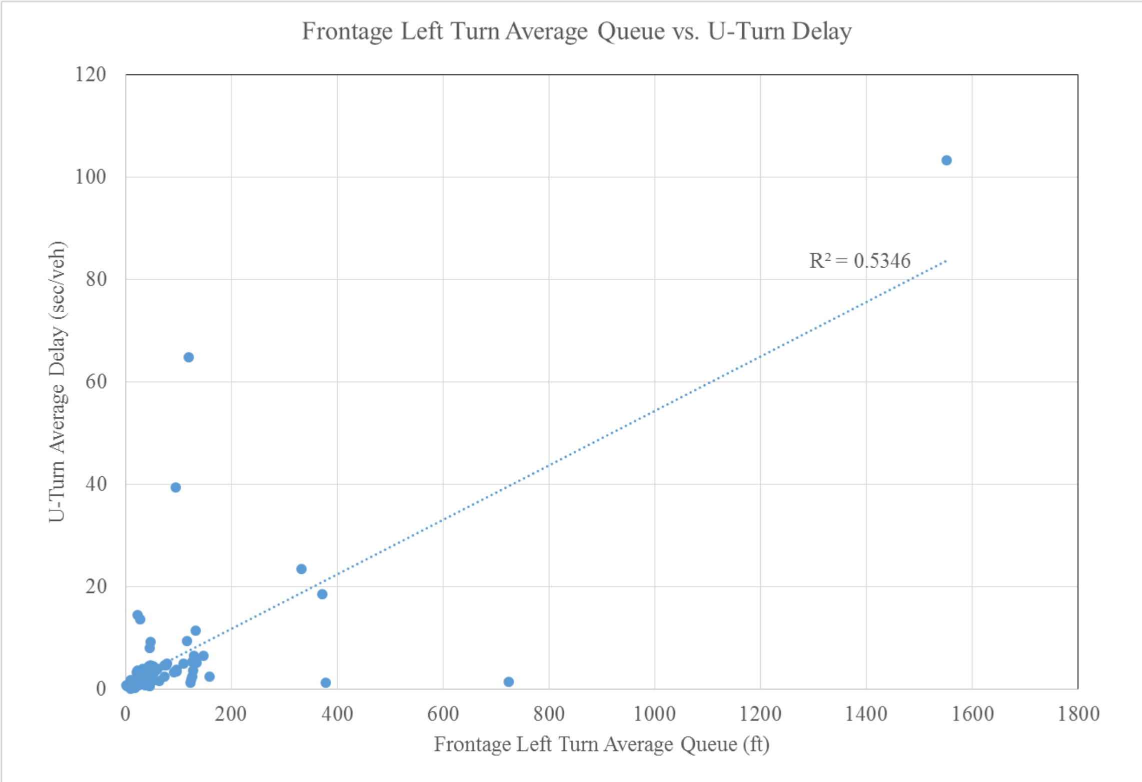


Figure 11. Relationship between Frontage Left-Turn Queue and U-Turn Performance.

Volumes Conflicting with U-Turn Departures

Among the more complex relationships documented from the study of field sites around Texas is the correlation between those movements through the interchange that result in traffic conflicting with U-turning vehicles as they depart from a U-turn lane onto the receiving FR. Movements that conflict with U-turn departures include FR through volume, arterial left-turning traffic, and arterial right-turning traffic. Field data for the study sites with U-turn lanes were subdivided into each of these movements to shown how well they associate with U-turn average delay, as shown in Figure 12.

Researchers developed exponential best-fit lines for each of the sources of volume that conflict with U-turn lane departing vehicles, including the total volume of conflicting vehicles. Correlation coefficients for each of these conflicting volume sources show that FR through volume is the individual source most associated with U-turn delay, while the total conflicting volume has the highest overall correlation with U-turn delay. However, all of the conflicting volumes—including the total conflicting volume—are relatively poor predictors of U-turn delay, as shown by the low correlation coefficients (R^2 values of less than 0.3, where an R^2 value of 1.0 indicates a perfectly direct relationship).

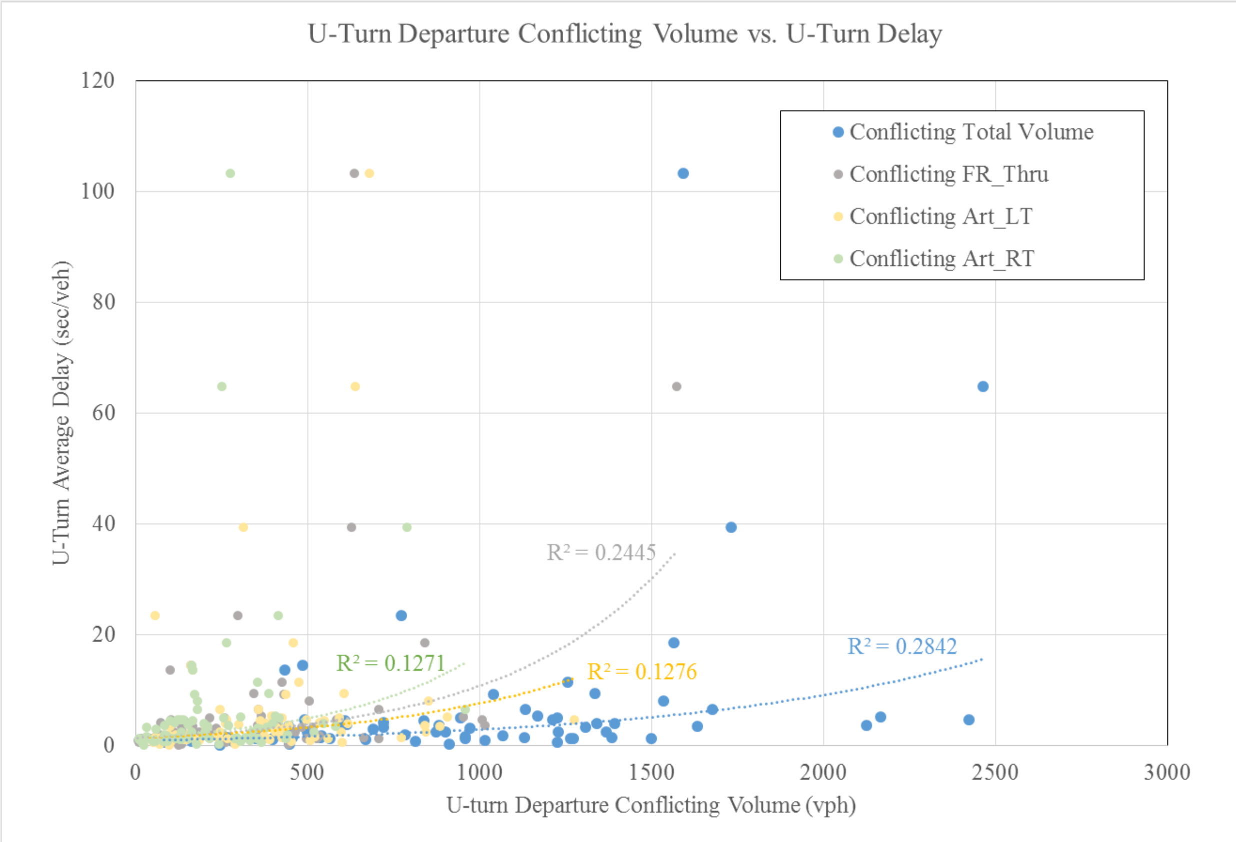


Figure 12. Relationship between Conflicting Volumes and U-Turn Performance.

Simulation Evaluation of Potential Countermeasures

After evaluating the base conditions of the 25 sites, researchers selected eight sites for detailed evaluation of the 14 potential countermeasures. For sites with multiple countermeasures, their isolated and combined effects were measured, when possible. For instance, if two countermeasures (e.g., A and B) were to be evaluated at a site, it was desirable to evaluate the impacts of countermeasure(s) A, B, and A + B for each demand level. In some cases, it may have been sufficient to evaluate a carefully chosen subset (e.g., A or B and A + B). To be consistent with previous work performed during this project, the average of each performance measure from seven replications of simulation runs was used.

Study Sites with Simulated Countermeasures

The eight sites chosen for detailed modeling were selected to reflect various site characteristics, including traffic volumes and geometric configurations, and these sites are listed in Table 22. Analysts reviewed the site characteristics and also the base model results for these sites and selected appropriate countermeasures to be applied at these sites. These countermeasures applied at individual sites are also listed in Table 22.

Just as was done for the base models, researchers used VISSIM simulation modeling to produce performance measures for each of these eight study sites and to document these performance measures for U-turn lanes operating under various scenarios. As mentioned earlier, the performance measures of greatest significance are volume, queue length, average delay, and stops.

Table 22. List of Eight Sites Chosen for Detailed Modeling.

Site	Simulated Countermeasures
SH 6 @ Briarcrest	Protected-Permissive interior left turn; added U-turn lane.
I-20 @ Hulen	Extended approach turn bays; RTOR restriction; driveway closure; RTOR yield to U-turn.
I-20 @ McCart	Added U-turn lane; RTOR restriction; driveway closure.
I-10 @ Gessner	Extended approach turn bays; added acceleration lane/bays; RTOR restriction; driveway closure.
I-10 @ Bunker Hill	Added acceleration lane/bays.
I-45 @ Rayford/Sawdust	Separation from conflicted traffic.
I-45 @ Research Forest	Signal timing adjustment; altering cat track for interior left turn; RTOR restriction.
I-410 @ Ingram	Extended approach turn bays; dual U-turn lane; signalized U-turn; adding signs; RTOR restriction; driveway closure.

Simulation Results for Potential Countermeasures

To effectively model the potential countermeasures listed in Table 22, researchers modified VISSIM parameters such as OD patterns at the U-turn departure side, lengths of links or connectors, priority rules, signal controller settings, etc., to reflect the changes in U-turn traffic caused by the countermeasures. The following sections describe the modeling techniques and evaluation results for individual countermeasures.

Extending Approach Turn Bays

The high FR volumes on the departure side of a U-turn may cause queuing into the approach/entering side of the U-turn lane, slowing both U-turning and left-turning vehicles on the approach to the interchange signal and possibly even resulting in a U-turn queue that spills back into a lane servicing both U-turns and left turns. On the other hand, high left-turn volume on the approach side of the U-turn lane may block the entry to the U-turn lane if the left-turn bay has limited storage space. Researchers evaluated this countermeasure at three sites: I-10 @ Gessner, I-20 @ Hulen, and I-410 @ Ingram.

At the I-10 @ Gessner site, westbound and eastbound traffic were experiencing long queues and high delays, especially during the PM peak hour. These delays were because high volumes of left-turn traffic often blocked the U-turn vehicles from entering the U-turn lanes due to the limited storage spaces for left-turn lanes/bays. Researchers extended the left-turn lane/bay along with the U-turn bays by 100 ft in both directions. Table 70 and Table 71 in Appendix D show the simulation results for the AM and PM peak hours. Extending turning bays significantly reduced westbound U-turn delay but only slightly reduced left-turn delay. This countermeasure did not reduce queue lengths significantly.

At the I-20 @ Hulen site, both eastbound and westbound U-turn bays were extended 300 ft to provide sufficient length for U-turn traffic to access their bay without interference from the left-turn or through queues from the adjacent lanes. Table 72 and Table 73 in Appendix D show the VISSIM results of extending the U-turn bay countermeasure for the I-20 @ Hulen site. The results indicate that the 300-ft U-turn bay extension did not significantly reduce the delay and queue length on either side's U-turn movement or the adjacent left-turn and through movements on the same side of the U-turn. The average delay reduction was 0.2 to 0.3 sec depending on the peak period and the direction, and the change on the queue length was almost negligible for each scenario.

At the I-410 @ Ingram site, the available ramp configuration (e.g., upstream freeway exit ramps on both sides are far away from the interchange) provided the opportunity for testing longer turn bays. Both northbound and southbound U-turn bays were extended 500 ft to provide sufficient length for U-turn traffic accessing their bay without the interference from the left-turn or through queue from the adjacent lanes. Table 74 through Table 77 in Appendix D show the VISSIM results of extending the U-turn bay countermeasure for the I-410 @ Ingram site. No consistent improvement of delay and queue length on the U-turn movement was observed on either side of the interchange. The only improvement from the 500-ft U-turn bay extension was from the southbound U-turn bay extension during the PM peak hour; the approach as a whole seemed to slightly improve compared to the base condition. With the higher U-turn traffic volume and interchange traffic demand in the PM peak hour, the queue length and the average delay for the other movements in the same approach benefited from the extension; however, the scale of the improvement was not significant.

As shown in Table 74 and Table 75 in Appendix D VISSIM results, the U-turn movements and the other movements from the same approaches have low average delay and queue length under the existing traffic volumes, and this factor could be part of the reason why the observed improvement of the U-turn bay extension on either side of the interchange was insignificant. To better evaluate the performance of countermeasures under different demand levels, researchers increased the traffic volumes on the FR by 25 percent, and these performance results are listed in Table 76 and Table 77 in Appendix D. The results indicated that during the PM peak hour and under the increased FR traffic volumes, extending turning bays significantly reduced U-turn delay by 34 percent and 58 percent for northbound and southbound U-turns, respectively. This countermeasure also reduced queue lengths slightly for both directions. No consistent improvement of delay and queue length on the U-turn movement was observed on either side of the interchange during the AM peak hour because the delay and queue length are still low even under the elevated travel demand levels. These results may indicate that the overall benefit from this improvement is highly sensitive to the level of the FR travel demand.

Dual U-Turn Lane

For sites with high U-turn volume, a two-lane U-turn facility may be considered. This countermeasure requires the additional available space if the freeway crosses with an overpass and may require bridge widening if the freeway crosses with an underpass. On the departure side of the U-turn, one additional receiving lane is required to accommodate the added U-turn lane.

At the I-410 @ Ingram site, a dual U-turn lane was modeled for both northbound and southbound directions. One additional FR lane was used to receive the added U-turn lane, and it was assumed that the added FR lane merged at the location further downstream. It was also assumed that on the U-turn lane departure side, the traffic on the inside U-turn lane moves freely and the traffic on the outside U-turn lane follows the same behavior mode as of those using the single U-turn lane. Tables Table 78 through Table 80 in Appendix D show the VISSIM results. The results indicate that the measure did not significantly improve the operation in the morning peak hour when the traffic is relatively light; the delay and number of stops for the U-turn movement only slight improved. During the PM peak hour, the results indicated the relatively greater improvement on the U-turn delay and queue length for both directions. In the southbound direction, the maximum queue length and the average delay for the other movements in the same approach were reduced in the PM peak hour since the traffic demand (especially the U-turn traffic volumes) is higher. However, in general, the scale of the improvement was not significant in any scenario. In Table 78 and Table 79 in Appendix D, the results of queue length and delay for U-turn movements record the performance for each of the entire movements, including the vehicular operation at the approach side and departure side of the U-turn. To separate the effect of the improvement on the departure side from the possible high left-turn and through volumes on the approach side blocking the entry to the U-turn lane, the additional queue lengths were measured on the U-turn departure for both directions. The results from Table 80 indicate some improvement from the dual U-turn lane during both AM and PM peak hours. The benefit is more significant during the PM peak hour when the high FR volumes on the departure side of a U-turn may cause more stops and longer queuing into the U-turn lane.

Adding U-Turn Lane

U-turning vehicles adversely affect the capacities of signalized intersections. U-turn lanes at diamond interchanges not only reduce delay for U-turning traffic, but also free up capacity for all other traffic passing through the interchange. The scenario of adding a U-turn lane for sites currently without one was tested at two sites: I-20 @ McCart and SH 6 @ Briarcrest.

At the I-20 @ McCart site, the U-turn lane does not exist in the westbound direction since the U-turn traffic is low due to the discontinued FR upstream. Adding the U-turn lane will re-direct the westbound U-turn traffic from going through the signalized intersection to instead using the U-turn lane.

Table 23 (and Table 81 in Appendix D) shows the VISSIM results of adding a U-turn lane for the I-20 @ McCart site for the AM peak hour. Results for the PM peak are in Table 82 in Appendix D. The results indicate that the operation of the westbound U-turn traffic was significantly improved. This improvement was expected because the delay difference is effectively the average intersection delay experienced traveling through the signalized intersections of the interchange. As a whole, the westbound approach gained slight improvement due to freed up capacity from the U-turn traffic to all other traffic passing through the interchange. The benefit of adding the U-turn lane may be more significant at locations where the U-turn traffic volume is higher.

Table 23. VISSIM Evaluation Results for Countermeasure of Adding a U-Turn Lane to Westbound at I-20 @ McCart AM Peak Hour.

Base Condition															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	360	370	718	247	211	102	195	120	262	239	29	621	148	187	**3809**
Avg. Queue Length (ft)	89	89	11	79	79	3	27	51	51	66	85	85	85	85	**48**
Max. Queue Length (ft)	514	514	392	287	287	123	187	187	187	208	322	322	322	322	**517**
Avg. Delay (sec/veh)	47.9	44.7	7.4	50.7	46.9	18.3	4.1	46.6	50.3	6.1	45.4	42.8	38.4	3.4	**31.2**
Stopped Delay (sec/veh)	34.0	33.7	2.3	39.0	36.5	13.5	1.4	40.8	42.3	2.8	36.9	34.3	30.2	0.8	**23.4**
Avg. Stops (stops/veh)	0.97	0.88	0.29	0.90	0.86	0.51	0.30	0.85	0.89	0.41	0.97	0.91	0.84	0.14	**0.68**
Add U-Turn (Westbound)															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	359	371	718	246	209	102	195	117	258	239	30	633	150	187	**3812**
Avg. Queue Length (ft)	89	89	11	78	78	2	27	51	51	66	40	80	80	80	**42**
Max. Queue Length (ft)	522	522	370	292	292	95	202	202	202	224	283	283	283	283	**523**
Avg. Delay (sec/veh)	48.1	43.6	7.2	49.9	47.4	17.8	4.1	47.8	49.6	5.7	6.2	42.3	39.4	3.7	**30.6**
Stopped Delay (sec/veh)	34.4	32.6	2.1	38.1	36.9	13.2	1.4	42.0	41.7	2.6	0.6	33.9	31.0	0.9	**22.9**
Avg. Stops (stops/veh)	0.96	0.88	0.31	0.90	0.86	0.47	0.31	0.85	0.89	0.38	0.19	0.91	0.85	0.17	**0.67**

For the SH 6 @ Briarcrest site, a U-turn lane was added for northbound U-turn traffic. Table 83 in Appendix D shows the results of the VISSIM experiment for the AM peak hour. As expected, the countermeasure greatly reduced northbound U-turn delay and average queue length. Northbound frontage movements and the westbound arterial left-turn movement were also slightly improved because of the separation of U-turn traffic from those traffic flows at the two signals. However, U-turn traffic merging to the southbound FR at the departure end may have slightly increased the maximum queue length to the southbound through traffic.

Adding Acceleration Lane or Bays

At the I-45 @ Research Forest site, the northbound to southbound U-turn lane has a Yield departure type without any driveways in the near vicinity (the nearest driveway opening is 812 ft downstream from the U-turn curb gore). The southbound to northbound U-turn lane also has a Yield departure type, but the nearest driveway is located much closer (at 150 ft downstream). Researchers conducted sensitivity analyses at this site to evaluate the effectiveness of U-turn departure acceleration lanes considering factors of (a) length of the acceleration lane and (b) with or without driveways in the vicinity. An acceleration lane was added to each U-turn departure side, with its length varying from 50 ft to 400 ft with 50 ft increments (measured from U-turn curb gore to the end of full lane width of the acceleration lane). Two hypothetical locations of the driveway at the southbound to northbound U-turn departure side were considered—50 ft upstream and 50 ft downstream of existing locations, respectively. This experiment was to investigate if the effectiveness of an acceleration lane depended on the location of the nearest driveway.

Figure 13 shows the NB to SB U-turn delay varied by the length of acceleration lane. U-turn delay was reduced by 58 percent and 60 percent during AM and PM peak hours, respectively, with the 50-ft acceleration lane. With the 100-ft acceleration lane, U-turn delay was further reduced by 24 percent and 23 percent during the AM and PM peak hours. U-turn delay did not vary much with further increased length of acceleration lane. Simulation results did not provide clear evidence that the provision of an acceleration lane had a positive or negative impact on frontage traffic with any of the considered length.

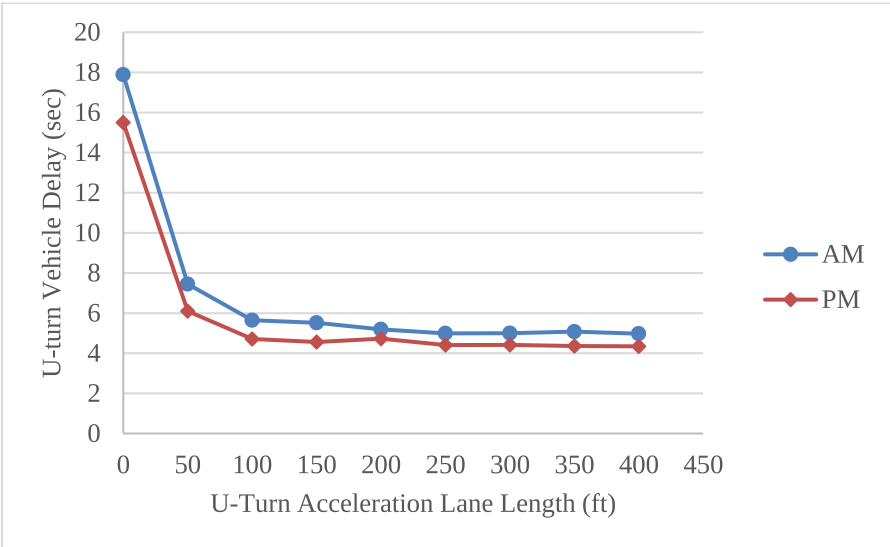


Figure 13. NB to SB U-Turn Delay Varied by Length of Acceleration Lane at I-45 @ Research Forest.

Figure 14 shows the SB to NB U-turn delay varied by length of acceleration lane with the nearest driveway at different locations during the PM peak hour. The provision of a 50-ft acceleration lane reduced U-turn delay by more than 65 percent for all driveway locations. With a 100-ft acceleration lane, U-turn delay was further reduced by more than 15 percent across all driveway locations. Further increasing the length of acceleration lane did not noticeably improve the U-turn delay. When comparing results of different nearest driveway locations, the higher U-turn delay was associated with the closer nearest driveway location.

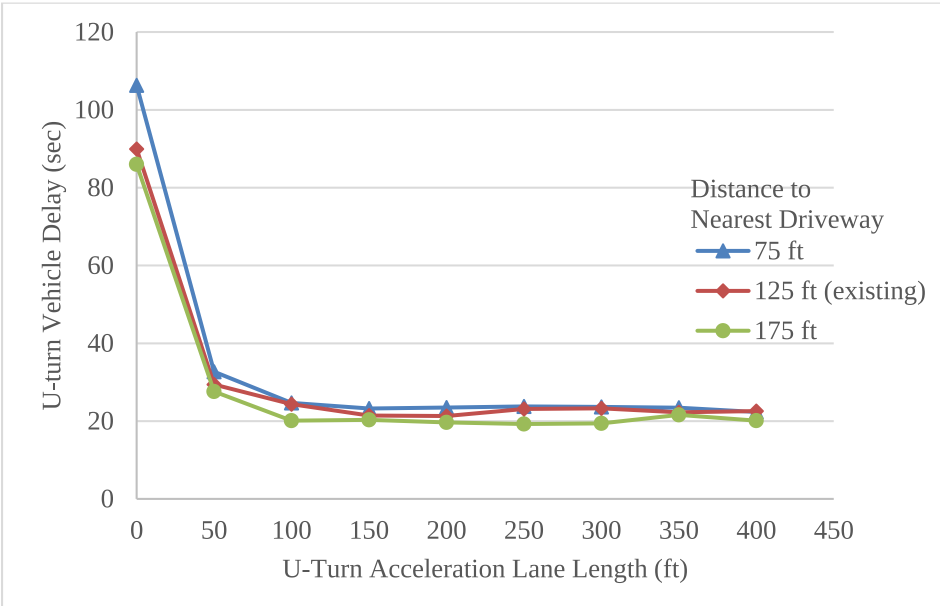


Figure 14. SB to NB U-Turn Delay Varied by Length of Acceleration Lane and Distance to Nearest Driveway at I-45 @ Research Forest.

At the I-10 @ Gessner site, the eastbound U-turn lane connects to a shared FR lane that leads to the freeway entry ramp (the lane diverge begins at about 350 ft downstream). Researchers simulated the countermeasure of adding an acceleration lane to this departure side. Table 84 in Appendix D shows the simulation results. Adding the acceleration lane generally improved U-turn departure traffic by reducing average and maximum queue lengths and queue stops. Slightly increased queue lengths were observed for the southbound right turn, westbound through, and northbound left-turn traffic due to the additional lane change maneuvers of the conflicting traffic traversing to the freeway entry.

At the I-10 @ Bunker Hill site, the eastbound U-turn lane connects to a shared frontage lane on the departure end that ends at the freeway ramp entry (turning begins at about 300 ft downstream). Researchers simulated the countermeasure of adding a tapered acceleration lane to this departure side. Table 85 in Appendix D shows the simulation results. During the AM peak hour, when volumes of eastbound U-turn traffic and the conflicting traffic were high, the added acceleration lane slightly improved U-turn departure traffic but also slightly decreased performance of the southbound right-turn traffic. Westbound through traffic and northbound left-turn traffic were not affected much. During the PM peak hour, when traffic volumes were higher, the added acceleration lane significantly improved southbound right-turn traffic but did not have much impact on the U-turn or the conflicting through and left-turn traffic.

Separation from Conflicted Traffic

The base model results indicated that the site at I-45 @ Rayford/Sawdust had congested U-turn operations in the NB to SB direction during both the AM and the PM peak hours and in the SB to NB direction during the PM peak hour. Particularly, the SB to NB U-turn departure side had very high U-turn volume and EB to NB left-turn volume and a great number of vehicles accessing the first gas station via the driveway 40 ft downstream from the U-turn gore. These factors caused the long queues and high delay of SB to NB U-turn traffic even though an added U-turn lane was presented at the departure side. These results were based on geometry and traffic data collected in October 2016, when there were old pylons that were worn off at this SB to NB U-turn departure side. Between then and February 2017, Montgomery County made several improvements to this corner (with permission from TxDOT): (a) a tapered outside lane was added to the NB FR for WB right-turn traffic; (b) the first driveway to the gas station was closed; (c) new pylons were installed for the U-turn acceleration lane; and (d) pylons were installed to delineate the tapered right-turn lane that extended to the middle of the new first driveway (previously the second driveway). Figure 15 shows the conditions before and after these improvements.

a. Before Condition at I-45 @ Rayford Rd b. After Condition at I-45 @ Rayford Rd

Figure 15. Before and After Condition at I-45 @ Rayford Road.

Researchers simulated the impacts of these actual improvements by applying three changes to the downstream NB FR in the base conditions:

- Closed the first driveway to all traffic (assuming traffic accesses the gas station at the new first driveway).
- Added a tapered lane on the FR for WB right-turn traffic based on the first change.
- Added pylons to delineate the SB to NB U-turn departure lane and the tapered lane for WB right-turn traffic based on the second change; considered 0 percent,

50 percent, and 100 percent compliance rates for southbound U-turn, eastbound left turn, and northbound through traffic accessing the new first driveway.

Table 24 (and Table 86 in Appendix D) shows the performance measures for southbound U-turn traffic under different conditions. Closing the first driveway to all traffic greatly improved U-turn traffic operations by reducing the average queue length and queue stops during the PM peak hour. Adding the tapered right-turn lane without installing pylons for the U-turn or right turn significantly increased queue lengths and stops to U-turn traffic. With pylons installed, the best performance for U-turn traffic was achieved under 0 percent compliance rate (assumed that traffic originally going to the service station ran over the last few pylons to access the new first driveway) followed by the 50 percent compliance rate condition during PM peak hours.

Table 24. VISSIM Evaluation Results for Countermeasures of Separation from Conflicted Traffic at Southbound U-Turn Departure End at I-45 @ Rayford Rd.

AM Peak Hour						
Measure of Effectiveness	Base	Change (1)	Change (2)	Change (3) with Compliance Rate		
				0%	50%	100%
Number of Vehicles	341	345	344	345	346	345
Avg. Queue Length (ft)	2.15	2.68	100.18	0.01	0.05	0.04
Max. Queue Length (ft)	145.56	133.33	544.38	18.64	28.13	18.78
Avg. Queue Stops (stops)	36	35	338	1	1	1
PM Peak Hour						
Measure of Effectiveness	Base	Change (1)	Change (2)	Change (3) with Compliance Rate		
				0%	50%	100%
Number of Vehicles	472	471	466	451	469	472
Avg. Queue Length (ft)	523.6	310.72	488.12	1.19	2.11	2.49
Max. Queue Length (ft)	1524.97	1524.45	1525.51	119.9	176.87	246.74
Avg. Queue Stops (stops)	1936	1215	1840	16	26	26

Table 87 through Table 89 in Appendix D show the simulation results for the westbound right turn, northbound through, and eastbound left turn, respectively. During the AM peak hour when the westbound right-turn volume was high, closing the driveway slightly reduced right-turn maximum queue length. Adding the right-turn lane and pylons significantly improved right-turn traffic, especially under the 0 percent compliance rate. The countermeasures did not have any apparent impact on the northbound through traffic. For the eastbound left turn, closing the driveway and adding the right-turn lane performed the best, followed by adding pylons under 50 percent and 100 percent compliance rates during the PM peak hour when left-turn volume was high.

Among the three improvements provided at this site, the pylon delineator for U-turn acceleration lane was of particular interest to researchers and TxDOT personnel because of its effectiveness in separating U-turn traffic from conflicted traffic and its relatively low cost of installation. Researchers further conducted a volume sensitivity analysis at this site in an attempt to identify thresholds of traffic volumes when pylon implementation was necessary for an added acceleration lane. The analysis used the following procedures to effectively model the impacts of varied demand levels and pylons on U-turn performance.

- Only traffic volumes of SB to NB U-turn, WB to NB right turn, and U-turn to the first driveway in the base conditions were varied to avoid excessive efforts in adjusting signal timing settings.
- The impact of pylons preventing U-turn traffic from accessing both driveways was simulated by changing the link/connector lengths and priority rule parameters in VISSIM with an assumed 100 percent compliance rate for a conservative evaluation.
- An exploratory analysis on driveway volumes was performed based on the AM base scenario to determine proper ranges of volumes to vary.
- A detailed volume sensitivity analysis was conducted based on the PM base scenario to estimate thresholds of volumes by comparing simulated queue results and actual queue storage space (300-ft U-turn lane plus 220-ft approach storage bay) in the field.

The AM base condition had two near driveways (named Dr #1 and Dr #2) located 40 ft and 150 ft downstream from the U-turn departure gore. Varying the U-turn to Dr #1 volume from 0 vph to 20 vph with 5 vph increments (and with U-turn and right-turn volume fixed at 450 vph and 1000 vph), the VISSIM simulation generated maximum queue length without and with pylons blocking U-turns' access to Dr #1, as shown in Figure 16.

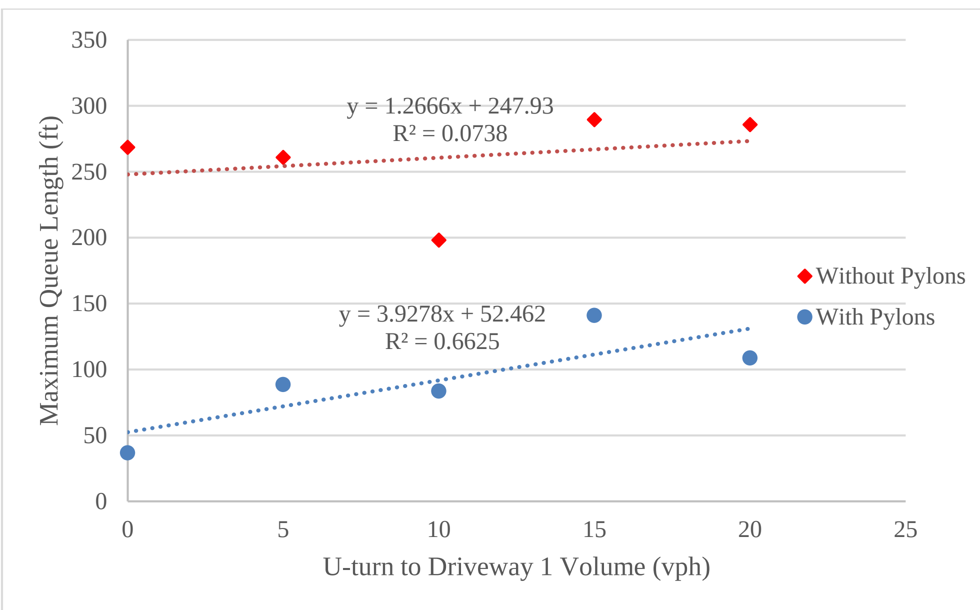


Figure 16. SB to NB U-turn Queue Results with Varied Driveway 1 Volumes Based on AM Scenario at I-45 @ Rayford/Sawdust.

Generally, longer U-turn queues were associated with higher driveway volumes in both scenarios without and with pylons. The maximum queue length without pylons ranged between 200 ft and

300 ft and the average queue length was shorter than 20 ft. This indicates that the 450 vph U-turn traffic can be accommodated well under the AM condition (600 vph flow of EB left turn and NB through, and 1000 vph WB right turn, and 120 sec cycle length). Nevertheless, with pylons added to direct U-turn to Dr #1 traffic to Dr #2, the maximum queue length was reduced by at least 50 percent.

Researchers conducted more detailed volume sensitivity analysis based on the PM scenario (1000 vph flow of EB left turn and NB through, and 135 sec cycle length). U-turn volume was varied from 350 vph to 650 vph with 100 vph increments; right-turn volume was varied from 200 vph to 600 vph with 100 vph increments; U-turn to Dr #1 volume was varied from 0 vph to 20 vph with 5 vph increments. Figure 17 displays the average queue length varied by the sum of U-turn, right turn, and U-turn to Dr #1 volumes without U-turn to Dr #2 traffic. If requiring the average queue length to be no longer than the 300 ft U-turn lane, pylons should be used to prevent U-turn traffic from entering Dr #1 when U-turn volume is 650 vph and right-turn volume is 600 vph. With pylons applied in directing U-turn traffic to Dr #2, the average U-turn queue length was no longer than 180 ft.

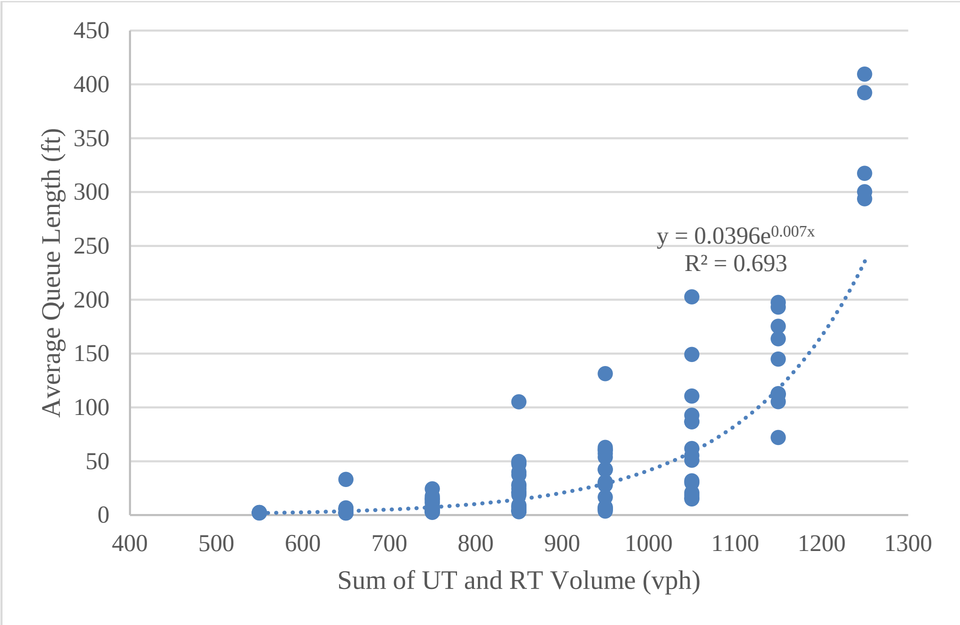


Figure 17. SB to NB U-turn Queue Results Varied by Sum of U-turn, Right Turn, and U-turn to Dr #1 Volume Based on PM Scenario at I-45 @ Rayford/Sawdust.

Signal Control Changes for Interior Left Turn

For sites without a U-turn lane, U-turn traffic makes two successive left turns at the interchange with other left-turn traffic. Passing through traffic signals twice causes additional signal control delay. The interior left turn can have protected-only (PO) and protected-permissive left-turn (PPLT) operations. With PO operation, left-turn traffic cannot make use of the long gaps, if available, in the opposing through traffic. Conversely, PPLT operation provides the interior left turn with additional capacity by allowing left-turn traffic to turn in those available long gaps. This facilitates U-turn traffic moving at interchanges with a U-turn lane.

At the SH 6 @ Briarcrest site in the Bryan District, the interior left turns are operated under PPLT operation, which is allowable because of the five-section signal head. Researchers simulated changing the PPLT operation to the PO operation to compare the impacts of different signal operations of interior left-turn traffic on U-turn traffic at sites without a U-turn lane. Table 25 (and Table 90 in Appendix D) shows the performance measures of the comparison. As expected, U-turn traffic in both directions experienced much higher delays under the PO operation compared to the PPLT operation. The cross-street left turns also had significantly increased delays.

Table 25. VISSIM Evaluation Results for Countermeasure of Interior Left-Turn Operations at SH 6 @ Briarcrest Dr during AM Peak Hour.

Base Condition with Protected-Permissive Left Turn for Interior Left-Turn Traffic															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	414	359	357	411	576	106	94	818	147	238	12	116	205	613	**4466**
Avg. Queue Length (ft)	57	57	57	91	91	0	81	81	81	81	41	41	41	41	**54**
Max. Queue Length (ft)	222	222	222	342	342	31	340	340	340	340	339	339	339	339	**387**
Avg. Delay (sec/veh)	51.7	40.2	2.4	53.8	49.3	1.6	49.0	33.7	26.8	3.1	83.2	37.9	34.2	12.2	**32.3**
Stopped Delay (sec/veh)	32.6	26.2	0.2	32.5	30.4	0.0	33.4	23.0	19.3	0.6	71.3	31.8	26.5	3.3	**20.3**
Avg. Stops (stops/veh)	1.02	0.73	0.04	0.88	0.77	0.02	1.61	0.69	0.60	0.12	1.84	0.79	0.68	0.47	**0.65**
Protected-Only Left Turn for Interior Left-Turn Traffic															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	416	359	357	412	576	106	93	818	147	238	12	116	205	612	**4468**
Avg. Queue Length (ft)	57	57	57	91	91	1	81	81	81	81	44	44	44	44	**55**
Max. Queue Length (ft)	222	222	222	335	335	104	338	338	338	338	359	359	359	359	**375**
Avg. Delay (sec/veh)	78.4	41.5	2.4	62.9	50.1	1.6	95.0	35.0	26.8	3.4	129.2	38.7	34.2	13.3	**37.4**
Stopped Delay (sec/veh)	53.1	26.5	0.2	37.6	30.5	0.1	79.0	23.6	19.4	0.7	113.0	32.3	26.5	4.0	**24.0**
Avg. Stops (stops/veh)	1.32	0.75	0.04	1.07	0.79	0.02	1.59	0.71	0.61	0.14	1.89	0.79	0.68	0.52	**0.71**

Signalized U-Turn

Different types of departure control can provide different levels of delay, and U-turn delay can decrease as U-turn departure treatments become more amenable to conflict-free and control-free movements. It is expected that the operation of the U-turn movement may be adversely influenced under signalized U-turn operation. From a safety perspective, this countermeasure may reduce the conflicts on the departure of the U-turn lane.

At the I-410 @ Ingram site, a signalized U-turn lane was modeled for both U-turn directions. On the departure side, the right turn from the cross street is restricted when the U-turn has the green. Table 91 and Table 92 in Appendix D show the VISSIM results of implementing the signalized U-turn control countermeasure for the I-410 @ Ingram site. As expected, the signalized U-turn control did significantly affect the U-turn operation. In the morning peak hour, the average delays increased to 31 seconds and 38 seconds for northbound and southbound U-turns, respectively. In the PM peak hour, the delays increased to 76 and 78 seconds, respectively. From the perspective of the whole interchange, the changes were much less significant. The average delay increased from 27 seconds to 29 seconds in the morning, and from 50 seconds to 58 seconds in the afternoon.

Added Lane Sign

The installation of traffic lane addition signing on the U-turn departure increases driver awareness of the nature of the downstream junction between the U-turn lane departure and the FR. Since the U-turns featured lane additions due to the U-turn (rather than merging with or without an acceleration bay), W4-6 (Entering Added Lane) or W4-3 (Added Lane) signs were evaluated. Researchers selected I-410 @ Ingram as the study site.

At this interchange, the added lane sign was modeled for both northbound and southbound directions. For the purposes of the VISSIM modeling, two cases were tested. Case 1 looked at a 100 percent compliance rate to this measure (meaning no traffic from the U-turn, except for those heading to the closest driveway, would yield to the upcoming FR traffic). Case 2 looked at a 50 percent compliance rate, (meaning 50 percent of the U-turn traffic still yields to the through traffic on the FR).

Table 93 through Table 94 in Appendix D show the VISSIM results of the added lane sign countermeasure for the I-410 @ Ingram site. The results indicate that under the 100 percent compliance rate, the added lane sign slightly improved the operation of the U-turn movement and the whole approach when compared to the base condition. However, the results from the cases of the 50 percent compliance rate did not show a consistent trend.

The additional queue lengths and stops were separately measured on the U-turn departure for both directions, and the results are shown in Table 95 in Appendix D. With the 100 percent compliance rate, the queue length and delay for both U-turns reduced to zero since all U-turn vehicles moved freely toward the added frontage lane without stopping or slowing down. Under the condition with half of the vehicles still yielding to the frontage traffic (50 percent compliance rate level), this countermeasure reduced U-turn average and maximum queue lengths during both AM and PM peak hours, except for the southbound U-turn during the PM peak hour. One possible reason for this exception was that U-turn traffic patterns may have larger portions of vehicles accessing the first driveway on the FR (those vehicles will yield to the frontage traffic even with the added lane sign). In general, the benefit is more significant during the PM peak hour when the high FR volumes on the departure side of a U-turn may cause more stops and longer queuing into the U-turn lane.

Altering Left Turn to Direct Vehicles to Alternate Receiving Lanes

Many diamond intersections include dual left-turn lanes for the cross-street internal left turns, and these left turns are allowed to turn into multiple receiving lanes on the FR. To provide more and longer gaps in the leftmost lane on the FR and thus reduce U-turn queue length and delay on the departure end, interior left-turn traffic can be directed to the outer receiving lanes on the FR. This result can be achieved by using additional dotted pavement markings or signs (or a combination of both). Effectiveness of this countermeasure depends on volumes of the U-turn and the conflicting left-turn traffic and the compliance rate of the applied markings and/or signs. Researchers selected the eastbound left turn at the site of I-45 @ Research Forest for the experiment of this countermeasure.

At this interchange, traffic in the eastbound left-only lane is allowed to turn into either the left or middle FR lane. Left-turning traffic in the middle eastbound lane must turn into the right FR lane. Figure 18 shows these left-turn destination lane options. For the countermeasure modeling, the researchers modeled the effect of restricting eastbound left-turning traffic from entering the left FR lane. It was assumed that this would be accomplished via additional cat tracks on the pavement and/or additional signage.

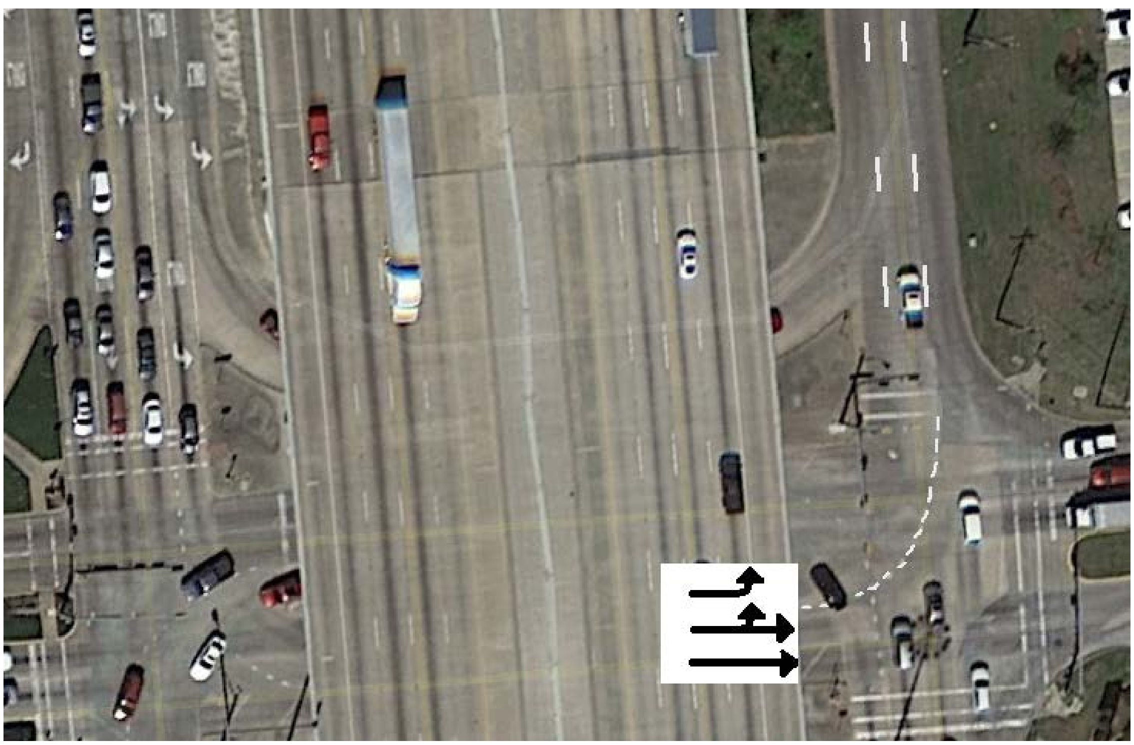


Image Source: Google Maps

Figure 18. I-45 @ Research Forest Interchange.

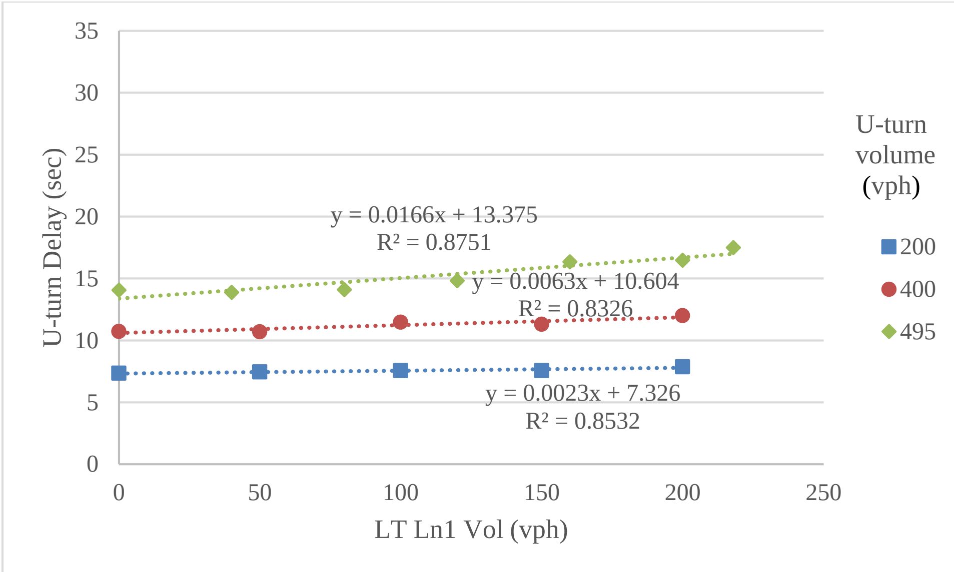
For the purposes of the VISSIM modeling, two cases were tested. Case 1 looked at a 100 percent compliance rate to this restriction (meaning no traffic from the eastbound left turns would access the left FR lane). Case 2 looked at a 50 percent compliance rate to the restriction (meaning only 50 percent of the eastbound left-turn traffic previously turning into the left FR lane would turn into the far left lane).

Table 26 (and Table 96 in Appendix D) shows queue results of the southbound U-turn at the departure end. Applying this countermeasure generally reduced U-turn queue lengths and delay in both AM and PM peak hours. For the PM peak hour when both left-turn and U-turn volumes were high, better results were associated with the higher compliance rate.

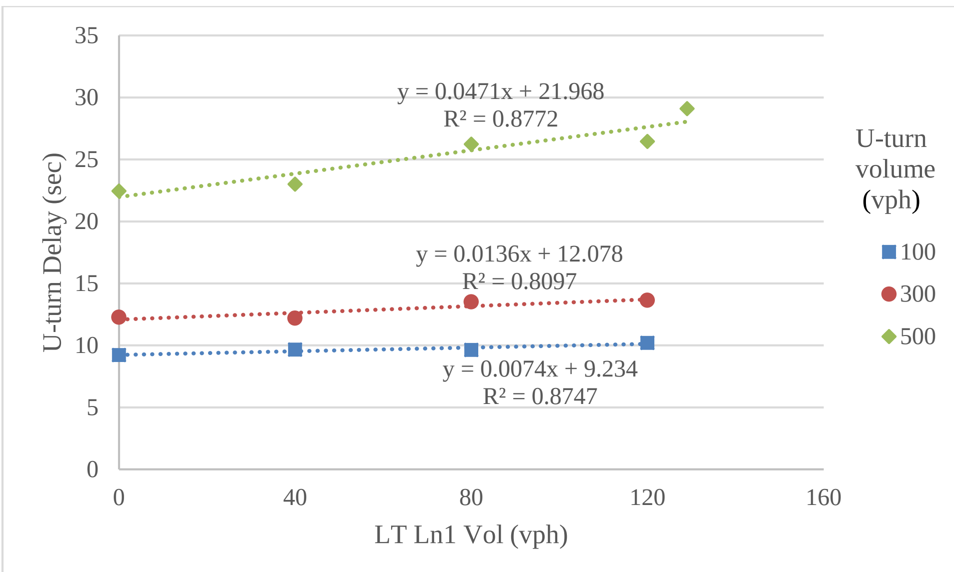
Table 26. VISSIM Countermeasures Results—Direct Vehicles to Alternate Receiving Lanes Performance Measures of Southbound U-turn Traffic at I-45 @ Research Forest Dr.

Measure of Effectiveness	AM Peak Hour			PM Peak Hour		
	Base	Compliance Rate		Base	Compliance Rate	
		50%	100%		50%	100%
Number of Vehicles	308	308	308	512	510	510
Avg. Queue Length (ft)	0.86	0.65	0.75	97.5	89.8	88.7
Max. Queue Length (ft)	90.0	72.6	84.1	612.5	557.7	566.3
Avg. Queue Stops (stops)	24	23	24	513	505	486

Researchers further conducted sensitivity analyses on the U-turn volume levels and conflicting left-turn volumes in Lane 1 on the FR at the U-turn departure side. Figure 19a shows the results of NB to SB U-turn delay under U-turn demand of 200 vph, 400 vph, and 495 vph (existing demand) with WB left-turn volume in Lane 1 on the SB FR varying from 0 vph to 200 vph with 50 vph increments. Figure 19b shows the results of SB to NB U-turn delay under U-turn demand of 100 vph, 300 vph, and 500 vph (existing demand is 510 vph) with EB left-turn volume in Lane 1 on the SB FR varying from 0 vph to 120 vph with 40 vph increments. Both figures show the same trends: 1) U-turn delay increases with increases in U-turn demand; 2) under the same U-turn demand scenario, U-turn delay increases with increases left-turn demand in FR Lane 1. Furthermore, the higher the U-turn demand, the steeper the slope of the linear trend line. Because the SB to NB U-turn departure side has a nearby driveway, SB to NB U-turn delay has a higher delay than the NB to SB direction under approximately the same demand level of 500 vph. Also, the slope of the linear trend line under the same 500 vph U-turn demand level for the SB to NB U-turn is greater than that of the NB to SB U-turn. These trends indicate that the countermeasure of directing conflicting left-turn traffic to alternate receiving lanes is expected to have better effectiveness in reducing U-turn delay under higher U-turn demand and at sites with nearby driveways. At this site, when U-turn demand is 500 vph or higher, this countermeasure could potentially reduce U-turn delay by at least 24 percent and 30 percent for NB to SB and SB to NB U-turn traffic, respectively.



a. NB to SB U-Turn delay sensitivity analysis based on AM volume



b. SB to NB U-Turn delay sensitivity analysis based on PM volume

Figure 19. U-Turn Delay Varied by U-Turn Demand and Left-Turn (LT) Volume in Lane 1 (Ln1) on Frontage Road at Research Forest.

No Right Turn on Red

When modeling effects of no RTOR restrictions for the cross street, it is assumed that signage and possibly enforcement would be used. Researchers evaluated this countermeasure at four sites: I-45 @ Research Forest, I-20 @ McCart, I-20 @ Hulen, and I-410 @ Ingram. Once again, two cases were tested at each site. Case 1 looked at a 100 percent compliance rate to this restriction (meaning no traffic on the cross-street treatment approach would turn right during the red signal indication). Case 2 looked at a 50 percent compliance rate to the restriction (meaning only 50 percent of the traffic previously making cross-street right turns on red would make that movement).

At the site of I-45 @ Research Forest, a westbound RTOR restriction was simulated. Table 27 (or Table 97 in Appendix D) shows the measures of queue and delay results for the southbound U-turn and westbound right-turn traffic. Under the 50 percent compliance rate level, this countermeasure reduced U-turn average and maximum queue lengths during both AM and PM peak hours. With the 100 percent compliance rate, queue measurements increased for the PM peak hour when U-turn volume was high. One possible reason for this was that U-turn vehicles arriving randomly throughout the cycle encountered generally fewer conflicts from cross-street right-turn vehicles when under the 50 percent compliance rate compared to the base condition. With the 100 percent compliance rate, U-turn vehicles arriving during cross-street red intervals were relatively free of conflicts from cross-street right-turn traffic; but U-turn vehicles arriving during cross-street green intervals experienced higher delay due to higher right-turn flow during green compared to that of the lower compliance rate. Intuitively, the restriction of RTOR generally increased delay to cross-street right-turn traffic. The results from this site indicate that this countermeasure may be good for sites with low U-turn volume and low right-turn volume on the cross street.

Table 27. VISSIM Countermeasures Results—No RTOR from Cross-Street Performance Measures at Houston District I-45 @ Research Forest Dr.

Measure of Effectiveness	AM Peak Hour			PM Peak Hour		
	Base	Compliance Rate		Base	Compliance Rate	
		50%	100%		50%	100%
Southbound U-turn Departure End						
Number of Vehicles	308	308	308	512	510	510
Avg. Queue Length (ft)	0.86	0.80	0.85	97.5	95.8	102.6
Max. Queue Length (ft)	90.0	84.8	89.6	612.5	562.8	597.2
Avg. Queue Stops (stops)	24	26	25	513	520	538
Westbound Right Turn						
Number of Vehicles	134	134	134	138	137	137
Avg. Queue Length (ft)	89	90	91	107	109	108
Max. Queue Length (ft)	260	255	254	294	297	296
Avg. Delay (sec/veh)	42.9	45.6	47.9	50.9	53.4	53.9
Stopped Delay (sec/veh)	35.5	37.9	40.1	42.2	44.5	45.0
Avg. Stops (stops/veh)	0.83	0.86	0.88	0.86	0.87	0.87

At the I-20 @ McCart site, the restriction of no RTOR from the cross street was modeled on the southbound direction only. This site does not contain a westbound to eastbound U-turn lane; therefore, the northbound right-turn movement does not have any conflicting U-turn from westbound lanes. Table 98 through Table 100 in Appendix D show the VISSIM results of the no RTOR restriction for the I-20 @ McCart site. The results show that this restriction increased delay (which doubled at the 100 percent compliance rate) and stops for the southbound right turn. This result was expected because those vehicles were prevented from making right turns on red and had to wait a longer time at the stop line. However, the operation of the eastbound U-turn did not gain significant improvement as the result of the compromised RTOR.

At the I-20 @ Hulen site, the restriction of no RTOR from the cross street was modeled on both northbound and southbound directions. This site features moderate right-turn traffic volume in the morning peak hour for both directions, and during the PM peak hour, the right-turn traffic volumes for both directions are high. Table 101 through Table 103 in Appendix D show the VISSIM results of the no RTOR restriction for the I-20 @ Hulen site. The results show that this restriction greatly increased delay and stops for the cross-street right turns. Again, just like at the McCart site, this result was expected because those vehicles were prevented from making right turns on red and had to wait a longer time at the stop line. However, the operation of both U-turns did not gain significant improvement as a result of the compromised RTOR. It is worth noting that during the PM peak hour when the cross-street right-turn traffic volumes were high, the no RTOR restriction had a severely negative impact on the cross-street traffic and even gridlocked the whole approach.

At the I-410 @ Ingram site, the restriction of no RTOR from the cross street added was modeled on both the westbound and eastbound directions. This site features very high right-turn traffic volume in both peak hours for both directions. Table 104 through Table 106 in Appendix D show the VISSIM results of the no RTOR restriction for the I-410 @ Ingram site. The results show that this restriction greatly increased delay and stops for the cross-street right turns. Just as with the other two sites using this countermeasure, the operation of both U-turns did not gain significant improvement as the result of the compromised RTOR. Again (for this site as well), it is worth noting that during both peak hours when the cross-street right-turn traffic volumes were high, the no RTOR restriction had a severely negative impact on the cross-street traffic and gridlocked the whole approach.

No Right Turn on Red Except from Right Lane Sign

Researchers also tested the RTOR restriction at sites with exclusive and shared right-turn lanes by applying the no RTOR Except from Right Lane sign. The I-10 @ Gessner site was selected to evaluate this countermeasure for southbound right turns because of the relatively high right-turn volume and existence of one exclusive and one shared right-turn lane. The results are found in Table 107 in Appendix D. This countermeasure slightly reduced U-turn queue length during the AM peak hour without affecting the conflicting traffic significantly. During the PM peak hour, this RTOR restriction slightly increased U-turn maximum queue length.

Driveway Closure

When modeling the effects of closing the nearest driveway to the interchange, the OD values were adjusted to not allow traffic to access the driveway. It was assumed that this adjustment would be accomplished via pylons, raised curb, or a double white solid stripe at the departure of the U-turn lane. For the purposes of the VISSIM modeling, two cases were tested. Case 1 looked at a 100 percent compliance rate to this restriction (meaning no traffic from the U-turn would access the closed driveway). This result would likely be accomplished by the installation of a raised curb. Case 2 looked at a 50 percent compliance rate to the restriction (meaning only 50 percent of the traffic previously making that U-turn to driveway route would make that movement). In both cases, diverted traffic was sent to the next driveway on the FR, if one existed. Researchers evaluated this countermeasure at four sites: I-10 @ Gessner, I-45 @ Research Forest, I-10 @ McCart, and I-20 @ Hulen.

At the I-10 @ Gessner site, multiple driveways exist along the eastbound FR at the U-turn departure side. Researchers considered closing the first two driveways to westbound U-turn traffic by directing this traffic to use the third driveway. Table 28 (and Table 108 in Appendix D) shows the results. Closing the first driveway or the first two driveways eliminated westbound U-turn queues and stops completely without noticeable negative impact on the conflicting traffic.

Table 28. VISSIM Countermeasures Results—Eastbound Driveway Closure to U-Turn Performance Measures at I-10 @ Gessner Rd.

Measure of Effectiveness	AM Peak Hour			PM Peak Hour		
	Base	Driveway Closure		Base	Driveway Closure	
		1st	1st & 2nd		1st	1st & 2nd
Eastbound U-turn Departure End						
Number of Vehicles	184	184	184	224	220	226
Avg. Queue Length (ft)	0.31	0	0	0.72	0	0
Max. Queue Length (ft)	71.75	0	0	144.45	0	0
Avg. Queue Stops (stops)	4	0	0	12	0	0
Northbound Right Turn						
Number of Vehicles	275	275	275	276	276	276
Avg. Queue Length (ft)	58	58	58	85	85	85
Max. Queue Length (ft)	179	177	182	288	295	286
Avg. Delay (sec/veh)	26.9	26.3	26.0	23.6	22.9	22.9
Stopped Delay (sec/veh)	22.0	21.5	21.3	18.8	18.3	18.3
Avg. Stops (stops/veh)	0.63	0.61	0.61	0.53	0.52	0.52
Eastbound Through						
Number of Vehicles	1000	1000	1000	644	645	644
Avg. Queue Length (ft)	97	97	97	143	141	158
Max. Queue Length (ft)	294	295	304	455	442	471
Avg. Delay (sec/veh)	44.2	43.3	43.1	51.4	50.3	50.9
Stopped Delay (sec/veh)	33.5	33.3	33.2	40.8	40.6	41.3
Avg. Stops (stops/veh)	0.80	0.80	0.79	0.82	0.81	0.82
Southbound Left Turn						
Number of Vehicles	839	838	839	674	676	676
Avg. Queue Length (ft)	93	92	93	129	134	134
Max. Queue Length (ft)	323	316	328	379	406	413
Avg. Delay (sec/veh)	45.1	44.4	44.7	67.9	69.9	70.3
Stopped Delay (sec/veh)	30.9	30.6	30.8	53.1	55.2	55.3
Avg. Stops (stops/veh)	0.83	0.82	0.82	1.01	1.04	1.04

At the I-45 @ Research Forest site, the first driveway downstream of the northbound FR was modeled to be closed to southbound U-turn traffic by using traffic signs at the upstream U-turn arrival side. Table 109 in Appendix D shows the performance measures for the southbound U-turn and its conflicting movements on the northbound FR. This countermeasure generally reduced U-turn queues and did not have significant impact on conflicting movements when the volumes of the conflicting movements were low during AM peak hours. During the PM peak hour, when northbound through and eastbound left-turn volumes were high, this countermeasure also reduced queues and delays, especially when the compliance rate was high.

At the I-10 @ McCart site, the restriction of first driveway closure was modeled on only the westbound FR (affecting the eastbound U-turn movement) due to the lack of detailed OD data for the other direction. The nearest driveway on the westbound direction is a minor access road to a residential area, and the traffic impacted by the driveway closure was 14 veh/hour and

59 veh/hour for AM and PM peak hours, respectively (at 100 percent compliance). Table 110 through Table 112 in Appendix D show the VISSIM results. The only movement that was expected to be affected was the eastbound U-turn. It was observed that the delay and stops for the movement were improved slightly when compared to the base condition. The improvement was consistent across both peak periods and at the different compliance rates, but the level of the improvement was minor for all cases.

At the I-20 @ Hulen site, the modeling and analysis mainly focus on the westbound direction (affecting the eastbound U-turn movement). The U-turn traffic heading to the nearest driveway was 11 veh/hour and 23 veh/hour for AM and PM peak hours, respectively. In addition to the closure of the first nearest driveway, the researchers also modeled the cases with the closure of the second nearest driveway along with the first driveway closure. The U-turn traffic impacted by the second driveway closure was 63 veh/hour and 62 veh/hour for AM and PM peak hours, respectively (at 100 percent compliance).

Table 113 through Table 114 in Appendix D show the VISSIM results of the driveway closure restriction for the I-20 @ Hulen site. At this interchange, the only movement that was expected to be affected was the eastbound U-turn. It was observed that after the first driveway closure, there were no significant changes on the delay and stops for the movement for the AM peak hour; and the improvement for the PM peak hour was slightly greater such that the average delay for the eastbound U-turn reduced from 4.3 seconds to 2.5 seconds when the compliance rate was 100 percent. It is worth noting that the impact from the closure of the second driveway in addition to the first driveway closure was almost negligible for both peak hours, even though the affected traffic from the second driveway closure was much higher. The most likely reason for this result is that the U-turn traffic heading to the second driveway did not slow down or stop as much as the traffic heading to the first driveway in the base condition. As a result, the benefit gained from the second nearest driveway closures was capitalized in the base condition.

The additional queue lengths and stops were separately measured on the U-turn departure for both directions, and the results are shown in in Table 115 in Appendix D. Under both 50 percent and 100 percent compliance rate levels, it was observed that after the first driveway closure, there were no significant changes on U-turn stops and queue length for the AM peak hour. One possible reason for this exception was that U-turn traffic patterns may have fewer vehicles accessing the first driveway on the FR such that the effectiveness of the improvement was insignificant. The improvement for the PM peak hour was greater such that the average number of stops and maximum queue length for the eastbound U-turn reduced 29 percent and 45 percent, respectively. Similar to the overall interchange performance results, the results for the U-turn departure side also indicated that the impact from the closure of the second driveway in addition to the first driveway closure was almost negligible for both peak hours.

RTOR Must Yield to U-Turn Sign

To reduce or remove the conflict between the U-turn departure and the cross-street right turn, signs such as the RTOR Must Yield to U-Turn sign (R10-30) and U-Turn Yield to Right Turn (R10-16) can be used. Figure 20 shows these signs.

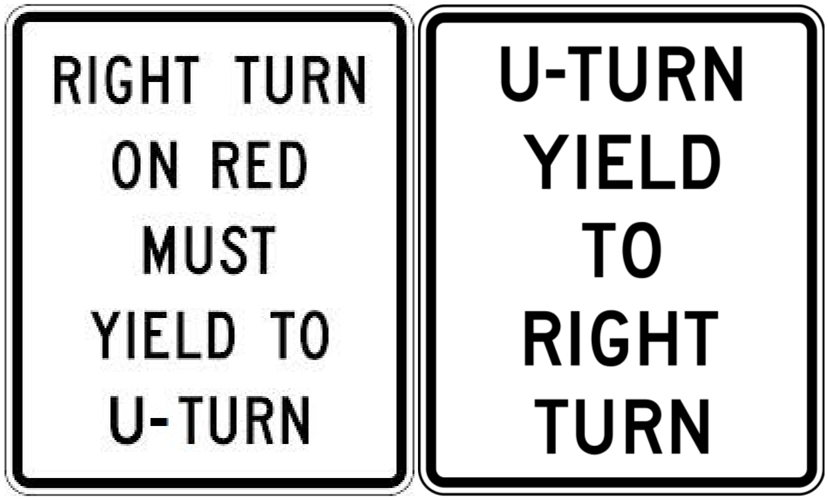


Figure 20. Example MUTCD RTOR and U-Turn Traffic Yield Signs.

At the I-20 @ Hulen site, the restriction of RTOR yield to U-turn traffic was modeled on both northbound and southbound directions. In this modeling, U-turn traffic still yields to the upcoming traffic from the FR. Table 116 through Table 117 in Appendix D show the VISSIM results. The results show that this restriction had minor (for northbound right turn) and moderate (for southbound right turn) adverse impacts on the queue length, delay, and stops for the cross-street right turns. These impacts are mainly caused by the RTOR traffic having to yield to more traffic and wait longer at the stop line. As the result of the compromised RTOR, it is expected that the operation of U-turn traffic on the FR would improve. The results show consistent yet insignificant improvement on both U-turns during both peak hours. The additional queue lengths and stops were separately measured on the U-turn departure for both directions, and the results are shown in Table 118 in Appendix D. The results show consistent yet insignificant improvement on both U-turns during both peak hours. The U-turn movements and the other movements from the same approaches have low average delay and queue length under the existing traffic volumes, and this could be part of the reason why the observed improvement of U-turn bay extension on either side of the interchange was insignificant.

FIELD TESTING OF SELECTED SOLUTIONS

In addition to using simulation modeling to evaluate results, researchers identified two field sites to be used for actual field implementation to evaluate proposed changes in a real field setting.

Selection and Description of Field Study Sites

Field study sites were selected to best leverage the local TTI resources for interaction with local TxDOT staff and for data collection when performing before/after studies of potential solutions identified in the research effort. Given the large research staff located in San Antonio and Houston, the sites identified within each of those two cities by TxDOT staff during the field site identification portion of the research became the set from which implementation sites were initially identified.

San Antonio Study Site

In San Antonio, the original site selection identified the I-410 @ Callaghan and I-410 @ Ingram interchanges as having high U-turn volume and/or operational issues with U-turns that would be of concern and interest in a U-turn-related research investigation. However, at the time the research project began, construction on Callaghan was underway for an arterial roadway expansion project being performed by the City of San Antonio. To avoid any potentially non-U-turn-related factors (associated with the construction) affecting the research findings, the I-410 @ Ingram site was selected for implementation of physical treatments that were identified in the early phases of the research project for positively influencing U-turn lane performance.

Figure 21 provides an aerial overview of the I-410 @ Ingram interchange and gives an indication of the fully developed area of San Antonio in which the interchange is located. U-turn activity is high at the interchange and serves the retail land use predominant along the I-410 corridor in this area. The interchange and U-turn lanes are skewed, but U-turn flow is facilitated by the fact that both NB to SB and SB to NB U-turns add a lane along the FR.

Image Source: Google Maps

Figure 21. I-410 @ Ingram Interchange.

Houston Study Site

Initial study site selection processes in the Houston region placed focus on the I-10 @ Gessner interchange as the location for experimenting with diamond signal timing as a means of positively influencing U-turn flow and performance and as a means of reducing congestion. Early discussions with the City of Houston, whose traffic operations staff manage the interchange, indicated that they could offer cabinet access (for researcher equipment to log controller activity) and make signal timing changes for the purposes of research testing and experimentation.

However, practical considerations with coordinating researcher involvement with both TxDOT Houston District and City of Houston staff in a detailed field study at multiple levels of agency interaction and scheduling soon revealed that an alternative site managed by only one agency would be most amenable to experimentation. Two alternative sites (and ones better situated to leverage the resources of researchers) at I-45 @ Research Forest and I-45 @ Rayford were reviewed for U-turn research in the area of signal timing. Figure 22 provides an aerial overview of the I-45 @ Research Forest interchange.

Image Source: Google Maps

Figure 22. I-45 @ Research Forest Interchange, The Woodlands, Texas.

Description of Solutions to Be Tested

Under Task 2 of the research effort, TTI staff identified and examined in some detail a number of potential treatments to facilitate U-turn lane access and/or flow. Some treatments, such as U-turn lane extension or driveway closure proximate to the U-turn entry or departure, either required roadway construction or high-level interaction with adjacent property owners; these solutions could not be executed and examined within the two-year time frame of the current research. Practical—and realistically more readily implementable—treatments that could be examined were those that could be accomplished with roadside signing, striping, or signal timing changes.

Signing Treatment

Typical U-turn signing treatment in the TxDOT San Antonio District includes R1-2 Yield signing at the U-turn lane departure regardless of whether an acceleration lane or full lane is added. At I-410 @ Ingram, these signs were in place prior to the research project. Testing conducted under the research investigation involved removing the Yield signs and analyzing before/after U-turn traffic data to assess whether drivers altered their U-turn departure yielding behavior.

A planned field treatment involved the installation of traffic lane addition signing on the U-turn departure to increase driver awareness of the nature of the downstream junction between the U-turn lane departure and the FR. Because the U-turns at Ingram featured lane additions due to the U-turn (rather than merging with or without an acceleration bay), W4-6 (Entering Added Lane) or W4-3 (Added Lane) signs would have been used (examples found in Figure 23).

W4-3L
Left Added Lane

W4-6L
Entering Roadway Added Lane Left

Figure 23. Example MUTCD Lane Addition Signing.

Unfortunately, this treatment option could not be installed in time for evaluation under the current research project. TxDOT San Antonio District staff and researchers from TTI have made arrangements to collect performance measures for this future installation that can be shared through TxDOT meetings and/or meetings among agency operations staff.

Striping Treatment

I-410 @ Ingram was also the implementation site for striping treatments being evaluated under the research effort. While signing treatments were intended for one side of the interchange, researchers treated the U-turn lane on the other side of the interchange with double white striping. As with the signing treatment, the purpose of the double white lines was to emphasize the nature of the junction between the lane added by the U-turn departure and the FR wherein U-turning drivers do not need to yield to through traffic on the FR. This treatment was installed by TxDOT contractors in summer 2017. TTI technicians collected video data following implementation so that researchers could compare driver behavior and traffic flow data between the original site condition (with the Yield signs removed), and the double white striping in place. Figure 24 shows the lane striping treatment applied to the I-410 @ Ingram study site.

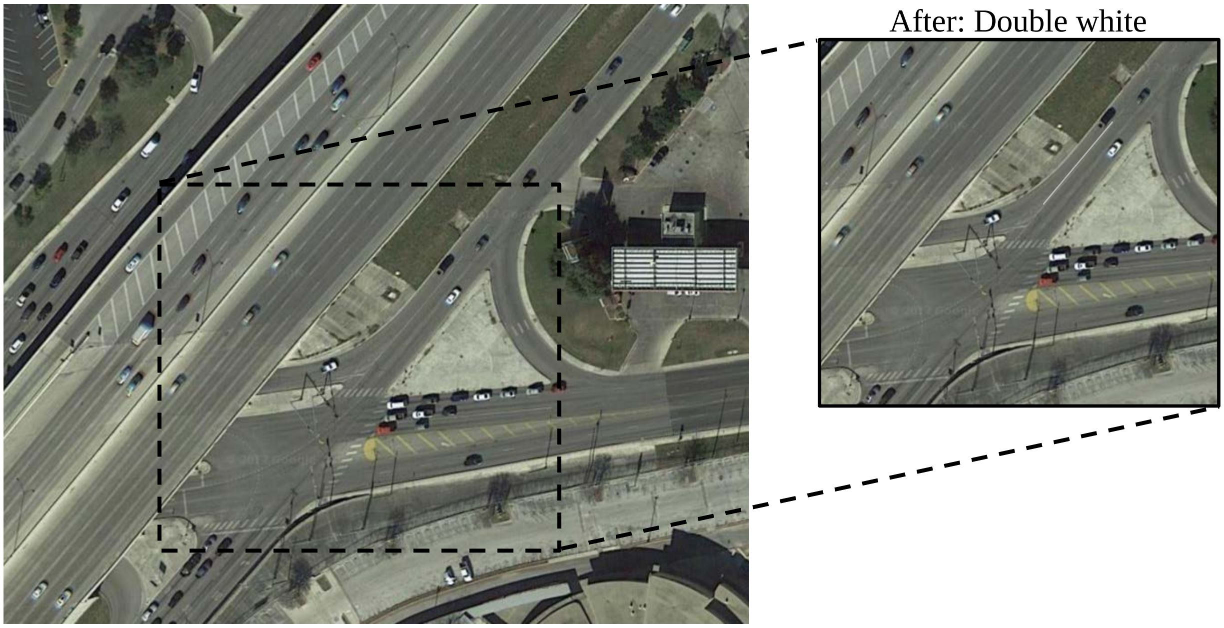


Image Source: Google Maps

Figure 24. I-410 @ Ingram SB to NB U-turn, Lane Striping Treatment.

If time had permitted during the course of the research project, a final plan was in place to install flexible pylons (example in Figure 25) over the double white striping at the I-410 @ Ingram interchange. However, as with the Ingram signing treatment, there was inadequate time to perform this installation during the course of the project. If plans for installing this treatment are later followed by TxDOT San Antonio District staff, both TxDOT and TTI will evaluate the treatment and share the results via TxDOT meetings/forums.

Image Source: Google Maps

Figure 25. U-Turn Departure Installation of Pylons.

Signal Timing

Signal timing changes to accommodate U-turn maneuvers can take several forms, including increasing the green percentage (of cycle) for the FR to allow U-turning vehicles to enter U-turns that may otherwise be blocked by through vehicles. Green time may also be restricted on the external arterial approaches to restrict an arterial right-turning movement that conflicts with U-turning vehicles entering the same segment of FR.

Field Evaluation Results—Removal of Yield Signs at I-410 @ Ingram

For the evaluation of the removal of the Yield sign at I-410 @ Ingram, U-turn delay and stop data were collected following implementation so that driver behavior and traffic flow data could be compared between the original site condition and the condition with the Yield signs removed.

Table 29 shows the comparison of the U-turn delay field data before and after the removal of the Yield sign. The total and average U-turn delay were higher after the sign removal for both directions during both AM and PM peak hours compared to delays before removing the Yield sign. However, there were significant increases in U-turn volume and the FR volume at the departure side of both U-turn directions: the NB to SB U-turn volume increased by 47 percent and 75 percent in AM and PM, respectively, and the FR volume at its departure side increased by 12 percent and 15 percent in AM and PM, respectively; the SB to NB U-turn volume increased by 115 percent and 5 percent in AM and PM, respectively, and the FR volume at its departure side increased by 47 percent and 27 percent in AM and PM, respectively. Using the FR volume to normalize average delay data, the site decreased normalized U-turn delay after the removal of the Yield sign in the PM peak hour for both directions (28 percent reduction for the NB to SB U-turn and 22 percent reduction for the SB to NB U-turn) and in the AM peak hour for the SB to NB U-turn (7 percent reduction). NB to SB U-turn delay increased significantly in both average delay and normalized average delay, which was associated with the tripled number of U-turn vehicles going to the first and second driveways in this direction. The field study result indicates that removing unnecessary Yield signs may have a positive impact on reducing U-turn delay.

Table 29. Field Collected U-Turn Delay Data before and after Yield Sign Removal.

	AM				PM			
	NB-SB		SB-NB		NB-SB		SB-NB	
	Before	After	Before	After	Before	After	Before	After
Total Delay (sec)	68	297	116	358	419	651	355	354
U-turn Vol (vph)	151	222	143	308	345	604	334	350
Ave Delay (sec)	0.450	1.338	0.811	1.162	1.214	1.078	1.063	1.011
FR Vol (vph)	547	610	1380	2031	2088	2406	1111	1409
Normalized Delay (10^{-3} sec)	0.645	1.608	0.533	0.497	0.499	0.358	0.736	0.575

Field Evaluation Results—Evaluation of Research Forest Signal Timings

Research Forest Drive is a three-lane arterial intersecting I-45, which carries heavy traffic during peak periods. The existing coordinated system spans 5.5 mi, starting from Kuykendahl Rd. and going to David Memorial Dr., and consists of the I-45 diamond interchange and 18 signalized intersections, all but one of which (David Memorial Dr.) are located on the west side of I-45. The system is operated by Montgomery County using four timing plans (AM peak, mid-day peak, PM peak, and off-peak).

It is not appropriate to consider retiming the diamond interchange in isolation. Retiming the entire corridor, on the other hand, was beyond the scope of this project. As illustrated in Figure 26, researchers decided to consider retiming a smaller subset of signals along with the diamond interchange. The intersection of Grogans Mill Rd. is like a split diamond (signalized intersection of one-way roadways) and served at the point where the coordination could be split, at least for this analysis.

Image Source: Google Maps

Figure 26. Map of Research Forest Subsystem Considered for Retiming.

With assistance from TxDOT staff, researchers obtained signal timing information for these signals from the county. Even though TxDOT has implemented the timings at the diamond interchange using Texas Diamond mode in a single controller, the timing data sheets present the data for the two intersections as though each has a separate controller.

Researchers used video cameras and post-processing to obtain traffic counts for morning and afternoon peak periods. Appendix E provides these counts. During video post-processing, researchers also counted truck traffic separately, but found it to be negligible. Appendix E presents total (car plus truck) traffic counts obtained from video. For use in signal timing analysis and optimization models, researchers identified the maximum 15-minute count for each approach (highlighted entries in Appendix E tables) and multiplied the identified count by four to obtain the hourly count. The last two rows in each table present these hourly volumes.

In the first step, researchers modeled existing AM and PM timings in PASSER V-09 (P5) along with collected count data. Figure 27 shows P5 representation of the modeled system.

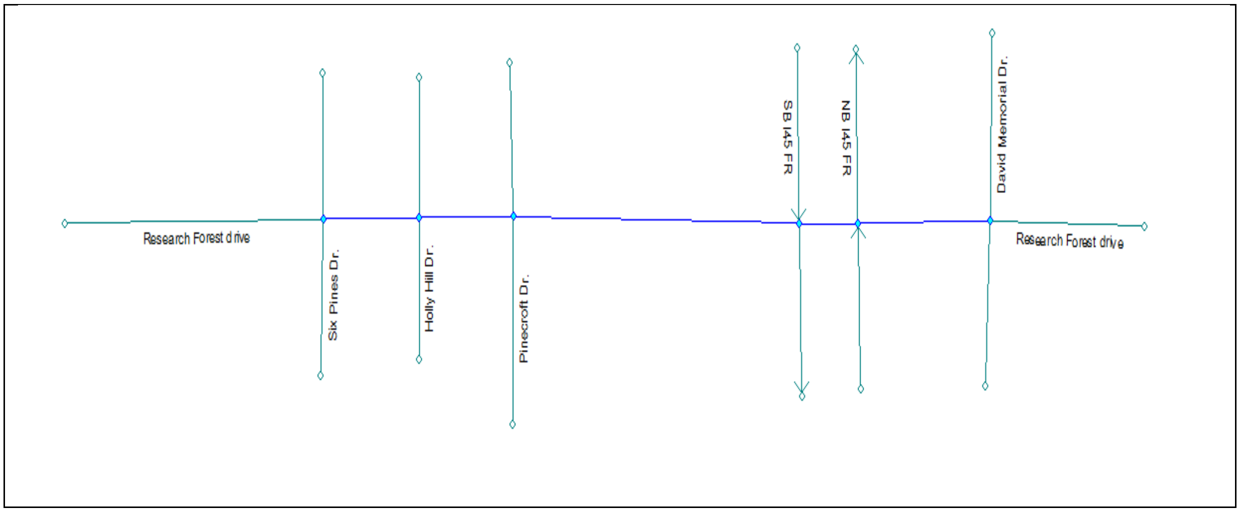


Figure 27. PASSER V-09 Representation of the Modeled System.

Figure 28 shows time-space diagrams for the existing AM and PM peak timing plans generated by P5. Notice that the AM peak plan provides a larger band for westbound traffic, and the PM peak plan provides a larger band for eastbound traffic to model observed traffic OD pattern. There is a minor difference between the interior offset between two signals of the interchange and the last signal (David Memorial Dr.) located on the west side. Also, notice that the signal phase sequences remain unchanged between the two timing plans.

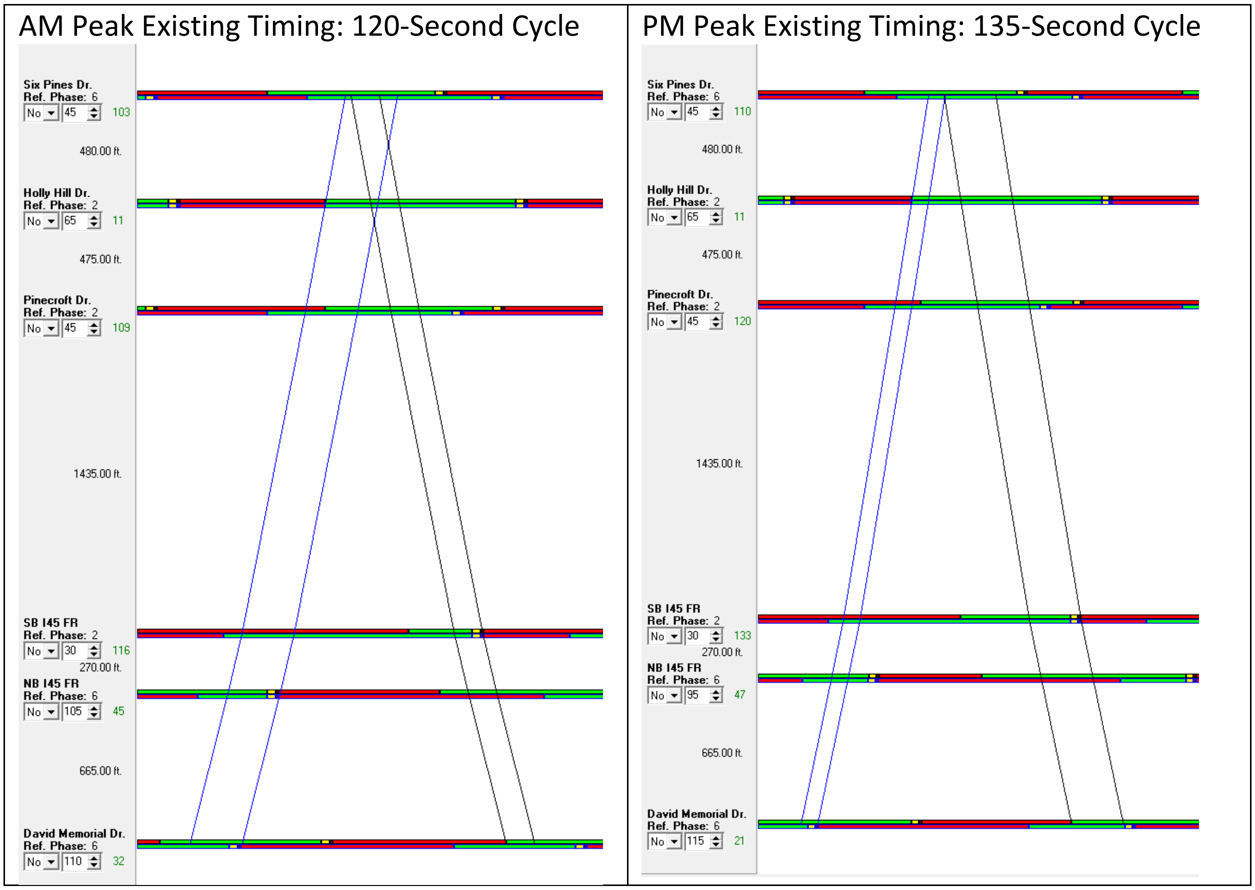


Figure 28. Time-Space Diagrams for Existing AM and PM Peak Timings.

Table 30 provides performance measures generated for the two existing timing plans by P5's mesoscopic traffic model using volume data collected by researchers. Through-progression efficiency and attainability for both these timings are much below the desired values of 20–25 percent and 100 percent, respectively. These factors are good indicators for the quality of traffic flow in signal systems where most traffic is through traffic along the entire coordinated corridor. In the case of the Research Forest Dr. signal system, a significant amount of traffic volume going westbound during the AM period enters the system from northbound left-turn and southbound right-turn movements at the interchange. Similarly, during afternoon peak, a significant amount of eastbound traffic turns right and left at the interchange, highlighting the need to use other measures such as delay and stops for signal timing evaluation. However, like efficiency and attainability, qualitative assessment of these two variables is not possible. Therefore, one must have alternates to compare.

Table 30. Performance Measures for the Two Existing Timing Plans.

	AM Peak	PM Peak
Cycle (s)	120	135
Efficiency (%)	11.67	10.74
Attainability (%)	51.85	34.94
EB Band (s)	10	22
WB Band (s)	18	7
Avg. Delay (s/v)	43.36	55.88
Total Stops (v/h)	15744	19224

To enable full assessment of the quality of signal existing timings for the selected signal subsystem, researchers performed the following two sets of optimizations using the bandwidth optimization model in P5:

- Offset optimization only (Option 1).
- Offset and phasing sequence optimization at adjacent signals only (Option 2).

Table 31 compares performance measures of these optimized timing plans against those for existing timing plans. The researchers kept cycle lengths and splits the same for all these cases to keep the subsystem timings compatible with the rest of the system in case partner agencies decide to implement optimized timings for this subsystem. Such an implementation will only require trivial offset adjustment to synchronize the subsystem with the rest of signals located on the west side. For both time periods, optimization significantly improved progression. Option 2 provided the most improvements. Results for total stops (vph) are similar. Both options resulted in reduced stops, but Option 2 provided the maximum benefit. There are no significant differences in seconds-per-vehicle delay between the three options for the two periods.

For reference purposes, Figure 29 shows AM- and PM-progression bands for Option 2, the optimization option with the most improvements. In this figure, the reader will note that as opposed to existing timings, these two timings have different phasing sequences. As in the timing sheets, researchers assumed two separate controllers for P5 optimization runs. To ensure that the offset between these two signals was equal to the travel time, they used lower speed between the signals. The minor kink on the bands is because of this change.

Table 31. Comparison of Performance Measures for Existing and Optimized Timings.

	AM Peak			PM Peak		
	Existing	**Option 1**	**Option 2**	**Existing**	**Option 1**	**Option 2**
Cycle (s)	120			135		
Efficiency (%)	11.67	17.5	21.67	10.74	15.93	30
Attainability (%)	51.85	77.78	96.30	34.94	51.93	97.59
EB Band (s)	10	17	24	22	24	51
WB Band (s)	18	25	28	7	19	30
Avg. Delay (s/v)	43.36	42.98	43.95	55.88	56.36	55.35
Total Stops (v/h)	15,744	14,719	13,548	19,224	18,822	17,553

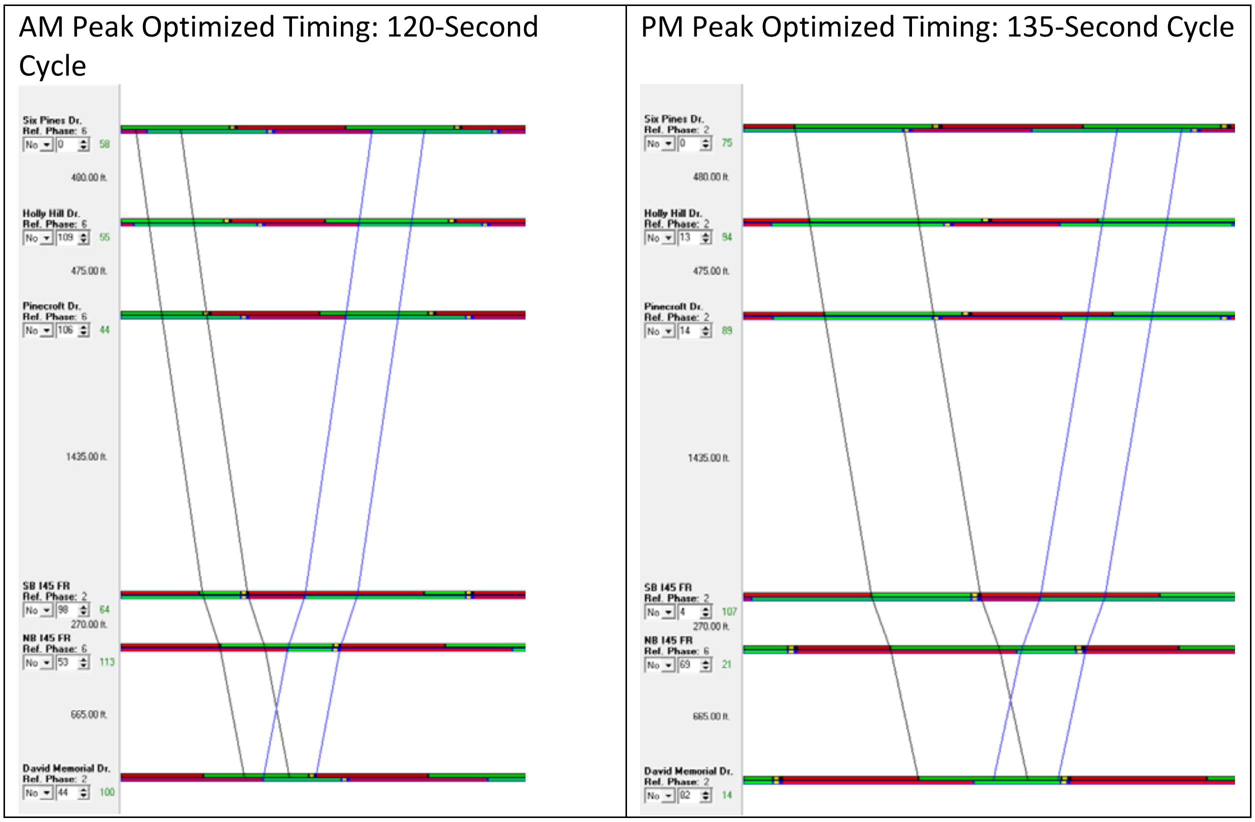


Figure 29. Time-Space Diagrams for AM- and PM Peak Optimized Timing Plans for Option 2 Optimization Runs.

Optimization results show that the Research Forest system has a good existing timing plan, but it can benefit from signal timing upgrades to improve progression and stops in the system, even though constrained optimization did not produce any benefit in terms of delay. If progression can be improved in the system and if the number of stops can be reduced, then traffic operations at the diamond can be improved. As such, U-turn operations can also benefit from these changes. However, it would be beneficial to conduct a detailed assessment using microscopic computer simulation prior to implementing signal timing changes.

Ultimately, this work was done in preparation for field implementation for the signals along Research Forest; however, the reality of coordinating with TxDOT and Montgomery County was time consuming and was too difficult to complete within the time period of this project.

CHAPTER 5. SAFETY EVALUATION OF U-TURN DESIGN

INTRODUCTION

This chapter summarizes the activities performed in Task 5 of this project. The objectives of this task were:

- To perform safety evaluations of U-turns.
- To develop a statistical equation suitable for producing a predictive safety model spreadsheet.

OVERVIEW OF SAFETY ASSESSMENT TASKS

For this research effort, researchers evaluated safety performance for U-turn designs at diamond freeway interchanges in Texas. First, researchers developed a large, randomly sampled data set for a statistically reliable assessment of U-turn safety performance at Texas interchanges. Researchers then reduced this larger sample size to a representative data set suitable for statistical analysis. Next, researchers conducted a qualitative evaluation for the study locations that were included in the companion operational analysis. Finally, the researchers conducted a statistical assessment of the safety performance at locations with and without U-turns.

Development of Study Sample

To perform this research and to have a sample representative of the overall population, the analysis required the inclusion of a randomly sampled number of interchanges with and without the dedicated U-turn configuration. At this time, TxDOT does not maintain a database that comprehensively identifies specific interchanges or intersection locations or their key road characteristics. As a result, the initial project tasks required researchers to develop a technique for identifying candidate study locations with and without U-turn configurations followed by the collection of the required supplemental data.

Researchers used existing roadway functional system information identified in the TxDOT Road–Highway Inventory Network (RHiNO) as an initial step toward identifying freeway and arterial networks where diamond interchanges could be potentially located. Table 32 depicts the combination of highway types extracted from RHiNO functional classifications where a diamond interchange could be expected to occur.

Table 32. Candidate Roadway Types in RHiNO.

ID	Highway Type 1		Highway Type 2
1	1 = Interstate	+	3 = Principal Arterial
2	1 = Interstate	+	4 = Minor Arterial
3	2 = Freeway and Expressway	+	3 = Principal Arterial
4	2 = Freeway and Expressway	+	4 = Minor Arterial

Researchers developed separate GIS-shape files based on the road categories identified in Table 32. Each road indicator also contained coordinate information that could then be used to help locate points of intersection that could potentially represent interchanges or intersections.

TransCAD is a GIS software platform that provides transportation-based mapping tools. GIS maps from TransCAD provide some intersection information, so researchers merged the RHiNO shape files with the TransCAD files as a way of initially selecting potential freeway intersection locations. The identified intersections included intersecting points as far as 200 ft (approximately 60 m) from the freeway line, but when this information was filtered using the RHiNO information, researchers were able to ensure that the intersecting points did represent intersections between freeways and arterials. Figure 30 depicts these identified intersections for the selected roadway network (freeway and arterial). In some cases, this method also identified extraneous intersecting points that do not represent interchanges, and the intersections that were identified did not have any interchange type designations.

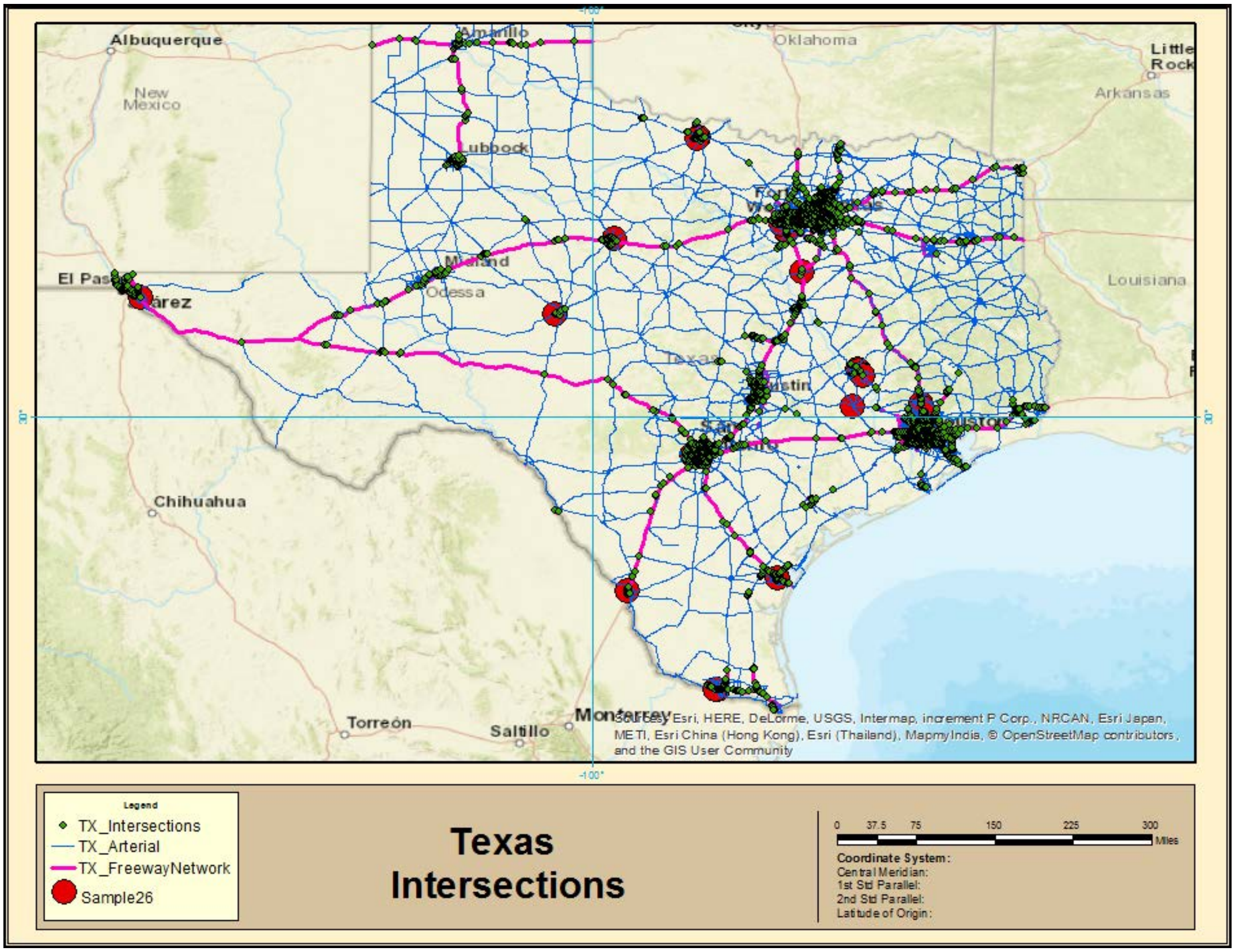


Figure 30. GIS Intersection Points for Freeways and Arterials.

Following this initial GIS screening for intersecting points, researchers next needed to reduce the large number of intersecting points (a total of 11,289 identified points) that did not represent interchanges to potential locations where an interchange was likely. To narrow the search, an intersection with multiple points (generally four to six points) in close proximity often represented some sort of interchange. As can be observed in Figure 31a, an interchange with a U-turn will normally have at least six intersecting points. Therefore, to identify these potential interchange locations, researchers applied an additional 300 ft (approximately 100 m) buffer to all intersecting points, as shown in Figure 31c, to measure the distance between two intersecting points of randomly selected interchanges. Because researchers wanted to include interchanges with and without U-turns, a diamond interchange with four or more intersection points within this buffer region represented a potential U-turn location.

a. Aerial View b. GIS Street Map View c. Applying 60 m Buffer d. Consolidated Intersection Point

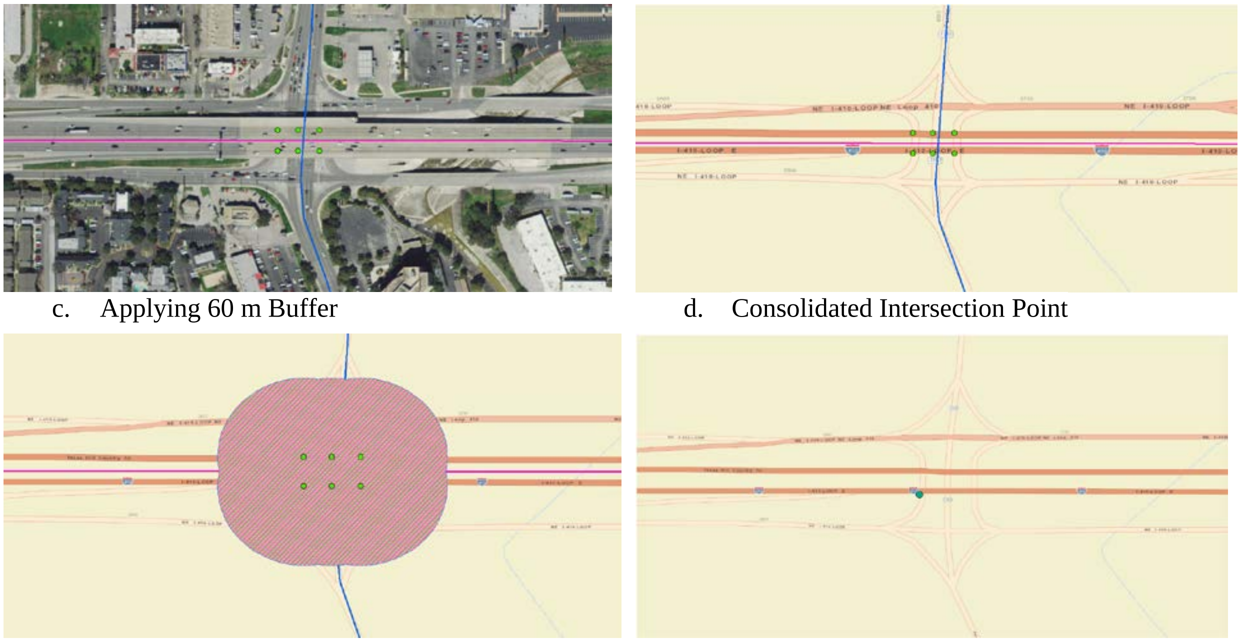


Figure 31. Locating an Interchange with a U-Turn.

Next, the researchers filtered locations and assigned a single consolidated intersecting point to represent the latitude and longitude for the potential study interchange. Figure 32 depicts an example of two identified U-turn interchanges that resulted from this process. The next step required the development of a subset of these potential interchange locations suitable for subsequent analysis.

Developing a Stratified Random Sample

The selection of representative interchange locations that can be assumed to represent the larger population requires a random selection of additional study locations. Because the state of Texas is very large, geographic representation should also be considered when developing the sample by developing a stratified random sample for diamond interchange locations from across the state. The identification of these potential sites, therefore, required multiple stages of selection, as summarized in the following sections.

Sampling U-Turns

The procedure previously summarized resulted in a total of 656 potential diamond interchanges that represent 22 TxDOT districts and 50 Texas counties, but the interchanges across the counties are not evenly dispersed. Metropolitan regions such as Dallas, Houston, and San Antonio, for example, have a larger number of diamond interchanges when compared to the western and southwestern regions. The random sample, therefore, included selection for these varying geographic regions.

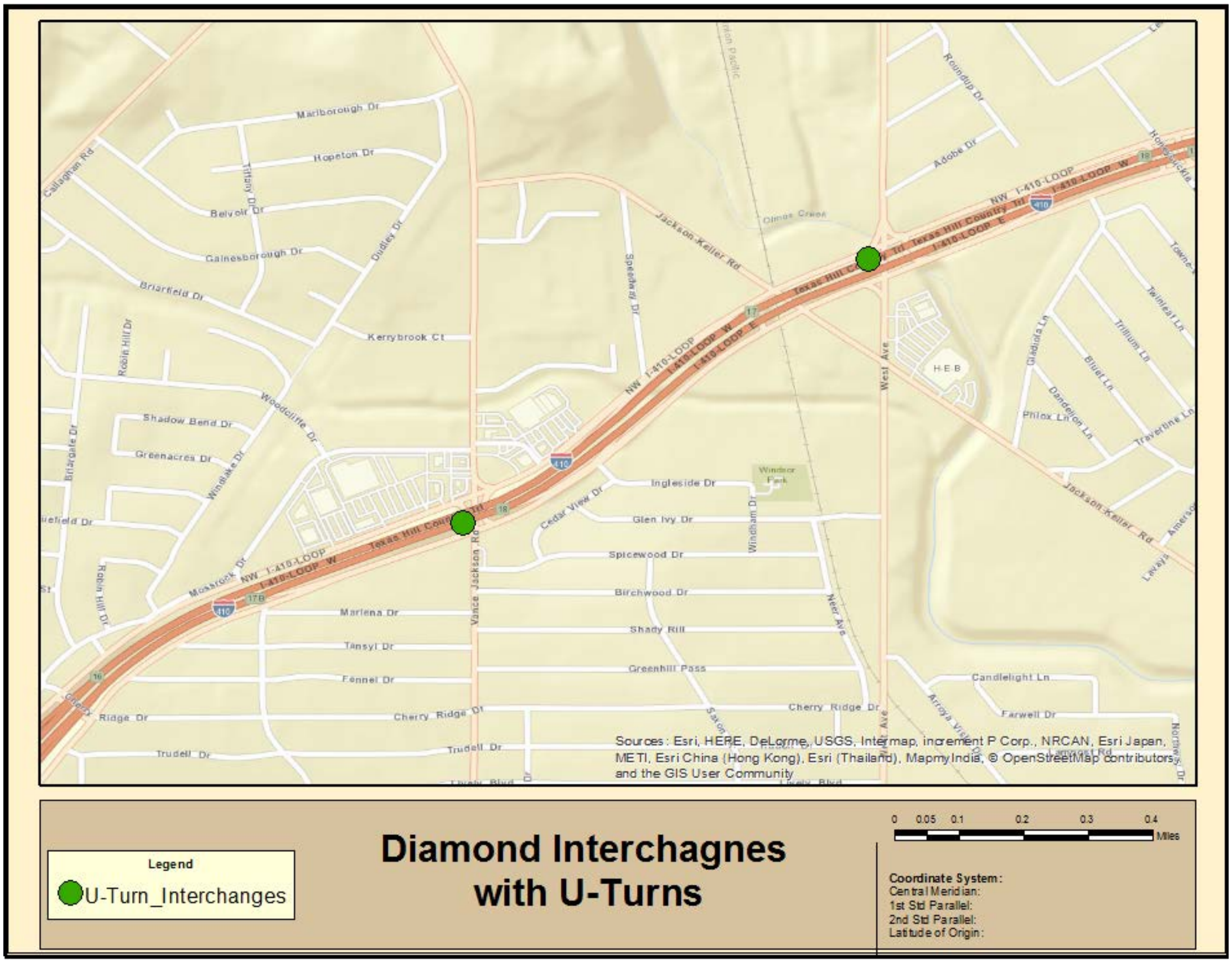


Figure 32. Diamond Interchanges with U-Turns.

The selection of a representative sample of interchanges required a two-stage stratified sampling process. First, the interchanges were divided into strata where each stratum is a county. Then, each county received a weight that represented the total percentage of candidate interchanges located within the region. Researchers then used this weighting to identify a target sample size for each county.

First Stage Stratified Sampling

As part of the sampling process, researchers elected to develop as large of a sample as possible (i.e., oversampled) so as to accommodate the removal of interchanges that were not representative of the type of configuration studied for this research effort. Based on a potential interchange population size of 656, this larger sample size included almost 450 prospective locations (using a 95 percent confidence interval and a 2.5 percent margin of error). Using this larger sample size, the county weight (previously reviewed) could then be applied. In some instances, however, a more remote county might only have one or two diamond interchanges. This condition applied to 27 of the interchange locations, so the data were divided into two groups. The first group represented these more remote interchanges and included 27 prospective interchanges, while the second group included 629 interchanges (representing counties with three or more diamond interchanges). The interchanges were then sampled from these two groups. Due to the potential inclusion of interchanges that might not be diamond configurations, researchers also noted that many of these interchanges might ultimately be filtered out during data collection activities. Consequently, researchers assigned each of the selected interchanges a discrete sample number, and the selection order was documented so that if an interchange had to be removed, the next randomly selected one could be identified and added. As a result of the first stage of stratified sampling, approximately 450 potential interchanges remained in the sample pool (see Figure 33).

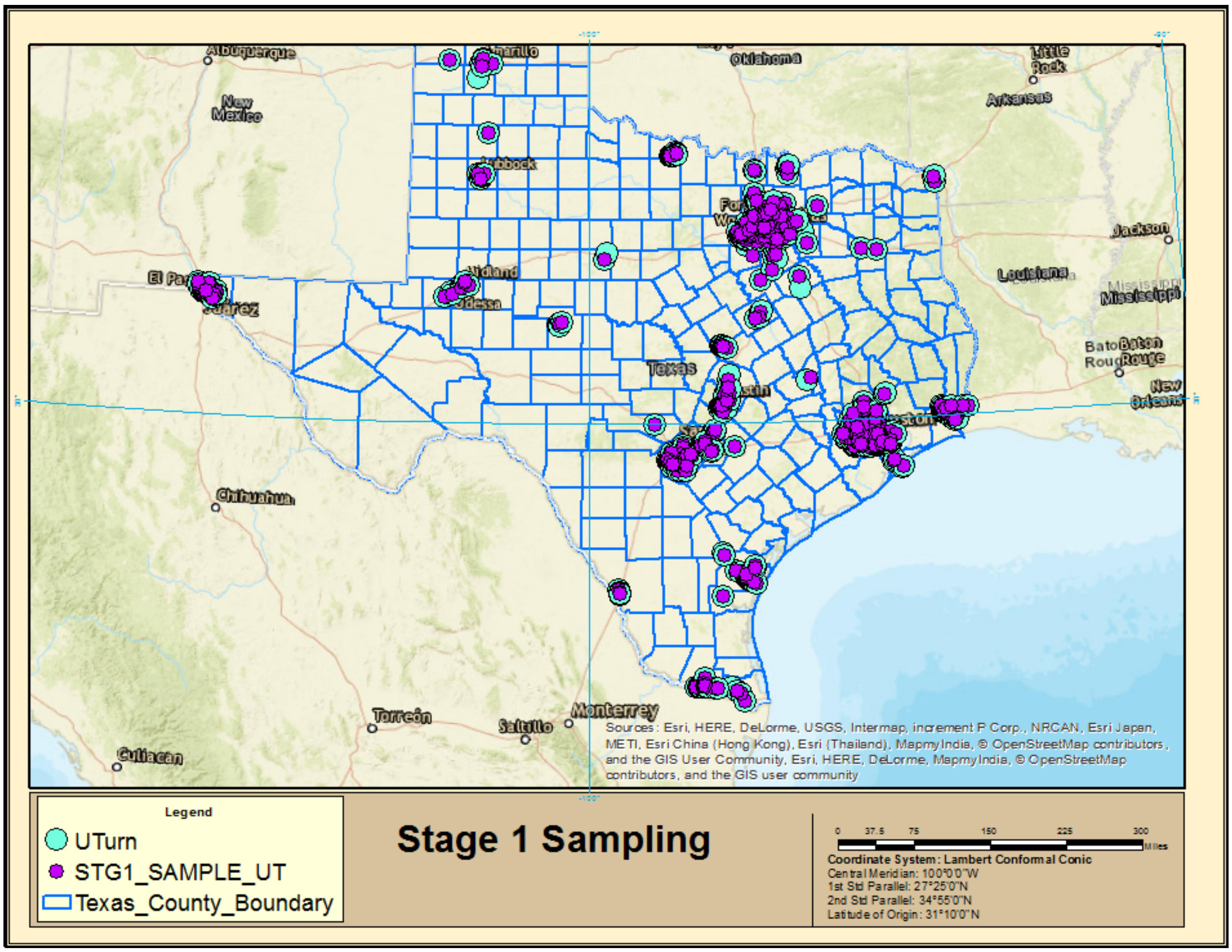


Figure 33. Stage 1 Sample Interchanges.

Second Stage Stratified Sampling

In the second stage, researchers performed a random sampling based on the TxDOT district level. Since the interchanges are not uniformly dispersed across the districts, four district categories were defined as follows:

- Districts with one to 10 interchanges.
- Districts with more than 11 and less than 40 interchanges.
- Districts with more than 41 and less than 60 interchanges.
- Districts with more than 61 and less than 130 interchanges.

After ensuring geographic representation within the dataset by district, the remaining sample included diamond interchanges representing 19 districts and 32 counties.

Final Sample

The final stratified random sample included 168 diamond interchanges with U-turns and 60 prospective diamond interchanges without U-turns (see Figure 34). Researchers used this data set for the subsequent site selection and data collection activities.

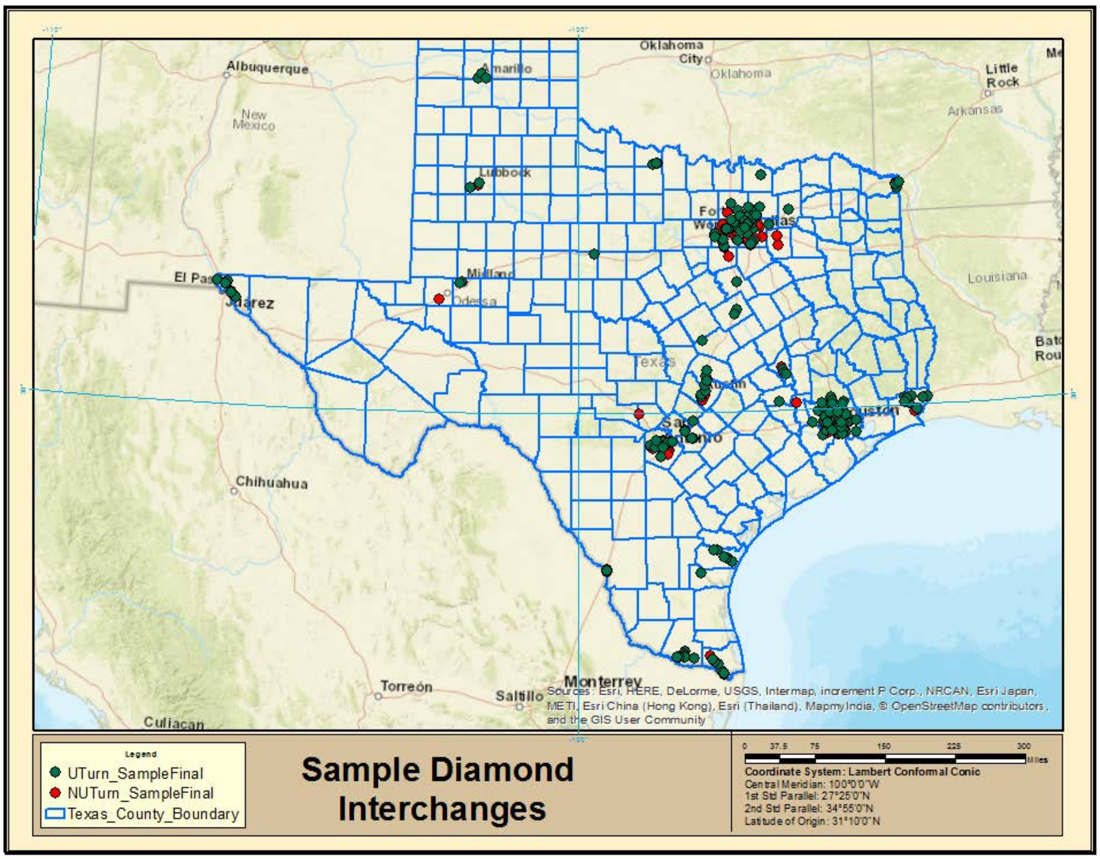


Figure 34. Diamond Interchange Sample.

DATABASE DEVELOPMENT

Following the identification of the prospective study sites, researchers used the Task 4 operational study sites as well as the randomly selected sites for the overall U-turn safety assessment. The data collected for each site included geometric data acquired from the TxDOT RHiNO database and additional data acquired using aerials from Google Earth Pro®. In addition, the database included average annual daily traffic (AADT), the K-factor (for converting daily traffic volume proportions during peak hours), and the directional distribution factor known as the D-factor. This D-factor proved to be a valuable way to confirm one-way FR operations. In addition, researchers assigned crash data acquired from the Texas Crash Records Information System (CRIS) for the years 2009 to 2015.

The randomly selected study sites included a mixture of locations with and without U-turns. For locations with U-turns, researchers reviewed historic aerial photographs to confirm the presence of the U-turn for the entire seven-year period. For locations where the U-turn construction occurred during this study period, the researchers removed the crash years that could not be confirmed. For example, if an aerial photograph indicated a U-turn was not present in 2010 but was present in 2013, the crash data for 2011 and 2012 were not considered. In addition, depending on the date of the aerial photograph, 2010 and 2013 crash data may have also been removed if the before-site aerial was not in December or the after-site aerial was not in January of the study year.

Site-Specific Data

Researchers collected various site-specific variables, including items such as the configuration of on- and off-ramps, posted speed limit, width of the U-turn, and U-turn turning radius. Appendix F identifies and defines the collected data variables. The overall data set included 168 sites, though some data elements could not be acquired for all locations. For example, the width at the middle of the U-turn, in some instances, occurred under a bridge and so could not be measured. As the subsequent analysis evolved, researchers handled this issue by varying the data set size based on the specific variables included in the models.

Ultimately, researchers selected 108 sites with U-turns and 60 sites without U-turns for inclusion in the analysis. In some cases, an interchange location only had a U-turn on one side of the cross street. Similarly, the U-turn configurations that did occur on both sides often had very different geometric characteristics. Consequently, the analysis considered each unique U-turn configuration by collecting the geometric and volume characteristics for each U-turn (resulting in two potential U-turns to study at many of the sites). In the final data set, 14 sites out of the 108 sites had a U-turn on a single side. Figure 35 shows an example of this type of site with a single U-turn.

Figure 35. Example Interchange with a U-Turn on only One Side.

Some of the characteristics of the interchanges included in this study could potentially influence the safety performance of the location based on the unique U-turn configuration. To explore these issues in more depth, researchers assessed the descriptive statistics for the following data elements:

- Number of total lanes on the FR(s).
- Distance between the first downstream driveway and the U-turn exit.
- Traffic control (with or without a traffic signal) at the study intersections.
- Arterial right-turn treatment.
- Depressed or elevated U-turn configuration.
- Distance between stop lines (i.e., interchange interior spacing).
- U-turn leg dimensions (widths and lengths).

In some cases, the data reduction process could not determine all site characteristics for a location. Often, this constraint was a result of the interchange configuration. For example, U-turn leg dimensions could not always be determined at underpass turnaround locations due to occlusion from the structure. Researchers elected to retain these sites and use a varying sample size during the statistical analysis stepwise variable assessment. The following sections review each of these roadway characteristics.

Number of Total Lanes on the Frontage Road(s)

For each side of a study site (referred to from this point forward as a half site), the data set included the number of FR main lanes. Table 33 summarizes the number of lanes on the FRs based on locations with and without U-turns present. As shown, most FRs had two to three lanes adjacent to the U-turn locations.

Table 33. Number of Lanes on the Frontage Roads.

Half Site Condition	Number of Lanes—First Leg Frontage Road					Number of Lanes—Second Leg Frontage Road				
	1	2	3	4	No Frontage Road	1	2	3	4	No Frontage Road
U-turns Present	8	86	112	10	0	4	87	118	7	0
No U-turns	19	66	26	5	4	4	65	16	0	35

The measurement for the number of lanes for each frontage road occurred at each intersection approach, as designated by the FR_A and FR_B shown in Figure 36.

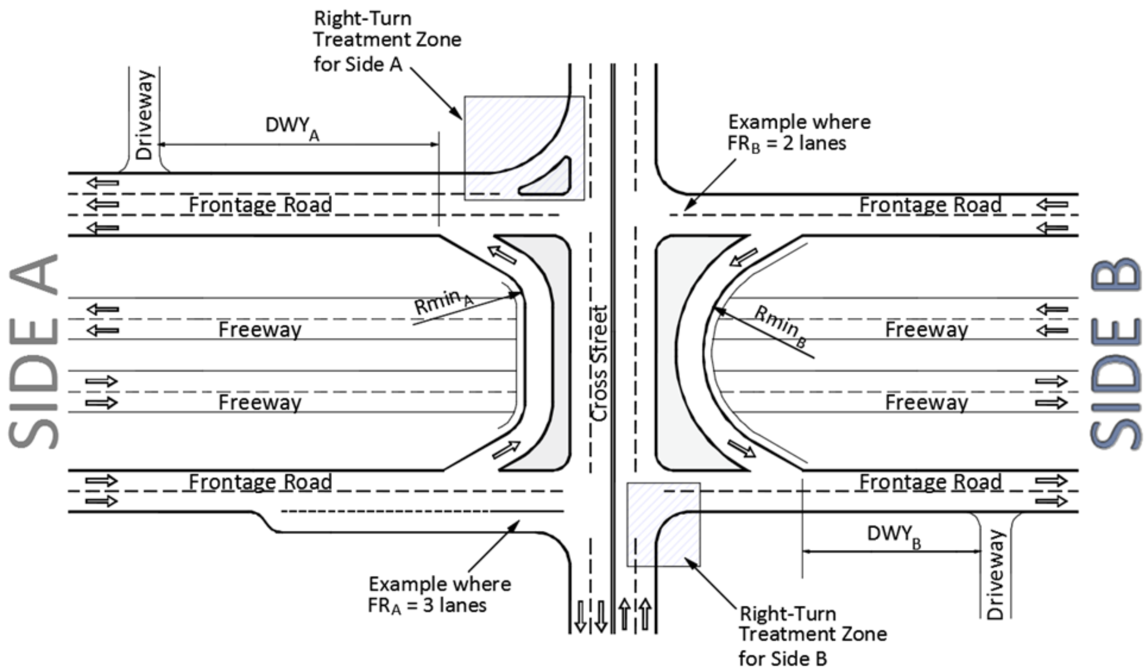


Figure 36. Turnaround Configuration and Influential Site Characteristics.

Longitudinal Distance between the U-turn Exit and the First Downstream Driveway

The longitudinal distance to the first downstream driveway, measured from the point on the FR where the U-turning vehicles exit the U-turn and merge onto the FR, can influence weaving and merging conditions on the FR. This value is represented as DWY_A and DWY_B in Figure 36. For this study, researchers measured this distance for the driveways that were within 500 ft of the U-turn exit. Out of 202 ((108 × 2) − 14) half sites, 53 of the locations had driveways positioned outside of this 500 ft threshold. Researchers assigned a default DWY value of 500 ft for these locations.

Presence of Traffic Signal at Study Intersection

Researchers used aerial photographs and the companion StreetView tool to determine the type of traffic control present at each FR and cross-street intersection location. At locations with a traffic signal, this method did not permit acquiring signal timing information. Approximately 93 percent of the intersections had traffic signals. Ultimately, researchers included only signalized intersection locations in the subsequent safety assessment.

Arterial Right-Turn Treatment onto Frontage Roads

Because a vehicle that is turning right from the cross street onto the one-way FR may potentially encounter a conflict with a vehicle exiting the U-turn, researchers categorized the configuration of the cross-street right-turn treatment zone for each one-way FR location. Each configuration included the following two variables:

- Right-turn treatment zone entrance (see Figure 37).
- Right-turn treatment zone exit (see Figure 38).

Table 34 and Table 35 summarize the distribution of the right-turn configurations for the study sites. Figure 39 and Figure 40 graphically depict this right-turn distribution.

Table 34. Right-Turn Treatment Zone Entrances.

Right-Turn Treatment Zone Entrance Configuration	Number of Half Sites		Total Number
	With U-Turn	Without U-Turn	
Shared Right, No Island (Option A)	44	37	81
Exclusive Right, No Island (Option B)	44	25	69
Exclusive Right, Painted Island (Option C)	6	2	8
Exclusive Right, Raised Island (Option D)	51	13	64
Shared Right, Raised Island, Large Radius (Option E)	52	44	96
Subtotals:	**197**	**121**	**318**
Alternative Configurations	6	12	18
Grand Total	**203**	**133**	**336**

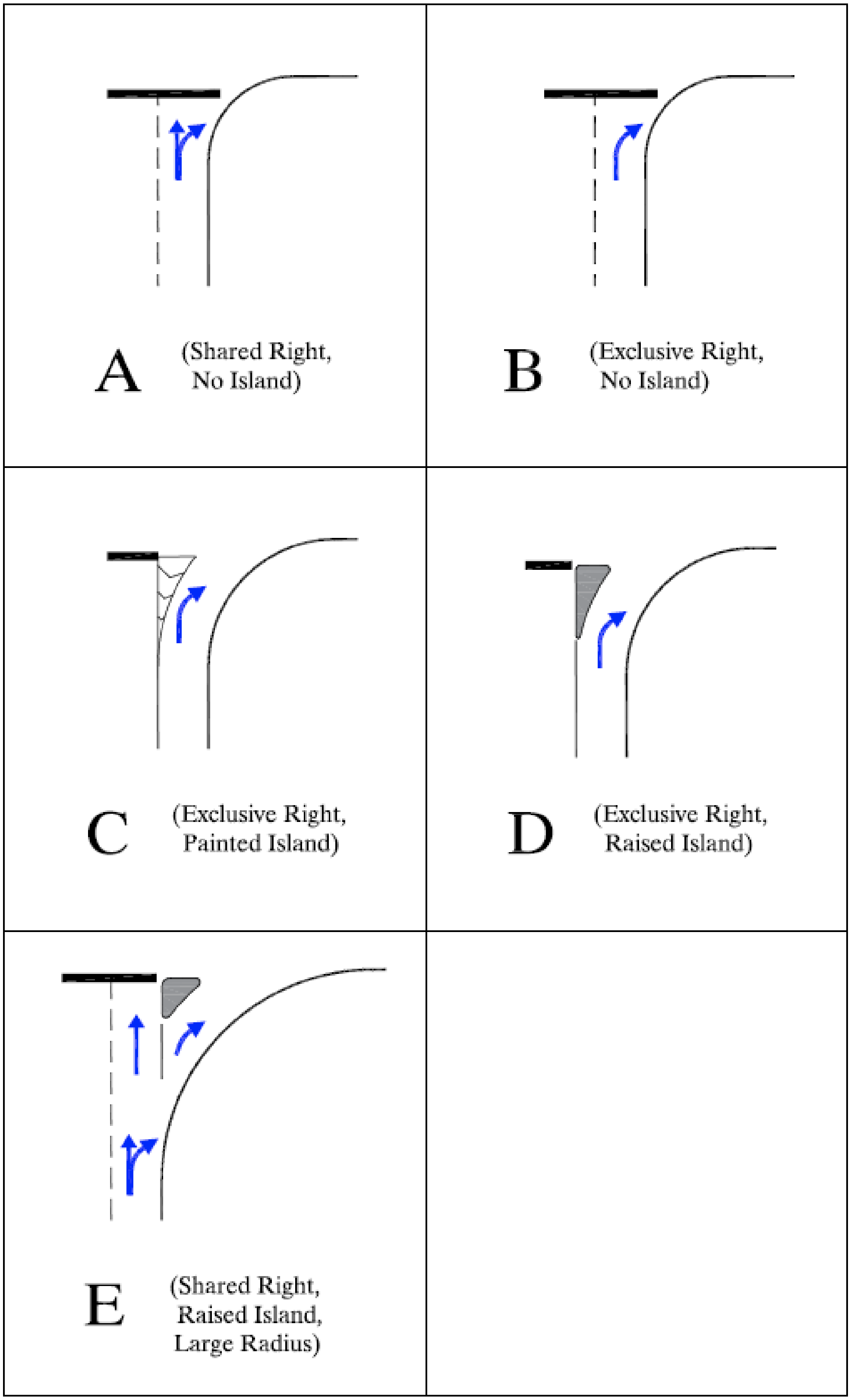


Figure 37. Right-Turn Entrance Options.

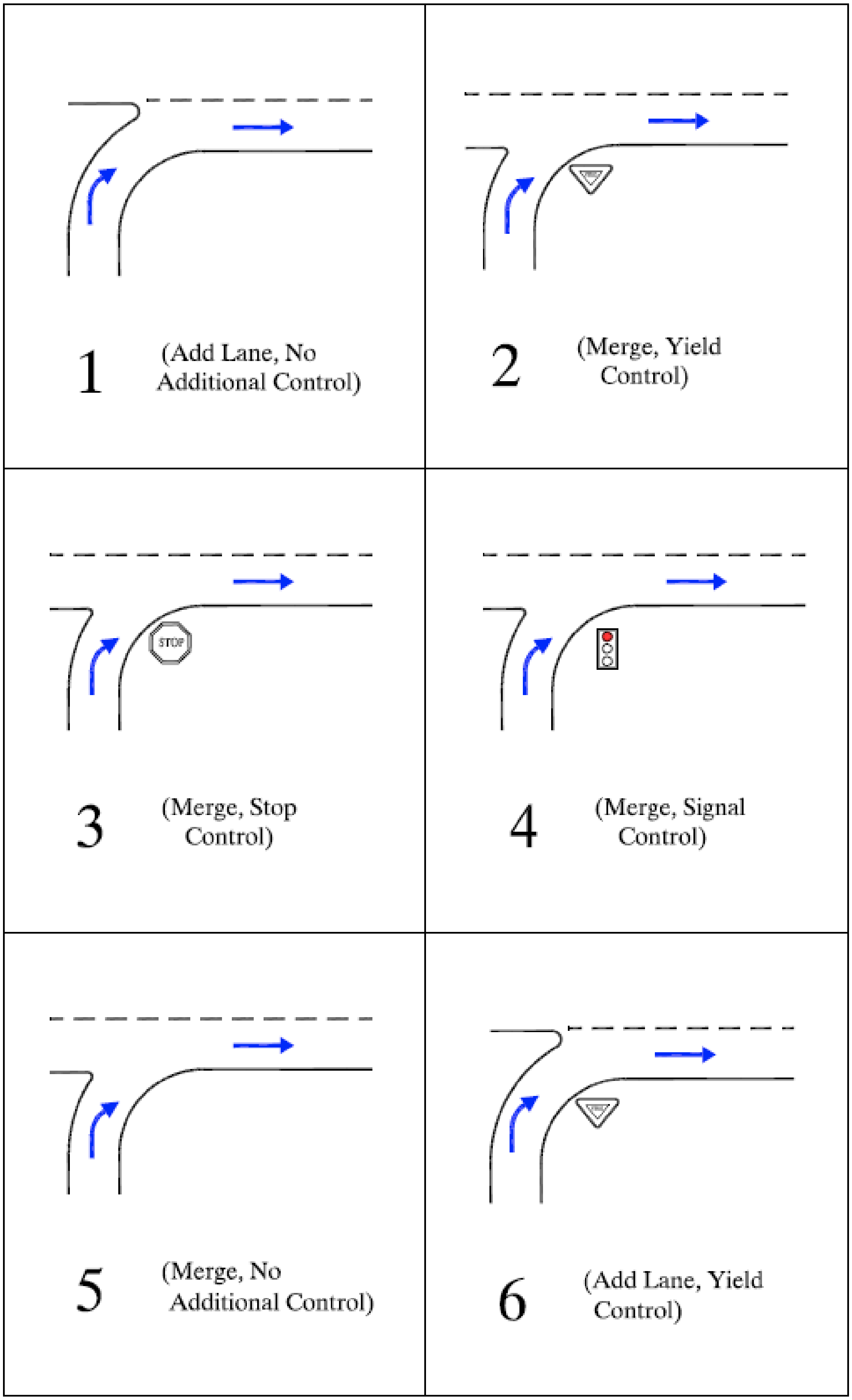


Figure 38. Right-Turn Exit Options.

The letter designations in Figure 39 refer to the right-turn treatment zone options previously defined in Figure 37. Similarly, the numeric designations depicted in Figure 40 are aligned with the right-turn exit options previously in Figure 38.

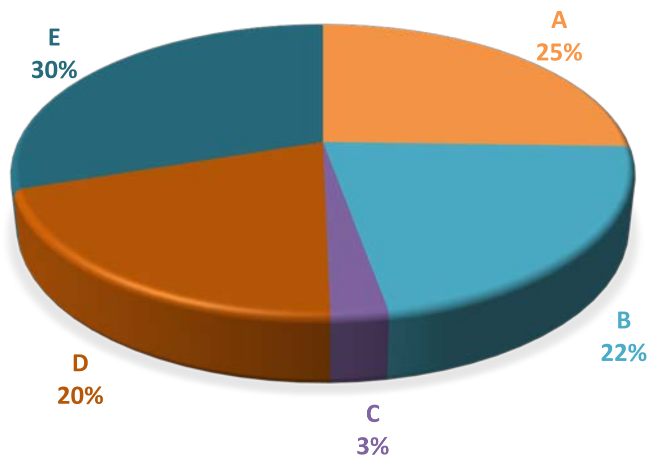


Figure 39. Distribution of Cross-Street Right-Turn Treatment Zone Entrance Options.

Table 35. Right-Turn Treatment Zone Exit Configurations.

Right-Turn Treatment Zone Exit Configurations	Number of Half Sites		Total Number
	With U-Turn	Without U-Turn	
Add Lane, No Additional Control (Option 1)	6	12	18
Merge, Yield Control (Option 2)	92	48	140
Merge, Stop Control (Option 3)	5	3	8
Merge, Signal Control (Option 4)	64	10	74
Merge, No Additional Control (Option 5)	19	47	66
Add Lane, Yield Control (Option 6)	10	1	11
Subtotals:	**196**	**121**	**317**
Alternative Configurations	7	12	19
Grand Total	**203**	**133**	**336**

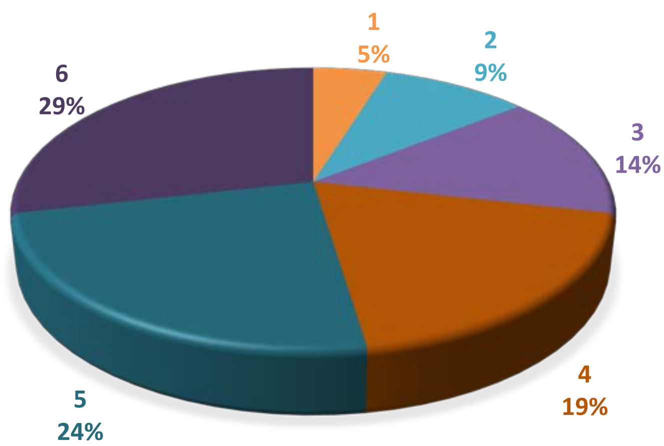


Figure 40. Distribution of Cross-Street Right-Turn Exit Options.

Depressed or Elevated U-Turn Configuration

Of the 336 half site locations, 300 (150 of 168 sites) of them had turnarounds located below a freeway bridge, while 36 (18 of 168 sites) were elevated above the freeway. Though the placement of elevated versus depressed did not prove to be significant to intersection safety, the turnaround geometry was directly influenced by these factors.

U-Turn Leg Dimensions (Widths and Lengths)

Due to the orientation of the U-turns, interchange bridges, and similar characteristics, researchers acquired, where feasible, the turning radius for the U-turn, the turning bay length, and the lane length. Though researchers evaluated the significance of all of these candidate variables, the minimum U-turn radius proved to be the only critical variable that influenced safety performance. Figure 41 graphically depicts these dimensions. Table 36 summarizes the U-turn dimension data.

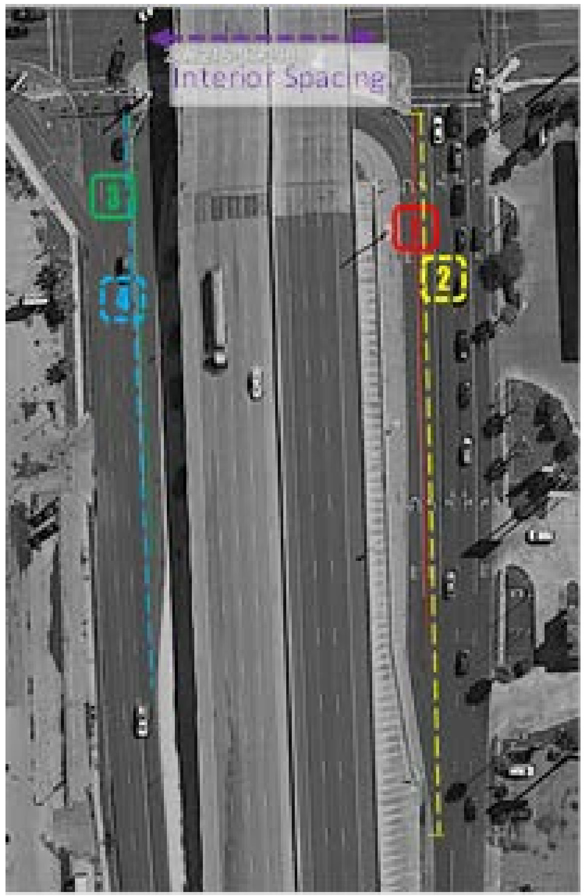


Figure 41. U-Turn Leg 1 and Leg 2 Interior Spacing.

Table 36. Characteristics for U-Turn Leg 1 and Leg 2.

Measurement Value		Lane Width (ft)	Radius (ft)	Diverging / Merging Length (ft)	Turning Bay Length (ft)
Leg 1	Minimum	8.90	20	86	144
	Maximum	29	155	577	724
	Mean	12.8	63.6	244.7	356.2
	Standard Deviation	2.8	24.1	93.7	106.8
	Count	197	201	153	153
Leg 2	Minimum	9.7	25	25	42
	Maximum	21.4	165.0	388.0	579.0
	Mean	13.0	62.5	165.6	312.6
	Standard Deviation	2.1	22.7	101.6	143.1
	Count	51	198	16	16

Matching the Crash Data

A critical step toward the development of the analysis database that contains both site and crash information is to match the crash data to the appropriate site location. In some cases, the coordinates where the crash occurred are known, and this matching is relatively straightforward. In other cases, however, the specific crash location coordinates are not available and the data require additional matching techniques. The following sections review these two matching techniques for linking the crash to the associated study location.

Crash Data with Known Coordinates

In recent years, the quality of the latitude and longitude for individual Texas crashes has improved substantially. For locations with this type of information available, researchers used the ArcGIS® software and geographically matched the crash data to the study site. Because researchers initially identified each pair of (U-turn) study sites with a single latitude and longitude interchange value, the critical issue for this analysis was to identify a boundary for the study region. Researchers graphically depicted this value of 300 ft in each direction (from the respective intersections) by drawing a line using Google Earth™ to represent the effective length of the study site. This technique is depicted in Figure 42 and illustrates the measurement used to define the effective length of the study site (in yellow) and the final line drawn (in orange) used by researchers to define the final buffer.

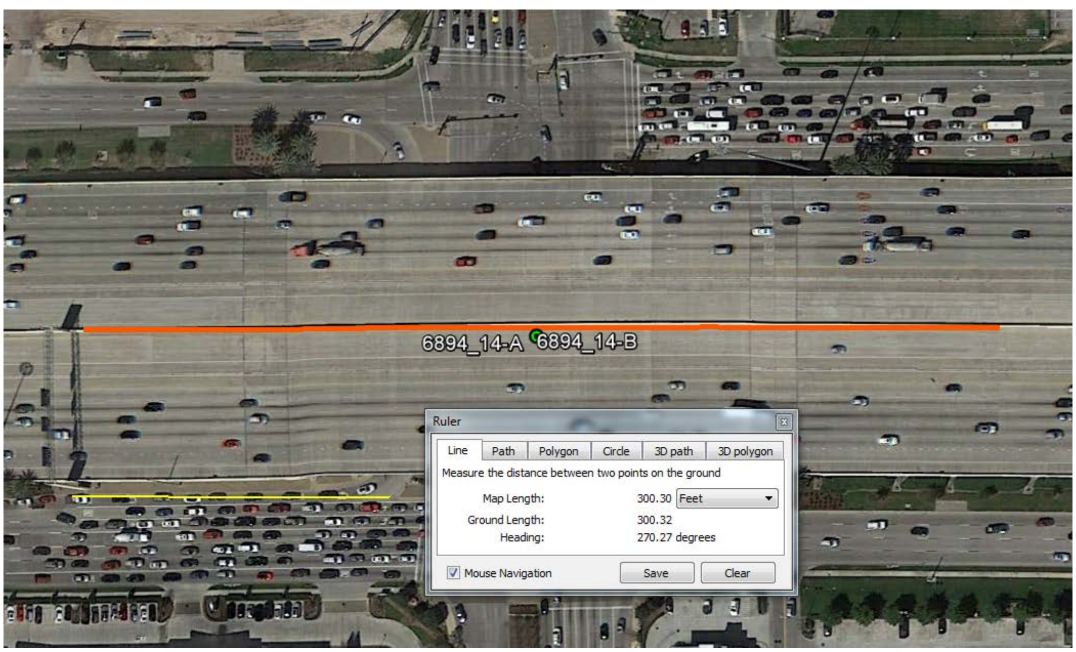


Figure 42. Effective Length of the Highway Used to Define Buffers around Study Sites.

Following this step, researchers developed a rectangular buffer that could be used to identify crashes within 300 ft upstream and downstream of the cross road. In a few instances, a skewed interchange configuration enabled crashes beyond this threshold to be captured, so researchers added a supplemental filter by applying a circular buffer with the center point defined around the center of each intersection location so as to rule out non-relevant crashes at these skewed locations.

Crash Data without Known Coordinates

Though some of the crash data did not have known coordinates, linear referencing information as well as street address data provided insights into how to identify where some of the crashes may have occurred. An additional challenge with determining where a crash occurred resulted from differences in the city names used for the CRIS city codes and those codes included in the RHiNO database. To match data from the two data sources, researchers converted the study sites' RHiNO-based city codes into the CRIS city codes. (These data are available in a data dictionary at http://www.txdot.gov/government/enforcement/data-access.html.) In a few instances, the data extracted from RHiNO did not include some of the intersecting highway data. Researchers manually completed this missing data.

To then match the crash data to each study site, researchers developed a VBA code in Excel® that used the city and two intersecting highways as a way of linking this additional crash data. This matching process enabled the addition of 977 crashes (extending over the 7-year period from 2009 to 2015). However, as shown in Table 37, these additional matched crashes were a very small percentage of the overall database.

Table 37. Number of Filtered Crashes Relative to Total Number of Annual Crashes.

Year	Crashes with coordinates	Matched crashes	Percent	Crashes without coordinates	Matched crashes	Percent
2009	423,932	3975	0.94%	99,567	21	0.02%
2010	400,302	3578	0.89%	71,986	42	0.06%
2011	385,697	3579	0.93%	70,311	33	0.05%
2012	424,275	3848	0.91%	71,369	30	0.04%
2013	440,070	4174	0.95%	79,326	66	0.08%
2014	469,456	4673	1.00%	83,408	76	0.09%
2015	540,687	5310	0.98%	57,666	51	0.09%
		Average	0.94%		Average	0.06%

Finally, researchers extracted traffic volume data from the RHiNO database. To confirm that the FRs for the study sites were selected correctly, researchers used ArcGIS® to evaluate each study site and deleted the irrelevant links remaining in the buffers. Also during this AADT matching process, researchers re-organized the data to link this information to each half site. In addition to the AADT, researchers added the D-factor and the K-factor to the database for each year. Note that a D-factor (or directional distribution) with a value of 1.0 indicates that the facility is a one-way road. When comparing sites with and without U-turns, this information is important to verify analysis of similar FR configurations.

CROSS-SECTIONAL QUALITATIVE ANALYSIS

The inclusion of statistical procedures as a means of determining safety performance can be a critical analysis step when a researcher is attempting to confirm how a facility performs under varying conditions; however, a qualitative analysis, where feasible, should be conducted so that the presence of trends in the data can be identified separately as part of this independent assessment. For this qualitative analysis, researchers used the 26 operational study sites evaluated for operational performance during the Task 6 activities and evaluated these study locations for a detailed site-specific qualitative safety assessment. This evaluation included the following three basic steps:

- Compile site-specific summary information for the 26 operational study sites.
- Examine crash types for before/after locations to identify potential safety trends.
- Develop summary statistics that examine crash severity and crash type for locations with and without U-turns.

The following sections provide additional details related to these three qualitative assessments.

Compile Site-Specific Summaries

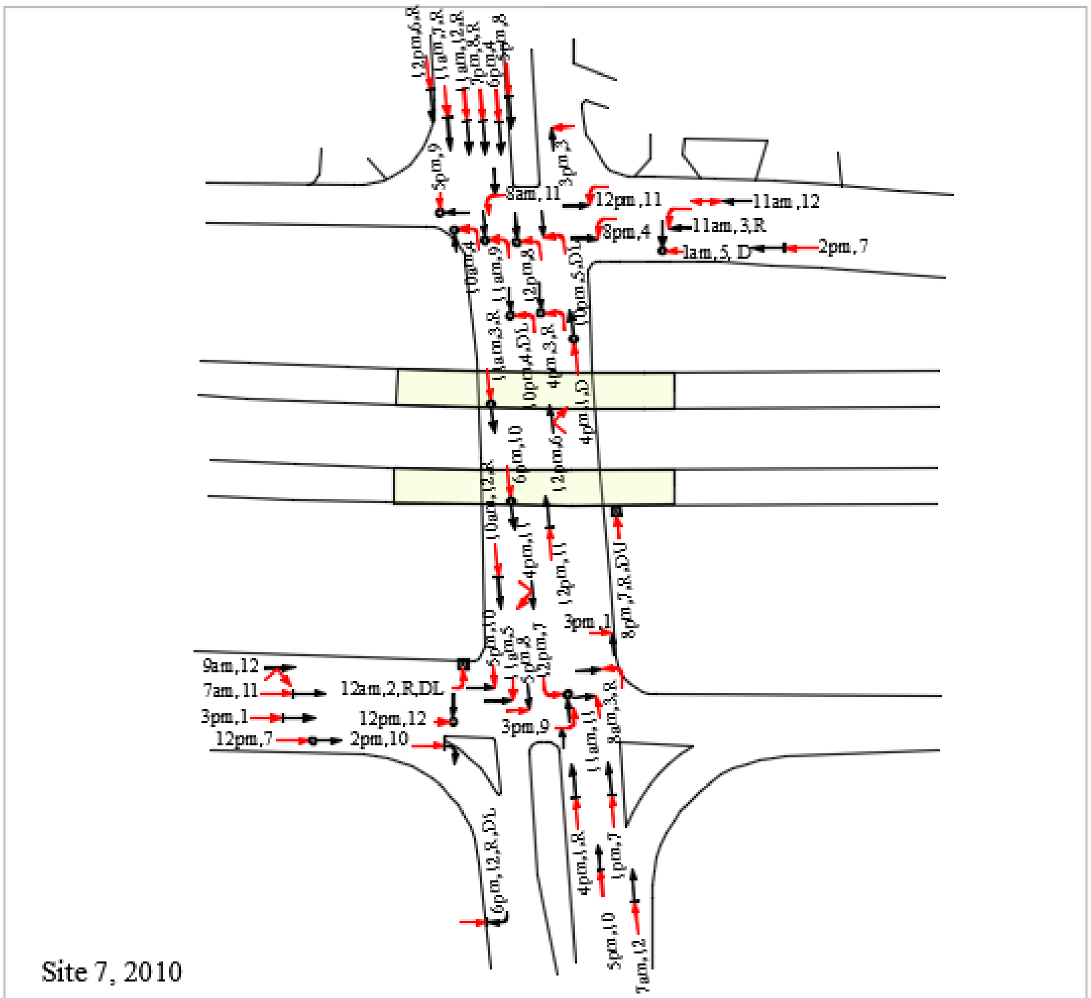
First, researchers compiled site-specific summary details that included site information, crash severity data, and a more in-depth look at how the left-turn maneuver appears to influence the crash condition for freeway interchange locations with and without U-turns. The resulting variable descriptions are summarized in Appendix F. Appendix G includes the detailed summaries of crash data from each site. During this process, researchers noted that the interchange configuration at Site #21 has an atypical configuration. The other locations researchers evaluated were located at diamond interchanges with and without U-turns. The Site #21 configuration included a loop and had a configuration similar to that of a partial cloverleaf. Consequently, researchers did not further evaluate Site #21 as part of the qualitative safety assessment.

For each remaining location, the summaries included in Appendix G incorporated the four following tables:

- Summary of site conditions.
- Summary of crash severity.
- Summary of left-turn crashes.
- Review of left-turn crashes and where they originated.

Examine Crash Types for Potential Safety Trends

Next, researchers evaluated safety performance at locations where before and after conditions could be assessed. While conducting this activity, researchers began to notice that the number of left-turn crashes originating from the FRs appears to be smaller at locations with U-turns than at locations without U-turns. To further demonstrate this observation, Figure 43 depicts a Site #7 collision diagram for 2010. At that time, the interchange did not have any U-turns. In contrast, Figure 44 depicts the collision diagram for the same site in 2015. Based on an evaluation of archival aerials at this study site, an aerial from January 2011 did not show a U-turn, but by April 2012, the U-turns were constructed and open to traffic.



LEGEND

Collision Type:

- Rear-End
- Backing
- Angle
- One Turning - Same Direction
- Both Turning - Same Direction
- One Turning - Opposite Direction
- Sideswipe - Same Direciton
- Sideswipe - Opposite Direction
- Single Vehicle - Fixed Object
- Head-On
- Single Vehicle

Severity:
- PDO or No Injury
- Injury
- Fatal

Weather:
Default = Clear or cloudy
S = Snow/Ice
R = Rain
O = Other

Light:
Default = Daylight
DN = Dawn
DU = Dusk
DL = Dark but lighted
D = Dark

Notes for Interpreting Labels:
4pm, 1 - The crash occurred at 4 p.m. in the month of January.
12pm, 9, R, D - The crash occurred as 12 p.m. in the month of September during rainy weather conditions, at a dark location (no lights).

Figure 43. Collision Diagram for Site #7 before Condition (2010 Example).

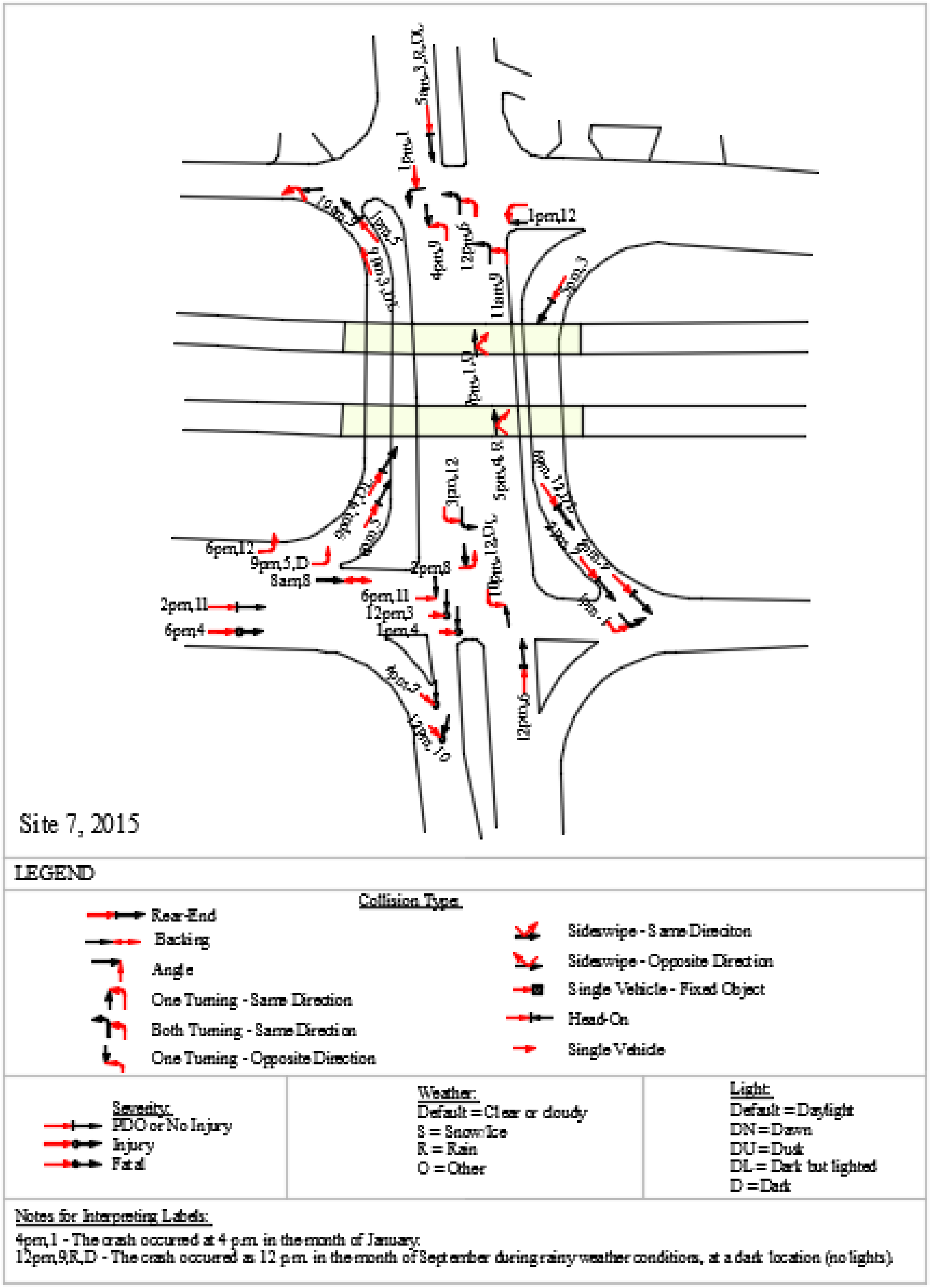


Figure 44. Collision Diagram for Site #7 after Condition (2013 Example).

By a simple inspection of the two collision diagrams at the same site, it is clear that shifting the FR U-turning traffic (and effectively removing two left-turn maneuvers from the cross street) results in a change in the number of these potentially severe turning maneuvers.

Develop Summary Statistics for Crash Severity and Type

The third and final qualitative analysis task contrasted the detailed site-specific findings and combined them into sites with no U-turns, sites with only one U-turn (this situation only occurred at two of the 26 sites), and locations with two U-turns. Table 38 summarizes the crash severity findings. Because shifting the FR left-turn maneuvers from the cross street to the U-turn can be expected to relocate the more severe left-turning vehicles (and eliminate two left turns on the cross street), the evaluation of crash severity may help to qualitatively determine if U-turns appear to improve safety. The severe crashes that involve a fatality (K), an incapacitating injury (A), or a serious injury (B) can be expected to be lower at locations with U-turns than at locations without. By inspection of the Table 38 crashes, the percentage of the K + A + B (severe) crashes at the remaining 25 operational study sites are as follows:

- 10.8 percent of the total crashes that are severe occurred at locations with no U-turns.
- 9.9 percent of the total crashes that are severe occurred at locations with only one U-turn.
- 7.2 percent of the total crashes that are severe occurred at locations with two U-turns.

Though this observation only applies to a limited number of study sites, this type of qualitative finding can be helpful in validating statistical analyses.

Table 39 further summarizes the percentage of left-turn crashes based on the U-turn configuration. The presence of one or more U-turns can be expected to reduce the number of left-turning vehicles at the cross- street intersections. As noted in this table, the percent of left-turn crashes are summarized as follows:

- 26.2 percent of the total crashes involved left-turning vehicles at locations with no U-turns.
- 18.7 percent of the total crashes involved left-turning vehicles at locations with only one U-turn (note that there are only two of the 26 sites that have this condition).
- 21.8 percent of the total crashes involved left-turning vehicles at locations with two U-turns.

These qualitative findings further indicate that locations with U-turns tend to have overall fewer crashes involving left-turning vehicles. Because these percentages represent all left-turning crashes, an additional expectation may be that the number of left-turning crashes that originate on the FRs would similarly be reduced with the construction of U-turns (see Table 40).

Table 38. Crash Severity Summary at Operational (Task 4) Study Sites.

Site	Time Period	Number of Fatal or Serious Injury Crashes			Total Site Crashes	K + A + B Percentage
		K	A	B		
No U-Turn						
1a	2009–2012	0	1	8	106	8.5%
2	2009–2015	0	3	13	217	7.4%
3	2009–2015	0	3	24	336	8.0%
4	2009–2015	0	0	35	195	17.9%
5	2009–2012	0	1	0	31	3.2%
7a	2009–2010	0	0	10	91	11.0%
10	2009–2015	1	1	3	26	19.2%
					Average for Sites:	**10.8%**
U-Turn (One Side Only)						
11	2010–2015	1	6	16	245	9.4%
12	2009–2015	0	7	18	241	10.4%
					Average for Sites:	**9.9%**
U-Turn (Both Sides)						
1b	2015	0	0	2	34	5.9%
6	2009–2015	0	2	12	78	17.9%
7b	2013–2015	0	0	13	134	9.7%
8	2009–2015	0	1	11	160	7.5%
9	2011–2015	0	3	9	187	6.4%
13	2009–2015	0	3	20	210	11.0%
14	2009–2015	1	3	34	360	10.6%
15	2009–2015	0	1	9	167	6.0%
16	2009–2015	0	2	19	326	6.4%
17	2009–2015	1	8	27	499	7.2%
18	2009–2015	0	1	3	200	2.0%
19	2009–2015	0	1	36	319	11.6%
20	2009–2015	0	0	9	809	1.1%
22	2009–2015	0	4	17	266	7.9%
23	2009–2015	1	9	32	637	6.6%
24	2009–2015	0	0	0	22	0.0%
25	2009–2015	0	0	8	144	5.6%
26	2009–2015	0	1	7	121	6.6%
					Average for Sites:	**7.2%**

Table 39. Percent of Left-Turn Crashes at Operational Analysis Sites with and without U-Turns.

Site	Time Period	Total Crashes	Number of Left-Turn Crashes	Percent Left-Turn Crashes
No U-Turn				
1a	2009–2012	106	49	46.2%
2	2009–2015	217	47	21.7%
3	2009–2015	336	66	19.6%
4	2009–2015	195	22	11.3%
5	2009–2012	31	13	41.9%
7a	2009–2010	91	25	27.5%
10	2009–2015	26	4	15.4%
			Average for Sites:	**26.2%**
U-Turn (One Side Only)				
11	2010–2015	245	46	18.8%
12	2009–2015	241	45	18.7%
			Average for Sites:	**18.7%**
U-Turn (Both Sides)				
1b	2015	34	10	29.4%
6	2009–2015	78	19	24.4%
7b	2013–2015	134	24	17.9%
8	2009–2015	160	61	38.1%
9	2011–2015	187	55	29.4%
13	2009–2015	210	22	10.5%
14	2009–2015	360	61	16.9%
15	2009–2015	167	32	19.2%
16	2009–2015	326	76	23.3%
17	2009–2015	499	108	21.6%
18	2009–2015	200	53	26.5%
19	2009–2015	319	95	29.8%
20	2009–2015	809	136	16.8%
22	2009–2015	266	38	14.3%
23	2009–2015	637	121	19.0%
24	2009–2015	22	4	18.2%
25	2009–2015	144	27	18.8%
26	2009–2015	121	22	18.2%
			Average for Sites:	**21.8%**

Table 40. Percent of Left-Turn Crashes Initiating on Frontage Road Contrasted to Other Left Turns.

Site	Time Period	Number of Left-Turn Crashes	Left-Turn Crashes Initiating on Frontage Road		Left-Turn Crashes from Cross Street	
			Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
No U-Turn						
1a	2009–2012	49	2	4.1%	47	95.9%
2	2009–2015	47	20	42.6%	27	57.4%
3	2009–2015	66	40	60.6%	26	39.4%
4	2009–2015	22	10	45.5%	12	54.5%
5	2009–2012	13	3	23.1%	10	76.9%
7a	2009–2010	25	5	20.0%	20	80.0%
10	2009–2015	4	2	50.0%	2	50.0%
			Average for Sites:	**35.1%**		**64.9%**
U-Turn (One Side Only)						
11	2010–2015	46	16	34.8%	30	65.2%
12	2009–2015	45	20	44.4%	25	55.6%
			Average for Sites:	**39.6%**		**60.4%**
U-Turn (Both Sides)						
1b	2015	10	0	0.0%	10	100.0%
6	2009–2015	19	13	68.4%	6	31.6%
7b	2013–2015	24	10	41.7%	14	58.3%
8	2009–2015	61	4	6.6%	57	93.4%
9	2011–2015	55	3	5.5%	52	94.5%
13	2009–2015	22	9	40.9%	13	59.1%
14	2009–2015	61	45	73.8%	16	26.2%
15	2009–2015	32	26	81.3%	6	18.8%
16	2009–2015	76	72	94.7%	4	5.3%
17	2009–2015	108	84	77.8%	24	22.2%
18	2009–2015	53	0	0.0%	53	100.0%
19	2009–2015	95	46	48.4%	49	51.6%
20	2009–2015	136	60	44.1%	76	55.9%
22	2009–2015	38	17	44.7%	21	55.3%
23	2009–2015	121	56	46.3%	65	53.7%
24	2009–2015	4	4	100.0%	0	0.0%
25	2009–2015	27	6	22.2%	21	77.8%
26	2009–2015	22	7	31.8%	15	68.2%
			Average for Sites:	**46.0%**		**54.0%**

INFLUENTIAL VARAIBLES FOR FINAL MODELS

The construction of a dedicated U-turn lane can be expected to shift left-turn maneuvers from the cross street to the U-turn location. This change effectively removes two potential left-turn conflicts (one when the vehicle turns left onto the cross street and the second when the vehicle then turns left onto the opposing-direction FR). Though the U-turn configuration may introduce rear-end and merging conflicts, the removal of the left-turn conflicts can still be expected to contribute to a smaller number of severe crashes at these interchange locations. For that reason, researchers explored the interchange data to determine what variables appeared to be influential as they related to crash frequency and severity. The variable assessment process required several iterations prior to researchers isolating which variables belong in the models and what format is appropriate for each model. Appendix H reviews these additional model development steps. The following sections focus on the resulting final models for total and severe crashes.

Based on the premise that the number of left-turn crashes that originate on an FR can be expected to be reduced at U-turn locations, researchers inspected the crash data (for years 2009 to 2015) and site data using the proportion of fatal and injury (KAB) crashes where the vehicle originated on the FR. In some cases, missing data required slight modifications in sample sizes. This occurred when a data element could not be determined using the aerial photos and the RHiNo file. For the initial inspection, the data included 2019 site periods (seven potential years per site) from 164 sites; however, 77 site periods did not have any speed limit data and could not be evaluated, resulting in 1016 site periods from 152 sites with U-turns (see Table 41).

The presence of a U-turn may introduce an issue between vehicles exiting the U-turn and then shifting across all FR lanes to turn right into a driveway. This maneuver may conflict with vehicles turning left or right from the cross street onto the FR so that they can enter the highway. Consequently, the placement of the closest downstream driveway can be important to FR operations. This distance may also be linked to the posted speed limit (a closer driveway may suggest lower speed limits). As shown in Figure 45, this relationship shows that as the average posted speed limit increased, the distance to the closest driveway similarly increased. This observation indicates a strong correlation between these two characteristics, suggesting that inclusion of both variables in a statistical model, without accounting for their interaction, is likely to introduce a bias.

Table 41. Summary of Data Characteristics.

Variable Name	Mean	Std. Dev.	Min.	Max.	Total	N
Summary of Observed Crashes						
Total crashes	25.59	23.53	0	118	25,999	1016
Total crashes (known coordinates)	25.38	23.36	0	117	25,783	1016
KAB crashes	7.22	6.87	0	41	7336	1016
Crashes involving a left-turning vehicle from frontage road	3.72	5.24	0	34	3777	1016
Crashes involving a left-turning vehicle from frontage road (known coordinates)	3.69	5.23	0	34	3754	1016
KAB crashes involving a left-turning vehicle from frontage road	0.89	1.47	0	12	908	1016
KAB crashes involving a left-turning vehicle from frontage road (known coordinates)	0.89	1.46	0	11	903	1016
Additional Variable Characteristics						
Minimum posted speed limit (mph)	45.4	5.9	30	55	46,145	1016
Maximum posted speed limit (mph)	46	6	30	55	46,705	1016

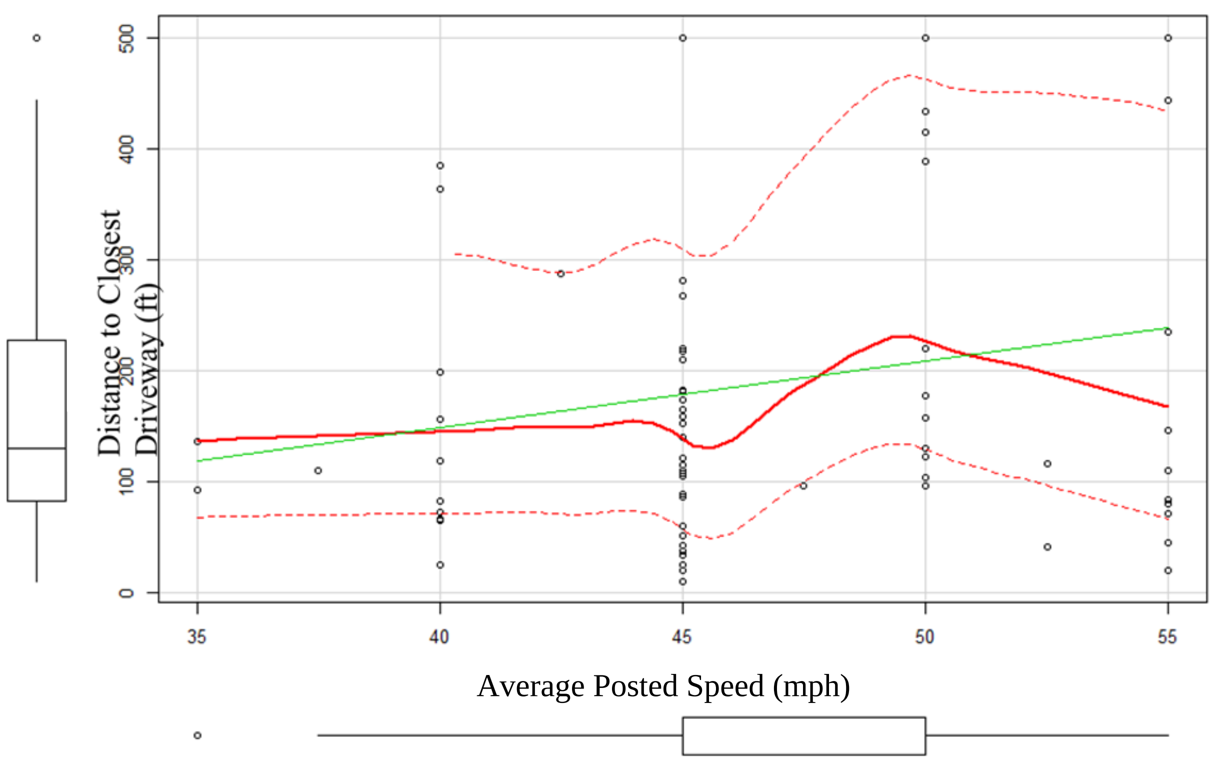


Figure 45. Relationship between Distance to Closest Driveway and Average Posted Speed.

OVERVIEW OF STATISTICAL ANALYSIS

Following the initial data inspection, researchers performed a stepwise regression analysis to assess the influence of significant variables on the total number of crashes and on crash severity for the dedicated U-turn lane locations. During this process, researchers noted that a small number of the interchanges had a posted speed limit of 30 mph and none of these intersections had traffic signals for both of the intersections associated with the interchange. To mitigate the influence of this subset of study sites, these intersections were removed as part of the stepwise regression analysis.

Model Development

Development of the crash model focused on the identification of statistically significant variables and an assessment of the best functional form that represents each identified variable. For example, the use of logarithmic adjustments may be appropriate for some variables that do not follow a linear format when graphically plotted. The following content reviews key issues considered during model development followed by a review of the resulting models for total crashes as well as KAB crashes.

Final Total Crash Model (No Yearly Factor)

The total crash model introduced in Appendix H incorporated a yearly factor to capture temporal effects related to the predicted number of crashes, but the use of this type of model can be limited, and it is not practical to apply it to future predicted crashes. Consequently, the researchers developed a simplified model that does not include the yearly factor. The goodness of fit for this model is quite similar to that noted for the model with the yearly factor, so the researchers recommend using this more flexible model. The resulting total crash model and associated descriptive statistic information for each continuous variable is included in Table 42.

Table 42. Simplified Predictive Model for Total Crashes (Signalized Sites No Yearly Factor).

Continuous Variable Descriptive Statistics							
Variable Name	**Description**	**Mean**	**Std. Dev.**	**Min.**	**Max.**	**Total**	**N**
AvgLn	Average number of frontage road lanes FR_A and FR_B (see Figure 36)	2.6	0.5	2	4	1195	459
DWY	Distance to Closest Downstream Driveway (ft)	196.3	155.8	10	500	90,110	459
CS_AADT	Cross-Street AADT (vpd)	13,516.8	10,059.6	200	54,609	6,204,220	459
Rmin	Minimum turning radius in U-turn (ft)	49.9	14.6	22	129	22,918	459

Total Crash Model					
Variables	**Estimate**	**Standard Error**	**Z Value**	**Pr(>|z|)**	**Significance[b]**
(Intercept)[a]	5.3041	1.0862	4.8834	1.0428×10^{-6}	***
RtA	-0.2708	0.1023	-2.6480	0.0081	**
AvgLn	0.7027	0.1616	4.3490	0.0000	***
scale(D_to_Closest_Driveway)	-0.2684	0.0719	-3.7320	0.0002	***
scale(CS_AADT)	0.1131	0.0489	2.3120	0.0208	*
ln(Rmin)	-0.9512	0.2454	-3.8760	0.0001	***

Where:

RtA = Number of instances at the site where RtA had a shared right-turn lane and no channelization island (see Figure 37). Value of RtA ranges from zero (no shared lane option) p to two (shared lane option at both cross-street right-turn locations.

Notes:

[a] Includes adjustment due to random effects.

[b] Significance levels are as follows:

* Statistically different from 0.0 at the 5% significance level.

** Statistically different from 0.0 at the 1% significance level.

*** Statistically different from 0.0 at the 0.1% significance level.

A common goodness of fit assessment is the cumulative residual (CURE) plot. Optimally, the CURE plot for each variable should oscillate around the line that represents zero. For the total crash model, these plots depict minimal deviations beyond the expected boundaries for key variables (see Figure 46).

The final model presents a functional form that incorporates scaling of some variables. This total crash model is represented by Equation 5-1.

Equation 5-1:

$$N_{Total} = e^{\left[5.304-(0.271\times RtA)+(0.703\times AvgLn)-\left(0.268\times\left[\frac{DWY-196.319}{155.752}\right]\right)+0.113\times\left[\frac{CS_{AADT}-13{,}516.82}{10{,}059.57}\right]-(0.951\times\ln(Rmin))\right]}$$

The equation included a scaling adjustment for the DWY and the CS_{AADT} variables. Researchers adjusted these scaled variables by subtracting the mean value and then dividing by the variable's standard deviation. The reduced N_{Total} model is shown in Equation 5-2.

Equation 5-2:

$$N_{Total} = \frac{e^{[5.491-(0.271\times RtA)+(0.703\times AvgLn)-(0.0017\times DWY)+(1.124\times 10^{-5})\times CS_{AADT}]}}{Rmin^{0.9512}}$$

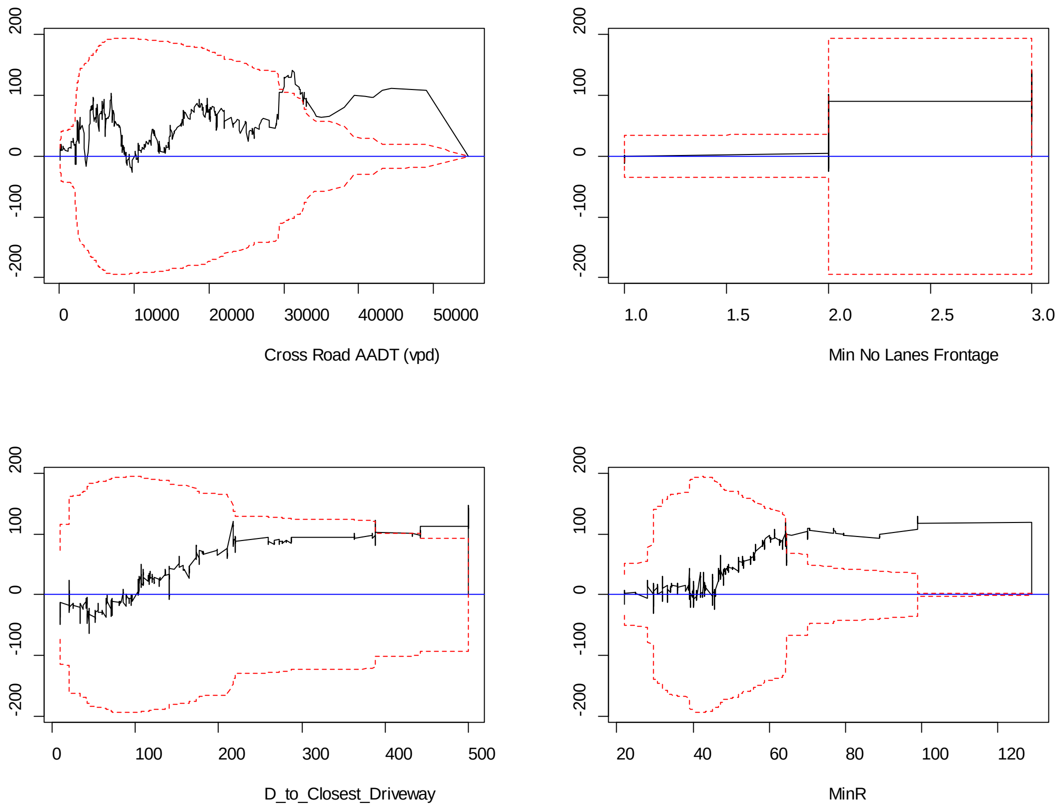


Figure 46. CURE Plots for the Total Crash Model.

One interesting observation about the final (and ultimately the KAB) model is that presence of a turnaround does not appear as a critical variable in the model. Researchers included this variable in the stepwise analysis, and it was not significant. This finding suggests that constructing a turnaround does not significantly affect the total number of non-freeway interchange crashes, so this treatment should complement operational benefits of adding turnarounds at diamond interchange locations.

By inspection of the variables included in the total crash model for interchanges with turnarounds, the following general observations merit consideration:

- Locations where the right turn from the cross street originates from a shared lane and does not have a large turning radius or a raised island (Option A) can result in 23.7 percent fewer crashes (calculated as $1 - e^{(-0.2708)} = 0.237$). (This finding is significant at 1 percent.)
- For FRs with two to four lanes, the number of crashes increases by a factor of 2.01 (doubles) for each additional FR lane (calculated as $e^{(0.7027)} = 2.014$). (This finding is significant at 0.1 percent). This finding is likely a surrogate for the varying FR AADT values.
- The number of crashes reduces as the distance to the closest downstream driveway increases. This reduction is approximately 1.7 percent for each additional 10 ft between the closest U-turn exit and the downstream driveway (calculated as $1 - e^{\frac{-0.2684}{155.752}\times 10} = 0.0171$). (This finding is significant at 0.1 percent).
- The number of crashes increases by 1.1 percent for each additional 1000 vpd increase in cross-street AADT (calculated as $1 - e^{\frac{0.1131}{10{,}059.6}\times 1000} = 0.011$). (This finding is significant at 5 percent).
- The number of crashes decreases by 8.7 percent for each increase of 10 percent in the turning radius of the U-turn (calculated as $1 - e^{-0.9512\times \ln(1.1)} = 0.0867$). (This finding is significant at 0.1 percent).

KAB Frequency Model (No Yearly Factor)

Proceeding similarly to the development of the final predictive model for total crashes, researchers focused on signalized intersection locations with speed limits greater than 30 mph to develop a predictive model for KAB crashes through the use of stepwise regression procedures. Table 43 depicts this resulting model.

Table 43. Predictive Models for KAB Crashes (Signalized Intersections).

Continuous Variable Descriptive Statistics for the KAB Model							
Variable Name	**Description**	**Mean**	**Std. Dev.**	**Min.**	**Max.**	**Total**	**N**
CS_AADT	Cross-Street AADT (vpd)	13,872.4	10,442.3	200	54,309	6,478,395	467
AvgLn	Average number of frontage road lanes FR_A and FR_B (see Figure 36)	2.62	0.45	2	4	1225	467
DWY	Distance to Closest Downstream Driveway (ft)	191.54	151.86	10	500	89,467	467
Rmin	Minimum turning radius in U-turn (ft)	49.99	14.84	22	129	23,344	467

Final KAB Model					
Variables	**Estimate**	**Standard Error**	**Z Value**	**Pr(>|z|)**	**Significance[b]**
(Intercept)[a]	3.0758	1.1096	2.7720	5.5718E-03	**
ln(CS_AADT)	0.0719	0.0392	1.8360	6.6290E-02	+
RtA	-0.3063	0.1013	-3.0220	2.5100E-03	**
AvgLn	0.4797	0.1580	3.0350	2.4000E-03	**
$Merge_{RT}$	-0.1801	0.0892	-2.0200	4.3350E-02	*
scale(DWY)	-0.1969	0.0699	-2.8190	4.8200E-03	**
ln(Rmin)	-0.6684	0.2412	-2.7710	5.6000E-03	**

Where:

RtA = Number of instances at the site where right-turn zone entrance treatment had a shared right-turn lane and no channelization island (see Figure 37). Value of RtA ranges from zero (no shared lane option) p to two (shared lane option at both cross-street right-turn locations.

$Merge_{RT}$ = Number of instances at the site where the right-turn zone exit treatment merged into an existing lane (see Figure 38). Value of $Merge_{RT}$ ranges from zero (only included added lanes) to two (all cross-street right-turn lanes require vehicles to merge into an existing lane.

Notes:

[a] Includes adjustment due to random effects.
[b] Significance levels are as follows:
+ Statistically different from 0.0 at the 10% significance level.
* Statistically different from 0.0 at the 5% significance level.
** Statistically different from 0.0 at the 1% significance level.
*** Statistically different from 0.0 at the 0.1% significance level.

CURE plots for each variable depict minimal deviations beyond the expected boundaries for key variables. An examination of the model residuals did not show any evidence of overdispersion, and the CURE plots did not show concerns about any variable, except perhaps a slight underprediction at the higher end of the minimum U-turn radius, as shown in Figure 47.

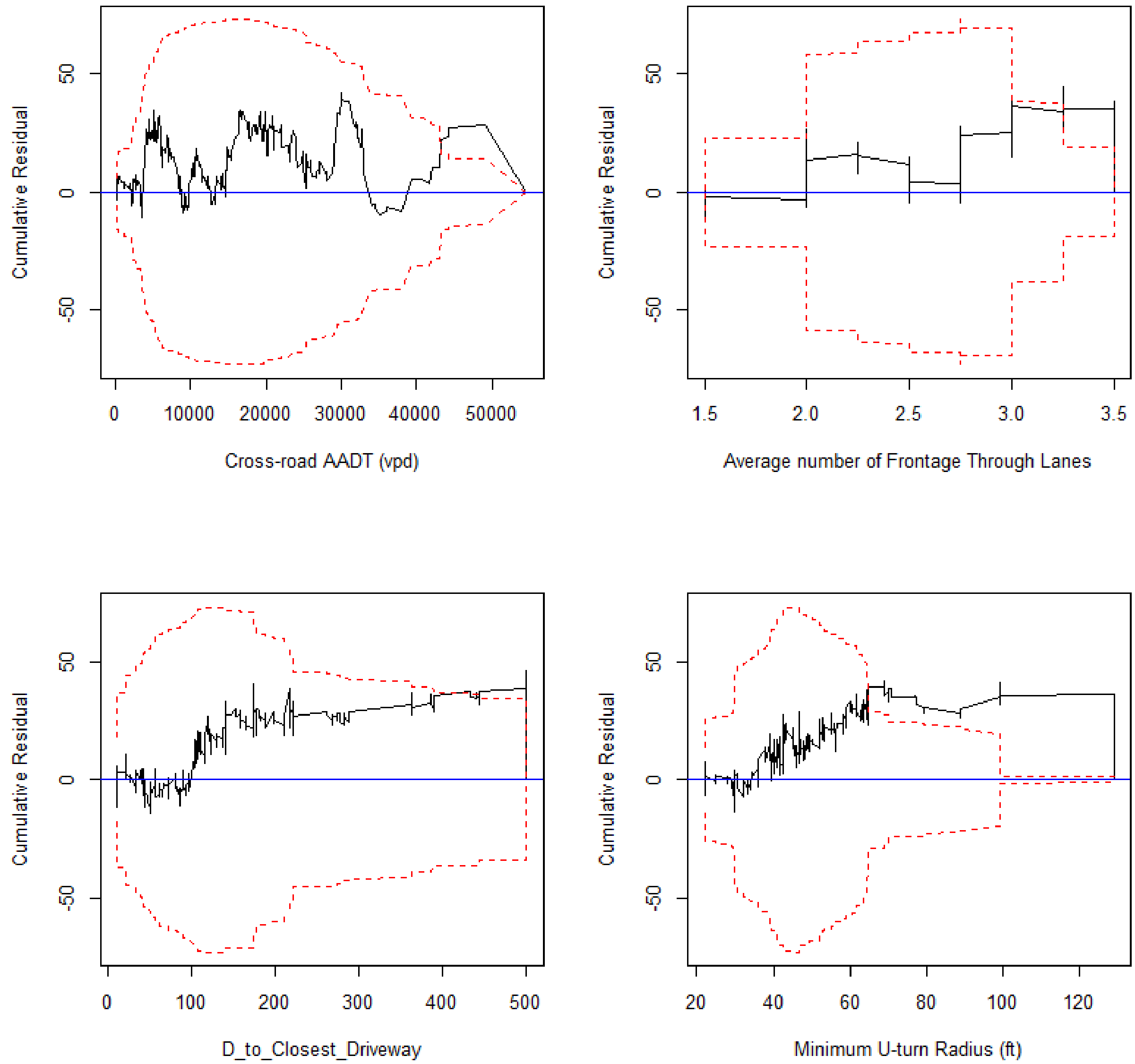


Figure 47. CURE Plots for KAB Crashes Predictive Model.

As shown in Figure 48, the site-specific KAB models have a narrower threshold (predict more precisely) than the models that fit the general population; however, the population models provide a greater amount of flexibility in future model applications toward the larger diamond interchange population.

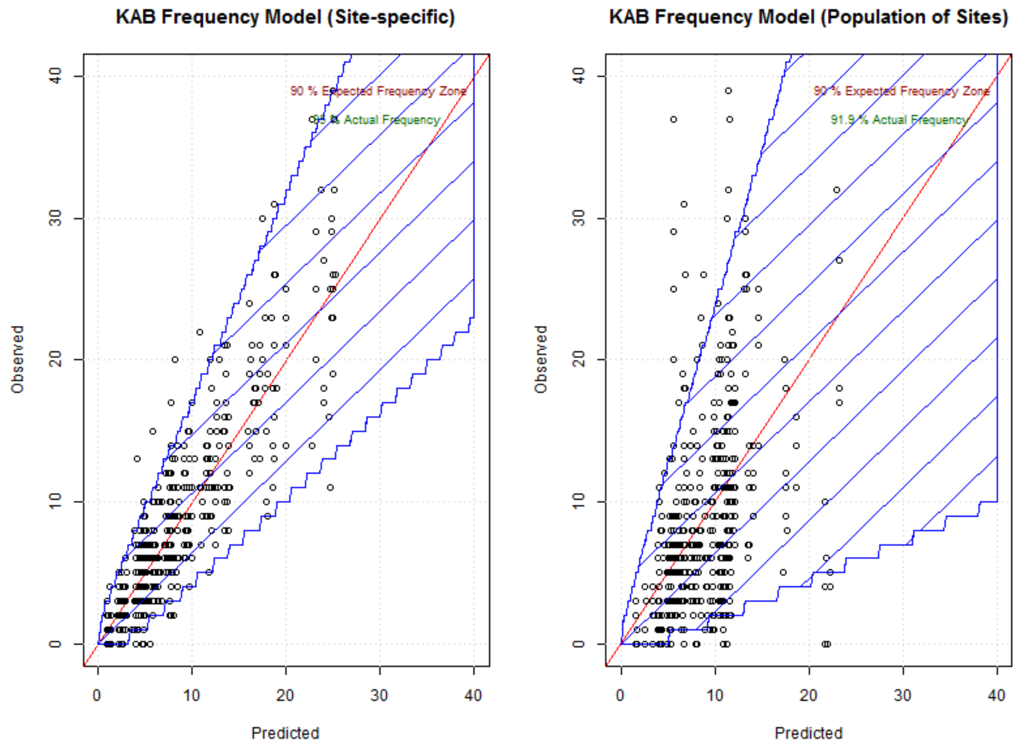


Figure 48. Model Fit for KAB Crashes Predictive Model.

KAB Frequency Model—Original Model Prior to Reduction

The format of the final KAB frequency model can be written as shown in Equation 5-3.

Equation 5-3:

$$N_{KAB} = e^{[3.08-(0.31\times RtA)+(0.48\times AvgLn)-(0.18\times LnMerge)-\left(0.20\times\left[\frac{DwyDist-191.58}{151.86}\right]\right)+(0.072\times\ln(CR\_AADT))-(0.67\times\ln(MinR))]}$$

Note that the DWY variable is scaled, where 191.58 represents the mean, and 151.86 represents the standard deviation for the DWY variable.

Reducing Individual Model Elements

The individual components for the distance to nearest driveway, cross-street AADT, and minimum U-turn radius of the model can be reduced further, as shown in the following sections.

Distance to Nearest Driveway

Based on the variable for the distance to the nearest driveway, this portion of Equation 5-3 can be reduced as follows:

$$= e^{[-\left(0.20\times\left[\frac{DwyDist-191.58}{151.86}\right]\right)]}$$

$$= e^{\left[-\left(0.20\times\left\lfloor\frac{DwyDist}{151.86}\right\rfloor\right)-\left(0.20\times\left\lfloor\frac{-191.58}{151.86}\right\rfloor\right)\right]}$$

$$= e^{[\,-(0.00126\times DwyDist)+0.242]}$$

Cross-Street AADT

Based on the variable that represents the cross-street AADT, this portion of Equation 5-3 can be reduced in the following manner:

$$= e^{[(0.072\times \ln(CR\_AADT))]}$$

$$= CR\_AADT^{0.072}$$

Minimum U-Turn Radius

Based on the variable that represents the minimum U-turn radius, this portion of Equation 5-3 can be reduced in the following manner:

$$= e^{[-0.67\times \ln(\,MinR)]}$$

$$= MinR^{-0.67} = \frac{1}{MinR^{0.67}}$$

Final Reduced Model

The individual model elements can then be incorporated into the final model, as shown in Equation 5-4.

Equation 5-4:

$$N_{KAB} = \frac{CR\_AADT^{0.0719}}{MinR^{0.6684}} \times e^{[(3.08+0.242)-(0.31\times RtA)+(0.48\times AvgLn)-(0.18\times LnMerge)-(0.0013\times DwyDist)]}$$

Equation 5-4 can then finally be reduced as depicted in Equation 5-5.

Equation 5-5:

$$N_{KAB} = \frac{CR\_AADT^{0.0719}}{MinR^{0.6684}} \times e^{[3.32-(0.31\times RtA)+(0.48\times AvgLn)-(0.18\times LnMerge)-(0.0013\times DwyDist)]}$$

By inspection of the variables included in the KAB model for signalized interchanges with turnarounds, the following observations merit consideration:

Sites where the right turn from the cross street must share a lane have 26.4 percent fewer KAB crashes (calculated as $1 - e^{(-0.3063)}$). (This finding is significant at 1 percent.)

Sites where the right-turn traffic must merge with the FR traffic (without adding a lane) have fewer severe crashes (significant at < 0.01 percent). This merge configuration is associated with a reduction of 16.4 percent (calculated as $1 - e^{(-0.1801)}$). (This finding is significant at 1 percent)

The number of KAB crashes is smaller by 1.3 percent for each additional 10 ft between the closest downstream driveway and the U-turn exit (calculated as $1 - e^{\frac{-0.1969}{151.86} \times 10} = 0.0126$). (This finding is significant at 1 percent.)

The number of KAB crashes is smaller by 6.2 percent for each increase of 10 percent in the turning radius of the U-turn (calculated as $1 - e^{-0.6684 \times \ln(1.1)} = 0.62$). (This finding is significant at 1 percent.)

CONCLUSIONS

This chapter examined the safety effects of interchanges with dedicated U-turn lanes. Based on both a qualitative and quantitative assessment of crash severity and frequency, researchers generally concluded that the addition of dedicated U-turn lanes at diamond interchange locations will result in fewer severe crashes, though this trend was not determined to be statistically significant.

Based on a statistical evaluation of total crashes and injury crashes, researchers concluded that key variables that significantly influence the number of crashes at a turnaround location include cross-street AADT, cross-street right-turn configuration, number of FR lanes, longitudinal distance from U-turn exit to nearest downstream driveway, and U-turn minimum radius values. Researchers limited the study sites to locations with posted speed limits on the FR of 35 to 55 mph and with signalized intersection configurations. Table 44 summarizes how each of these site or traffic characteristics influences the number of predicted crashes.

Table 44. Influence of Site or Traffic Characteristics on Crashes.

Site or Traffic Characteristic	Significant Influence	
	Total Crashes	**KAB Crashes**
As the cross-street AADT increases:	Total crashes increase	KAB crashes increase
Cross-street right-turn maneuvers onto the frontage road that originate in a shared lane result in:	Fewer total crashes	Fewer KAB crashes
Cross-street right-turning vehicles that merge into existing lanes result in:	No significantly noticeable change in total crashes	Fewer KAB crashes
As the number of lanes increase for each frontage road approach:	Total crashes increase	KAB crashes increase
As the longitudinal distance between the U-turn exit to the nearest downstream driveway increases:	Total crashes decrease	KAB crashes decrease
As the minimum U-turn radius increases:	Total crashes decrease	KAB crashes decrease

In addition, researchers evaluated sites with and without turnarounds. During the statistical analysis, it became clear that the variable that indicated the presence of a turnaround was not statistically significant. This finding suggests that the construction of turnarounds at diamond interchanges will not substantially affect the total number of crashes at these locations. Thus, the construction of a turnaround as a mechanism for improving operations and removing the two left turns from adjacent signalized intersections will not have any significant adverse safety implications and should complement operational improvements.

CHAPTER 6. DEVELOPMENT OF U-TURN GUIDELINES

INTRODUCTION

The work described in the previous chapters was intended for the development of guidelines for the planning, design, and operation of U-turn lanes. In this chapter, results from field-based U-turn observations, myriad simulations of site improvements with the potential to improve U-turn operations, and a full safety investigation of factors contributing to crashes at interchanges were combined to provide utilitarian guidance regarding U-turn planning, design, and operation.

Researchers performed extensive field investigations, simulation investigations, and statistical safety analyses of the factors contributing to and affecting U-turn design and operations at diamond interchanges in Texas. Findings from these research activities were combined and integrated to develop guidelines for U-turn planning, design, and operations.

GUIDELINES FOR U-TURNS

The guidelines were developed for the purpose of assisting TxDOT staff in the planning, design, and operation of U-turn lanes. Guidelines specific to each of these three categories are outlined below.

U-Turn Planning

- U-turn lanes should be considered for future interchanges with a projected (20-year) peak-hour volume of at least 2000 vph, or roughly 20,000 ADT. For existing interchanges, U-turn lane implementation should be considered when total interchange traffic volume reaches 4000 vph, or approximately 40,000 ADT. Field investigations conducted by researchers revealed that U-turn lanes justify themselves on a delay savings basis at relatively low interchange volume levels. Simulation studies affirmed that these findings demonstrate very large delay reductions for U-turn movements in cases where U-turn lanes were added.

U-Turn Design

- U-turn design should include an approach bay with a minimum length of 525 ft. In rural areas, this length primarily provides stopping sight distance on the U-turn approach for higher-speed operations. In urban areas, the bay length requirement is designed to allow U-turning vehicles to avoid interference from left-turn queues in the adjacent lane.
- Operations are improved if the U-turn lane departure features either a full added lane or an acceleration lane (minimum 100-ft length) with taper. U-turn departures featuring stop or yield control, or those that terminate with only a taper transition into

a FR lane, should only be used where geometric constraints or low-volume conditions exist. Providing an acceleration lane or full lane for the U-turn departure allows merges at higher speeds with smaller critical gaps and can increase U-turn capacity as well as decrease U-turn queues. Simulation study results revealed that acceleration lanes longer than 100 ft did not appear to reduce delay any more than lanes 100 ft in length, though additional acceleration lane distance will support and stabilize merging operations under higher volume conditions.

- Ensure U-turn lane design provides sufficient turn radii. Input from TxDOT planning and operation personnel identified concerns with outdated U-turn designs where turn radii were not adequate to efficiently process heavy vehicles. Further, the safety analysis conducted in this research effort found conclusively that as the minimum radius for the U-turn increases, the number of crashes decreases (for radii between 22 ft and 130 ft).

U-Turn Operation

- Consider closing driveways within 250 ft of the U-turn lane itself to prevent U-turn vehicles from weaving into those driveways. Observations from field operations and the findings from simulation analyses show that U-turn traffic traveling/weaving across the FR to the driveway immediately downstream from the U-turn lane causes increased turbulence in the FR traffic stream and causes delay to following vehicles in the U-turn lane. The simulation study also revealed that at some sites with medium to high U-turn volume and high demand for development access, closing the first driveway reduced queues in the U-turn lane. Safety investigations have shown that crashes decrease as the distance to the closest (accessible) driveway increases.
- Access controls (pavement markings, flexible pylons, and/or curbs) can be used to improve U-turn departure operations. As with many of the U-turn improvement methods described in this research project, both field observation and simulation studies verified the beneficial impacts on U-turn operations that result when constraints are placed on weaving maneuvers from U-turn departures to adjacent downstream driveways along the FR. If closing those adjacent driveways is not feasible, traffic control devices such as double white lines, flexible pylons, and semi-permanent or permanent curbs help realize the intended access control purpose for departure-side U-turn acceleration lanes. Simulation results showed significant improvement for U-turns by restricting access to nearby driveways, and the safety investigation reinforced this finding by concluding that the number of crashes decreases as the distance to the closest downstream driveway increases.
- Consider right-turn accommodations at the interchange and their impacts on operations and safety. Safety improvements were observed when right turns both turned from and into shared lanes (i.e., no right-turn bays or right-turn acceleration lanes were present). Evaluation of field data revealed that right-turn volume is not

clearly linked to increases in U-turn delay. Simulation analyses further estimated the benefits to U-turn traffic if restrictions on right turns (such as preventing right turns on red) occurred. Limited to no U-turn benefit resulted from this experimentation, while overall interchange delay increased, especially at higher interchange volumes. Though U-turns may gain additional flow during the cross-street red, the increased and concentrated right-turn flow during the cross-street green may increase U-turn delay. While no U-turning benefits could be clearly defined for right-turn restrictions, the safety analysis revealed fewer crashes when right turns originated from shared lanes and turned into shared lanes.

- Signal timing can be used as an interchange management tool to support U-turn operation. Simulation experimentation based on field sites examined in this research effort targeted signal timing adjustments as a means of facilitating U-turn movements through the interchange. Both cycle length and split adjustments successfully demonstrated a reduction in both FR queue length and average delay on FR approaches. As U-turns approach the interchange along the FR (along with all other frontage movements), shortened queue lengths reduced the likelihood of a left-turn queue blocking access to a U-turn lane. As observed in both field studies and simulation exercises, high-volume interchanges where queues blocked access to the U-turn bay resulted in the highest observed U-turn movement delays in the research study.
- Altering cat tracks can improve U-turn operations. Cat tracks, or dotted line markings to extend lane lines into the intersection and guide drivers through the appropriate turning path, resulted in U-turn movement delay reduction benefits under medium- to high-volume interchange operations. Interchange arterial left turns are typically directed into the leftmost receiving lane on the FR, but this lane is also the lane that receives U-turn traffic (when an added lane or acceleration lane is not provided to receive the U-turn). Directing internal left-turn vehicles to alternative receiving lanes—the middle and/or right FR lanes—results in reduced U-turn delay and has only a minor impact on overall interchange operation and delay. In essence, for interchanges without U-turn departure side acceleration lanes, alterations in left-turning paths can provide longer gaps in the left FR lane stream for U-turn traffic.

RECOMMENDED REVISIONS TO TXDOT *ROADWAY DESIGN MANUAL*

The TxDOT RDM provides the current description of and guidance for intersections and turnarounds on freeway FRs; specifically, the last subsection of Chapter 3, Section 6 contains the guidelines on the use of turnaround, or U-turn, lanes. To provide additional guidance to designers, researchers recommend that the following text, based on research from Project 0-6894, be added to the current (October 2014) version of the RDM after Figures 3–38 on pages 3–96 of the PDF version of the manual and at the corresponding location in the online HTML version:

"Results from field-based observations, myriad simulations of site improvements with the potential to improve U-turn operations, and a full safety investigation of factors contributing to crashes at interchanges have produced the following guidance on the planning, design, and operation of turnaround lanes:

- Turnaround lanes should be considered for future interchanges with a projected (20-year) peak-hour volume of at least 2,000 vph, or roughly 20,000 ADT. For existing interchanges, turnaround lane implementation should be considered when total interchange traffic volume reaches 4,000 vph, or approximately 40,000 ADT.
- Turnaround lane design should include an approach bay with a minimum length of 525 ft.
- Operations are improved if the turnaround lane departure features either a full added lane or an acceleration lane (minimum 100-ft length) with taper. Turnaround lane departures featuring stop or yield control, or those that terminate with only a taper transition into an FR lane, should only be used where geometric constraints or low-volume conditions exist.
- Turnaround lane design should provide sufficient turn radii to accommodate heavy vehicles.
- To minimize delay and queuing in the turnaround lane and to minimize the potential for crashes on the FR, consider closing driveways within 250 ft of the U-turn lane itself to prevent U-turn vehicles from weaving into those driveways.
- If closing adjacent driveways is not feasible, consider the use of traffic control devices and/or channelization (e.g., pavement markings, flexible pylons, and/or raised curbs) to improve turnaround lane departure operations. Field observation and simulation studies have verified the benefits (e.g., reduced delay and fewer crashes) of constraining weaving maneuvers from turnaround lanes to adjacent downstream driveways.
- Consider right-turn accommodations at the interchange and their impacts on operations and safety. Safety improvements have been observed when right turns both turned from and into shared lanes (i.e., no right-turn bays or right-turn acceleration lanes were present).
- Signal timing can be used as an interchange management tool to support U-turn operation. Both cycle length and split adjustments have been successfully demonstrated to reduce FR queue length and average delay on FR approaches. Shorter queue lengths reduce the likelihood of a left-turn queue blocking access to a turnaround lane.
- Consider the use of dotted line markings to improve operations. Dotted lines to extend lane lines into the intersection and guide drivers through the appropriate turning path have shown reduced delay for turnaround lane movements under medium- to high-volume interchange operations. Directing internal left-turn vehicles

to the middle and/or right FR lanes provides gaps in the left FR for vehicles using the turnaround lane."

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APPENDIX A. QUESTIONS DOCUMENT FOR STATE-OF-THE-PRACTICE REVIEW

BACKGROUND

In Task 2 of Project 0-6894, researchers collected information about TxDOT district practices related to the planning, design, and operation of U-turn lanes at diamond interchanges (i.e., turnaround lanes). To facilitate the information-gathering process, researchers developed a list of questions to ask each respondent. The questions document included a list of related factors possibly affecting demand and capacity and potential solutions for improving efficiency; these factors were identified by researchers based on their expertise in the subject area and literature review. Next, the researchers contacted staff in the TxDOT districts via telephone and email to solicit responses to the questions in the document. Researchers also asked TxDOT staff to review the list of related factors. In many cases, researchers emailed this document to the identified staff in each district and followed up with a telephone call. Collectively, these selected TxDOT staff members had familiarity/expertise in planning, design, operations, or a combination of these areas. Researchers received responses over the phone and/or in a written form using a copy of the above-mentioned document sent to them via email.

The questions document is reproduced in this appendix. Discussion of the findings from the information-gathering process can be found in Chapter 2 of this report.

DOCUMENT USED TO GUIDE TXDOT STATE-OF-THE-PRACTICE INFORMATION GATHERING

TxDOT Project 0-6894: Guidelines for Design and Operations of U-Turns

Introduction

This questionnaire has been prepared to solicit TxDOT district feedback for the above-referenced project. The scope of this project is limited to diamond interchanges, and includes:

- Assessment of TxDOT practice related to the planning, design, and operations of U-turn lanes.
- Identification and evaluation of factors affecting the use of U-turns.
- Field evaluation of a sufficient number of sites to include:
    - Geographic diversity across the state.
    - Diverse of geometric designs.
    - Diverse operational conditions and challenges.
- Field (after) studies at a few of the above locations to evaluate the application of strategies developed in the project.

Questions for TxDOT Districts

1. In your district, what percent of U-turns are?
    a. Urban:
    b. Rural:
2. What documents, guidelines, criteria, or practices are used by district staff in the planning, design and operations of U-turns?
3. Any locations where U-turns were added/constructed in recent years? If yes, please provide examples of where and why (i.e., to solve operational or safety problems).
4. Any locations where U-turns were redesigned or retrofitted to improve:
    a. Operations?
    b. Safety?
5. Any locations in the district currently experiencing:
    a. Recurring operational issues (congestion, queuing, etc.)?
    b. Temporary operational issues (i.e., due to construction, detours, or other factors)?
    c. Safety problems?
6. If suitable sites exist, would you be willing to allow researchers to conduct field studies? As appropriate, please provide the following information for each location:
    a. Location?
    b. Geometric characteristics of the interchange (interior distance, number of lanes and widths, bay lengths, U-turn lane, types of ramps, etc.)?
    c. Any problems or issues?
    d. Existing traffic control (3-phase/4-phase, U-turns yield at exit, RTOR allowed, etc.)
    e. Cabinet type?
    f. Controller brand and firmware (NTCIP compatible)?
    g. Detection type and detector design?
    h. Any field-to-field or center-to-field communications infrastructure and how it is used?
    i. Do you have any existing volume/classification data that can be made available to researchers?
    j. If there is room in the cabinet, would you be willing to allow researchers to place video or other data collection equipment in the cabinet?

Additional Information Requested

<u>Factors Possibly Affecting Demand</u>

- Lane use/assignment.
- Nearby development intensity.
- Proximity and number of nearby driveways.
- Ramp configuration (Diamond or “X”) and interchange spacing.

Factors Possibly Affecting Capacity

- Traffic volumes and patterns.
- Interchange geometrics.
- Traffic control (typically yield).
- Right-turn demand from the cross street.
- Driveway access near the interchange.

Potential Solutions and Techniques for Improving U-turn Efficiency

- Modifications to signal timing plans to reduce queue length and facilitate access to lanes or bays at the start of each U-turn.
- Modifications to signal timing plans to facilitate access to FR lanes at the end of each U-turn and/or signalized control of the U-turn approach.
- RTOR restrictions on cross street to reduce the conflicts between U-turning and right-turning traffic.
- U-turn bay extensions or added lane(s) to facilitate entry to the U-turn lane.
- Two-lane U-turn lanes for added capacity to serve unusually high traffic demand.
- Access controls and/or driveway closure proximate to the interchange U-turn lane.
- Access controls for either the U-turn lane or the right-turn lane from the arterial to remove the conflict between these two movements.

APPENDIX B. VOLUME DATA FROM STUDY SITES

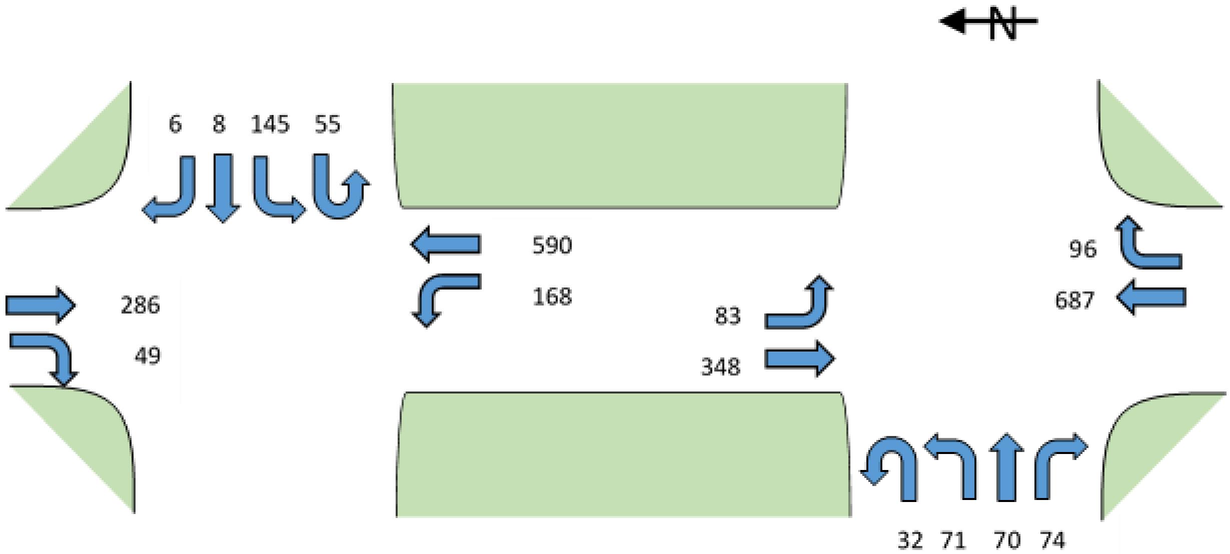

Figure 49. Abilene District—I-20 @ SH 351 (AM Peak Hour).

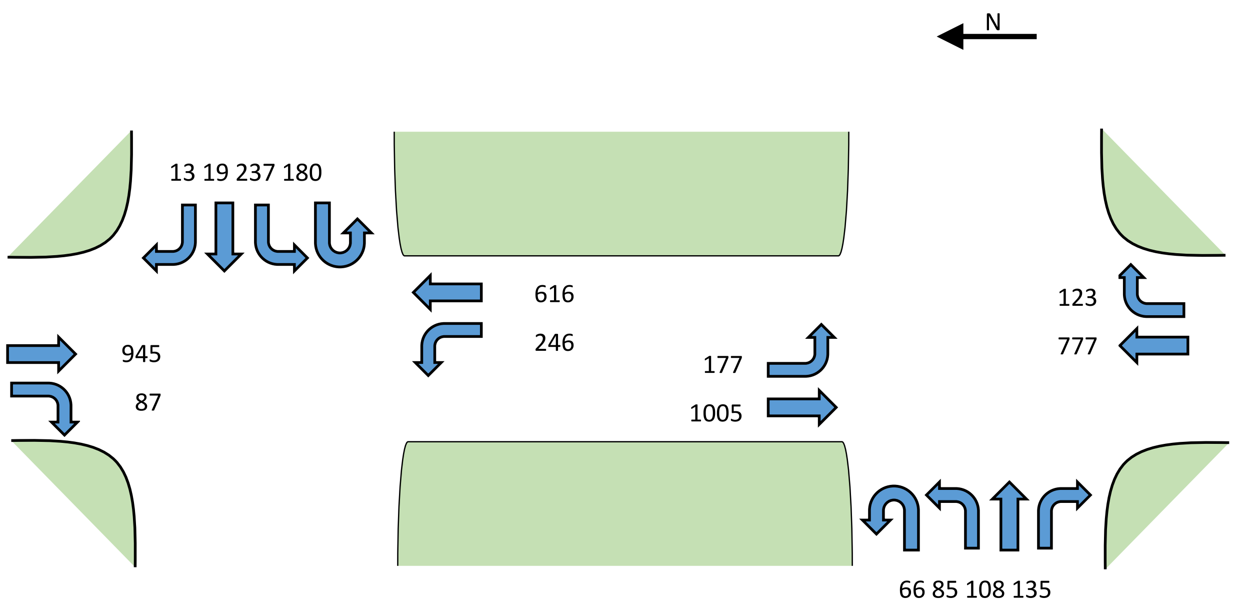

Figure 50. Abilene District—I-20 @ SH 351 (PM Peak Hour).

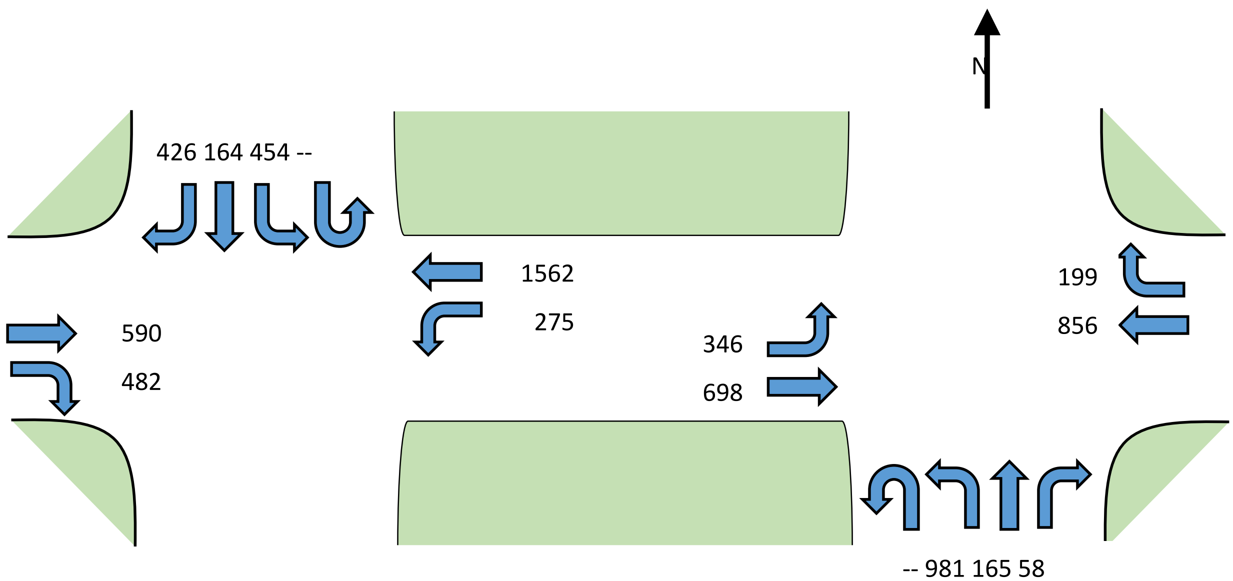

Figure 51. Bryan District—SH 6 @ Boonville (AM Peak Hour).

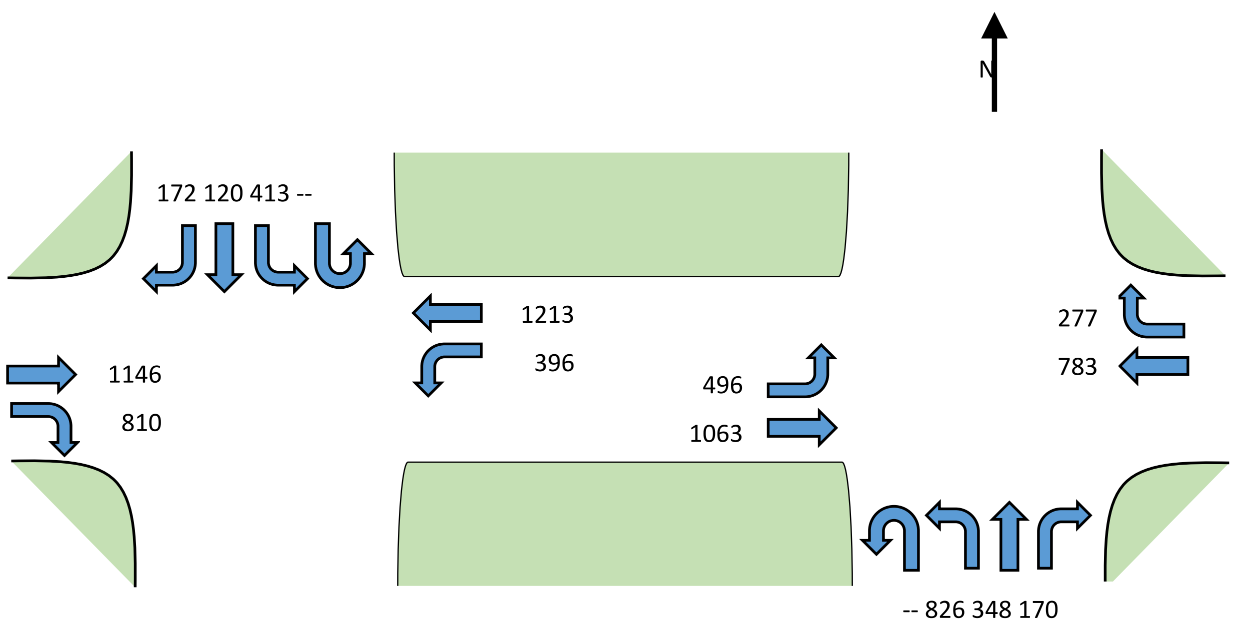

Figure 52. Bryan District—SH 6 @ Boonville (PM Peak Hour).

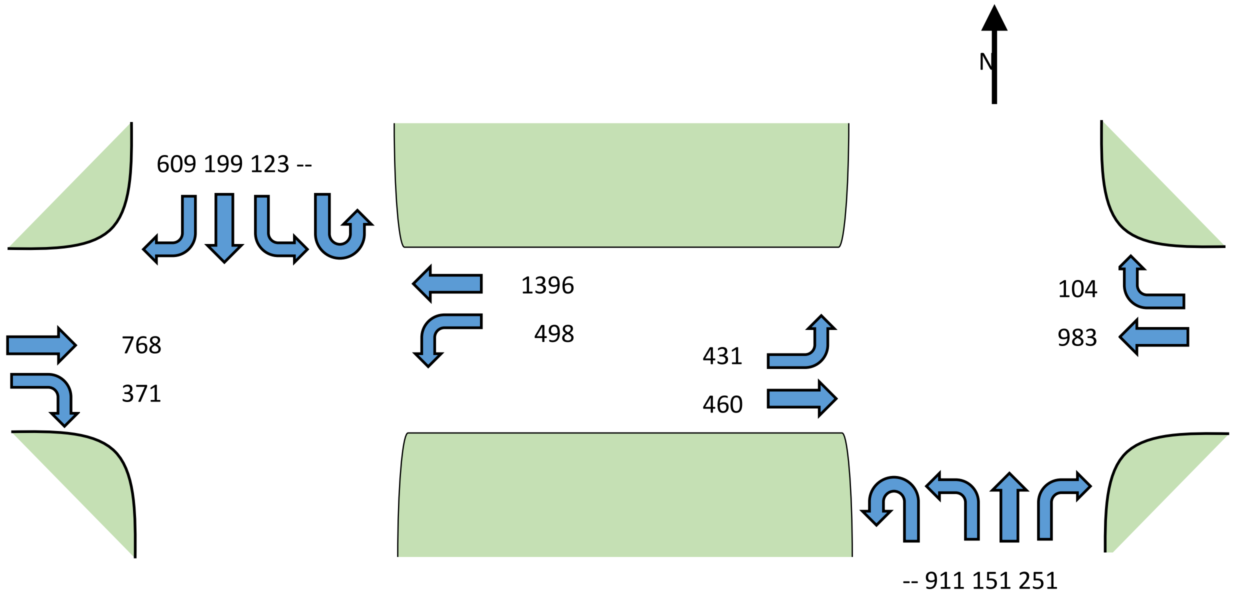

Figure 53. Bryan District—SH 6 @ Briarcrest (AM Peak Hour).

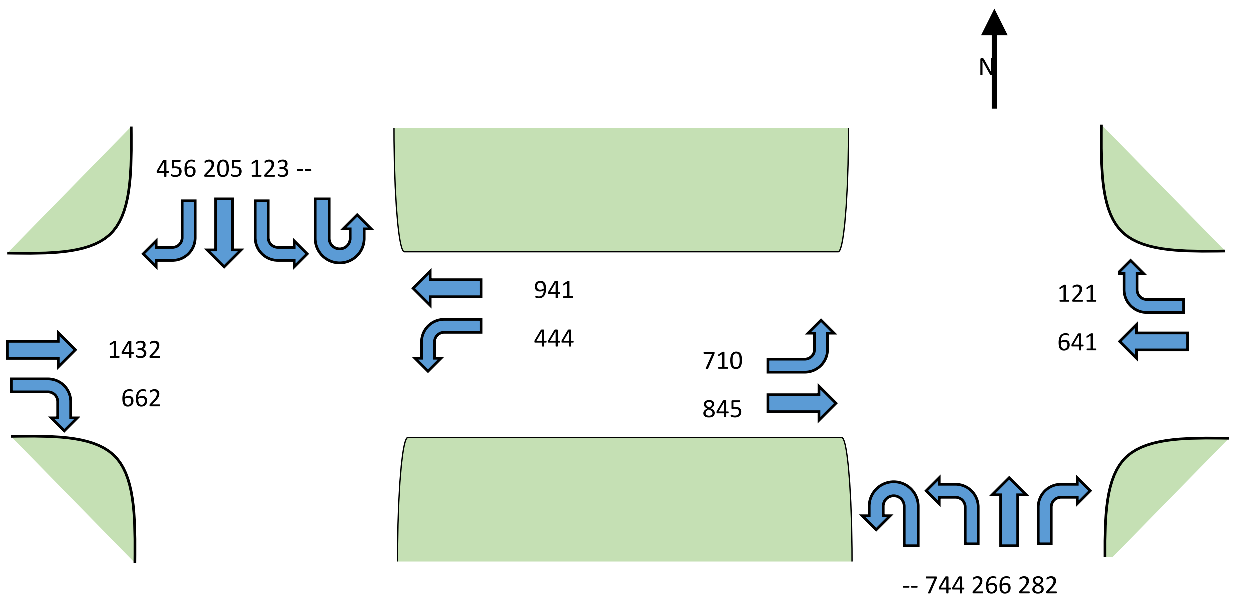

Figure 54. Bryan District—SH 6 @ Briarcrest (PM Peak Hour).

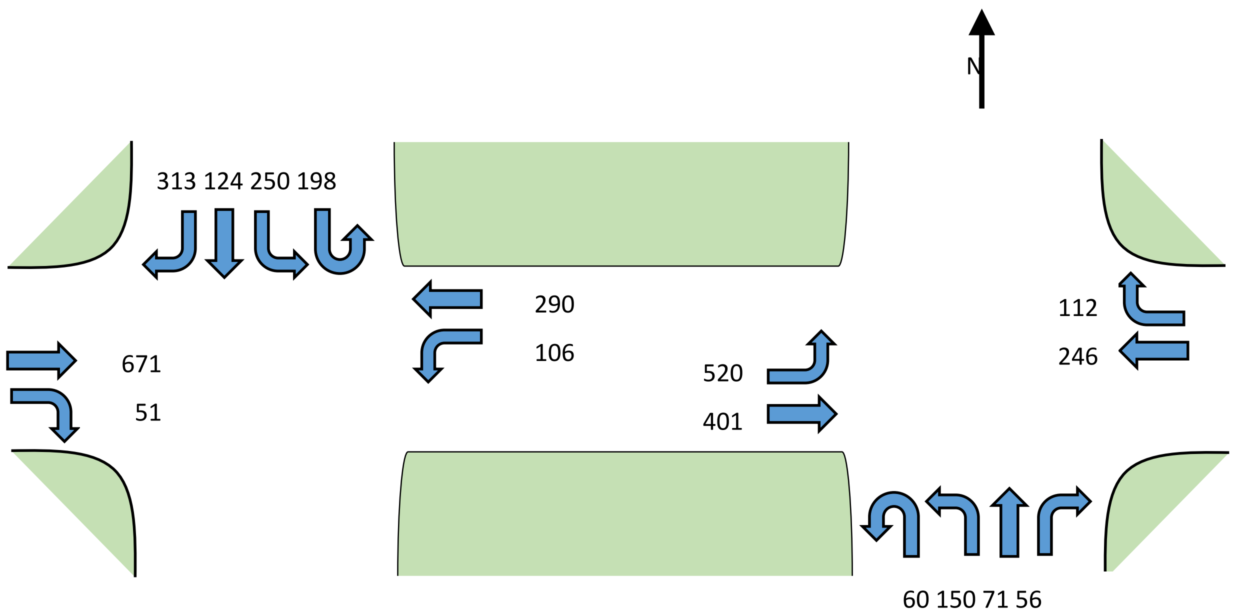

Figure 55. Bryan District—SH 6 @ Rock Prairie (AM Peak Hour).

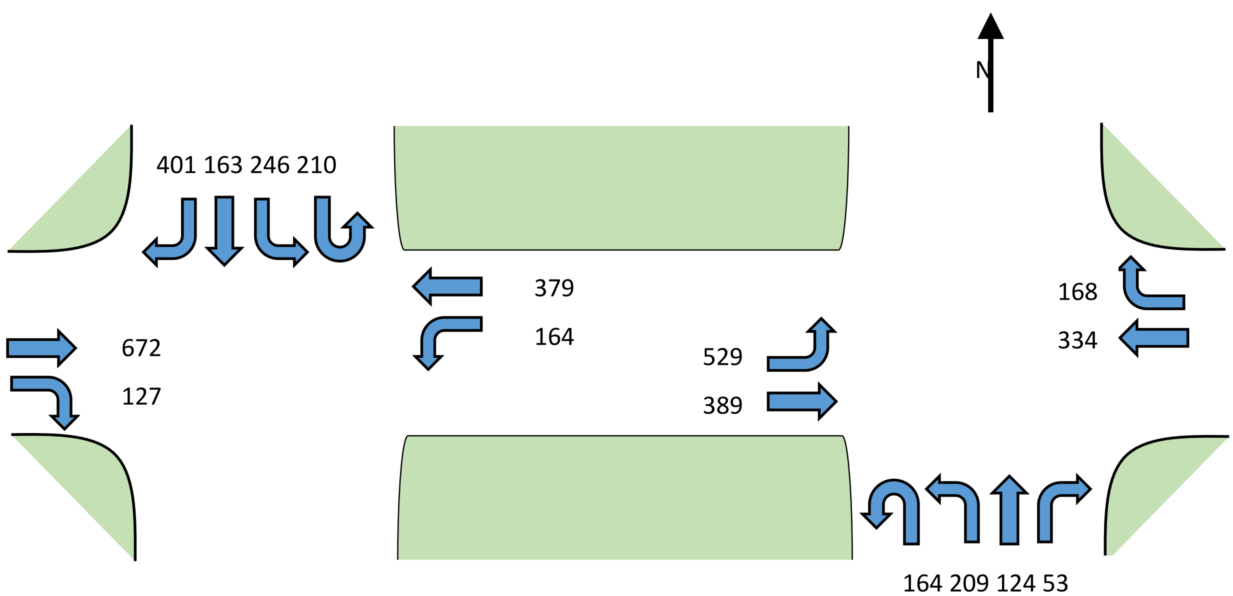

Figure 56. Bryan District—SH 6 @ Rock Prairie (PM Peak Hour).

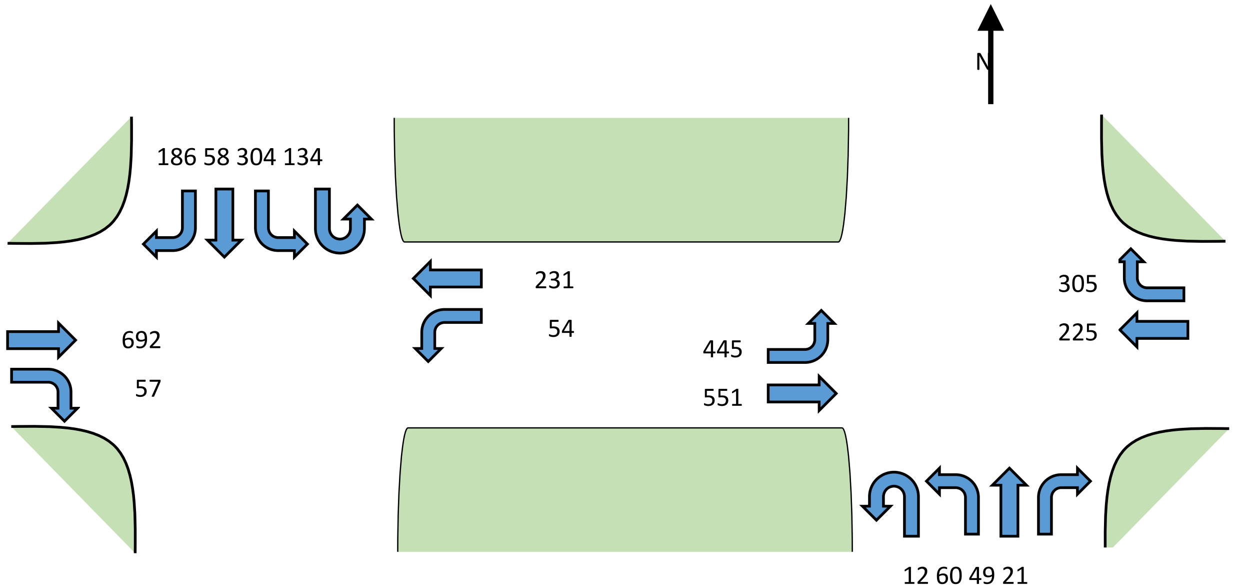

Figure 57. Bryan District—SH 6 @ SH 40 (AM Peak Hour).

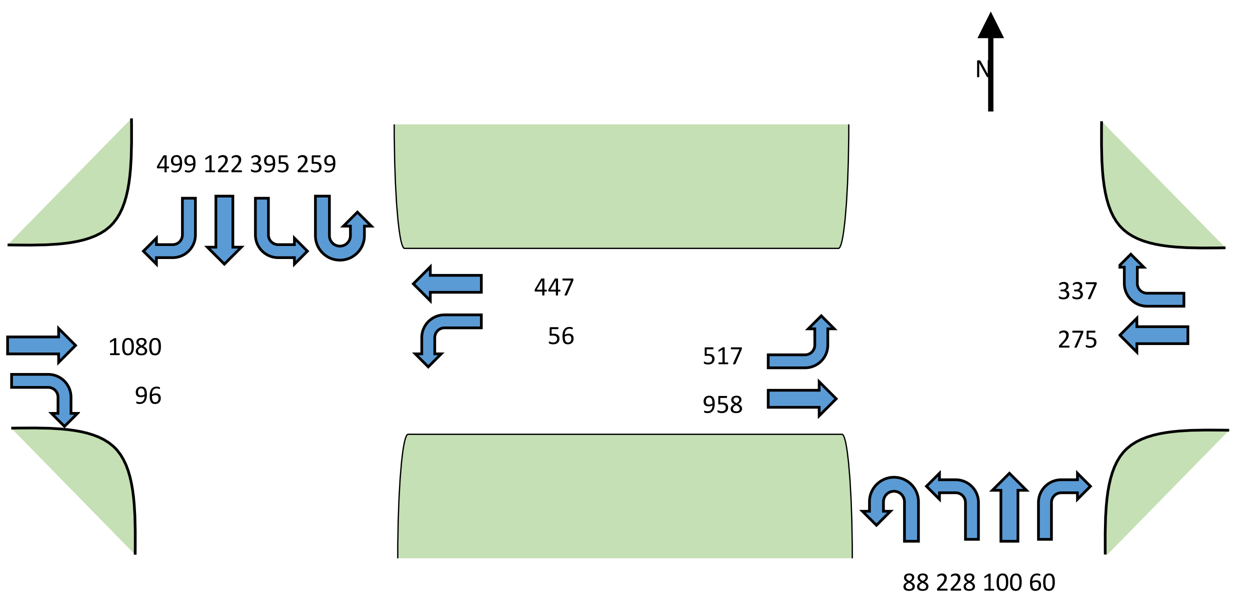

Figure 58. Bryan District—SH 6 @ SH 40 (PM Peak Hour).

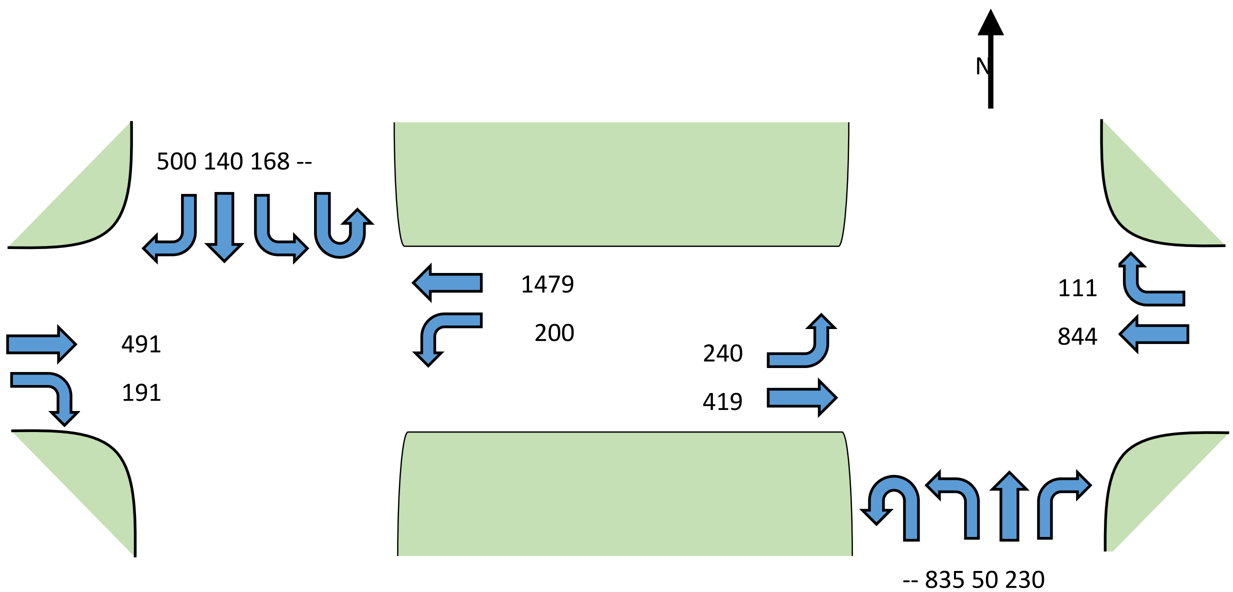

Figure 59. Bryan District—SH 6 @ University (AM Peak Hour).

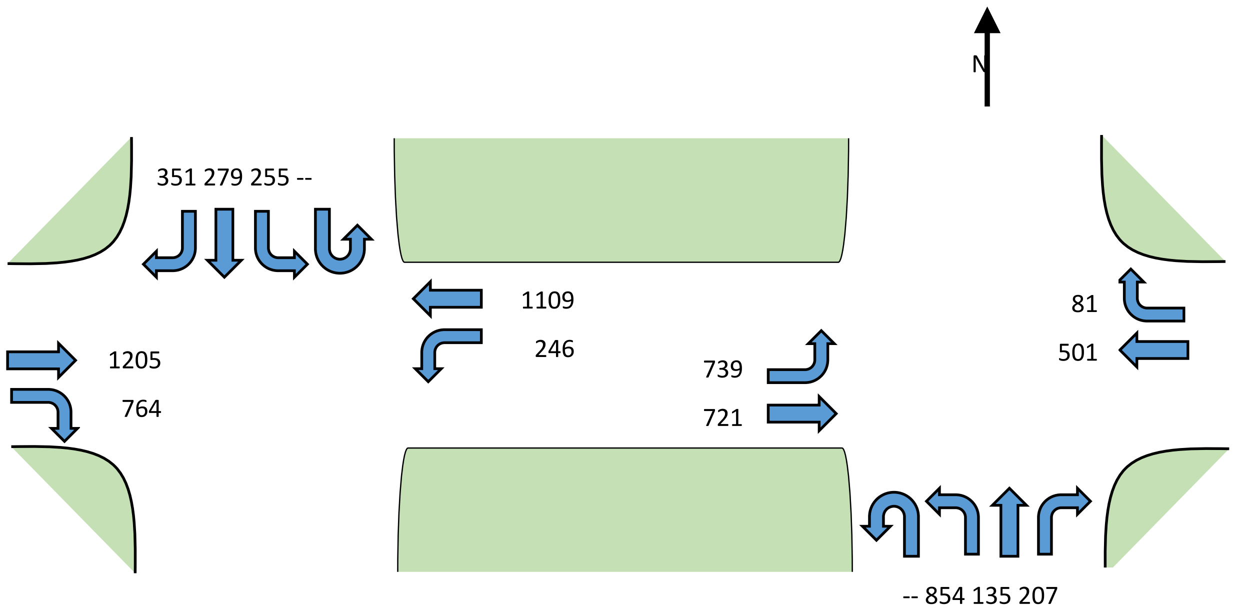

Figure 60. Bryan District—SH 6 @ University (PM Peak Hour).

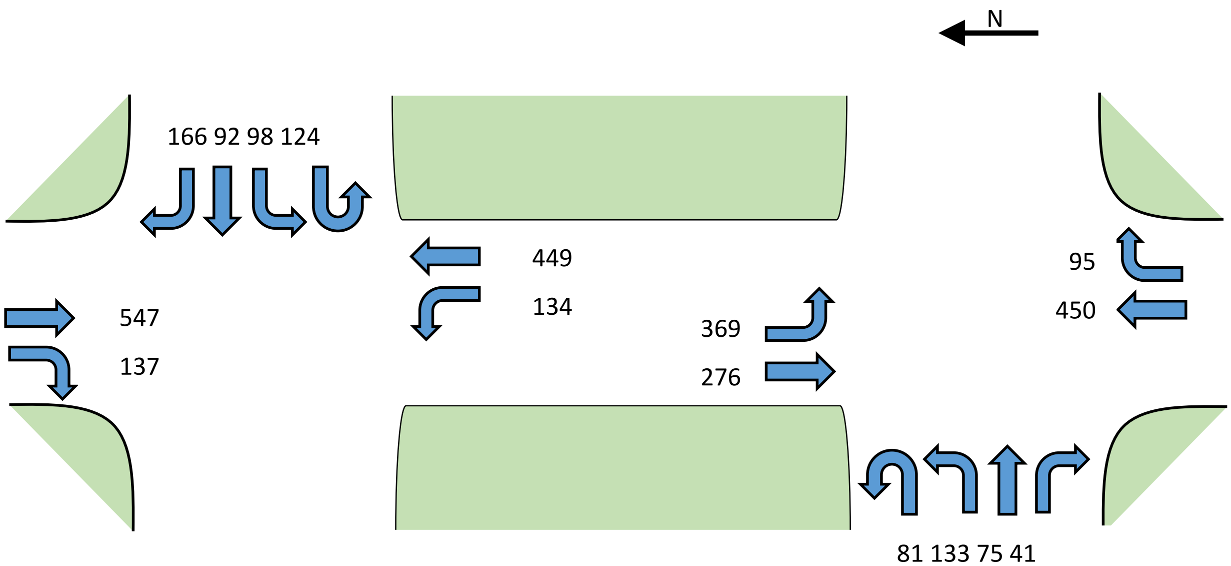

Figure 61. Bryan District—US 290 @ SH 36 (AM Peak Hour).

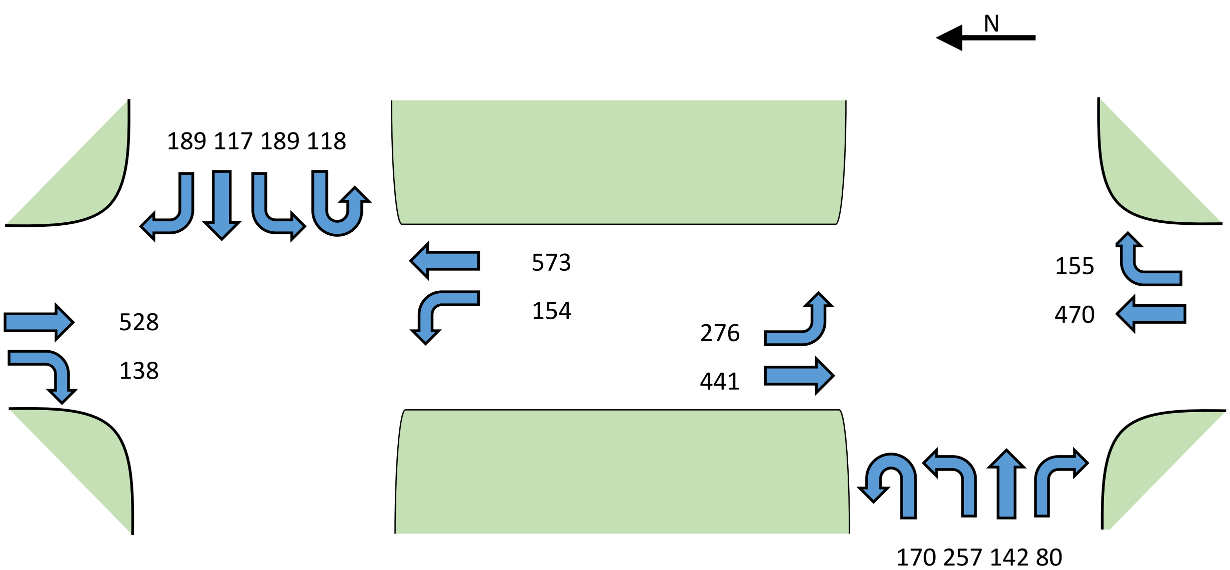

Figure 62. Bryan District—US 290 @ SH 36 (PM Peak Hour).

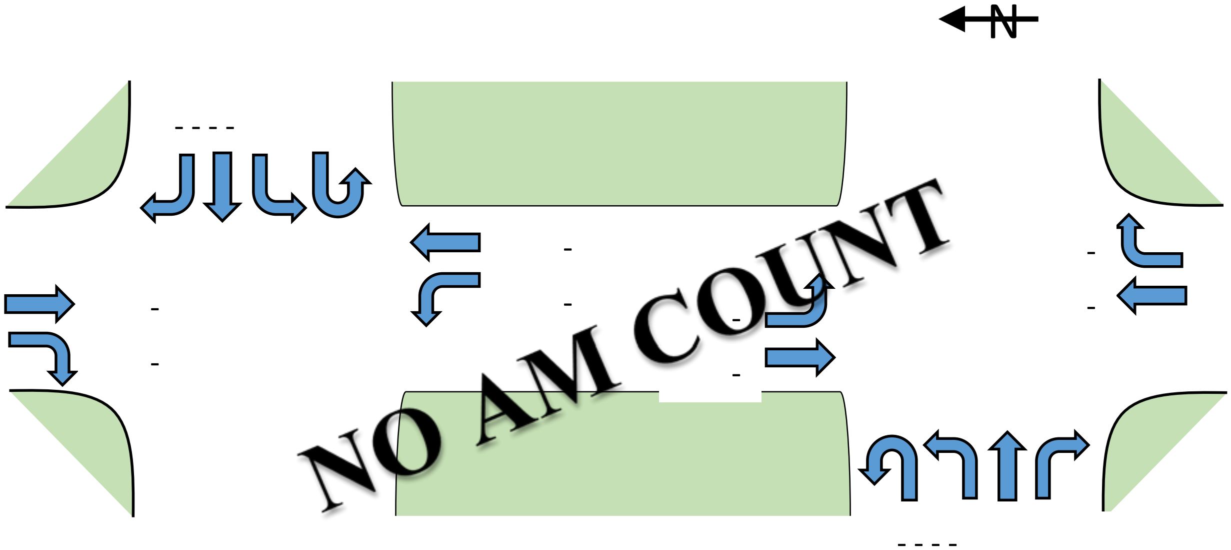

Figure 63. Corpus Christi District—SH 358 @ Greenwood (No AM Count).

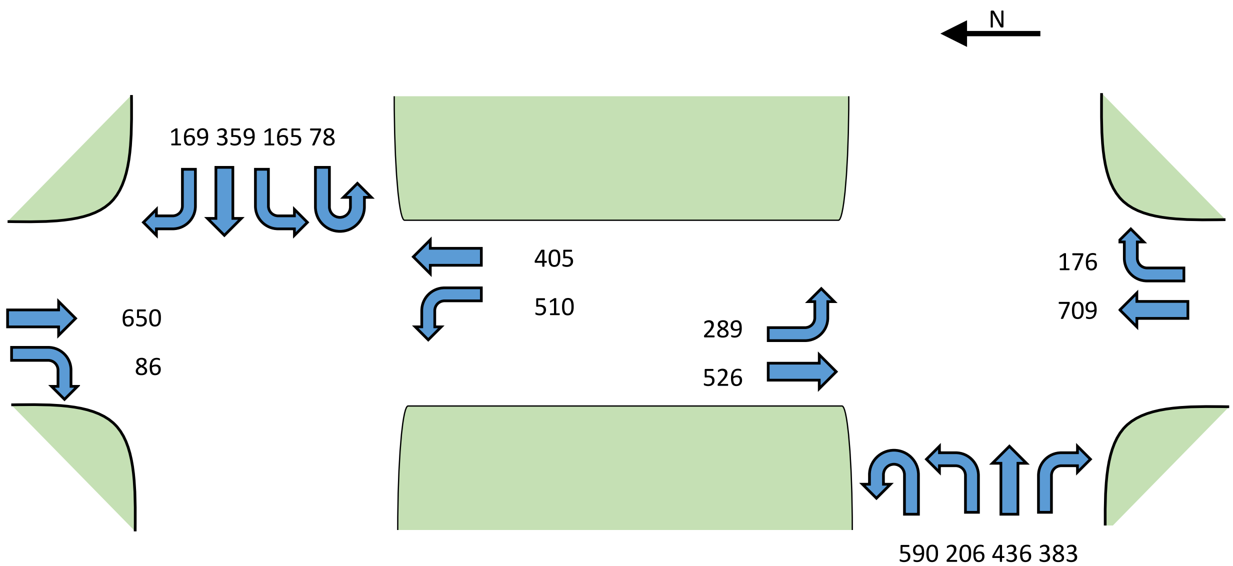

Figure 64. Corpus Christi District—SH 358 @ Greenwood (PM Peak Hour).

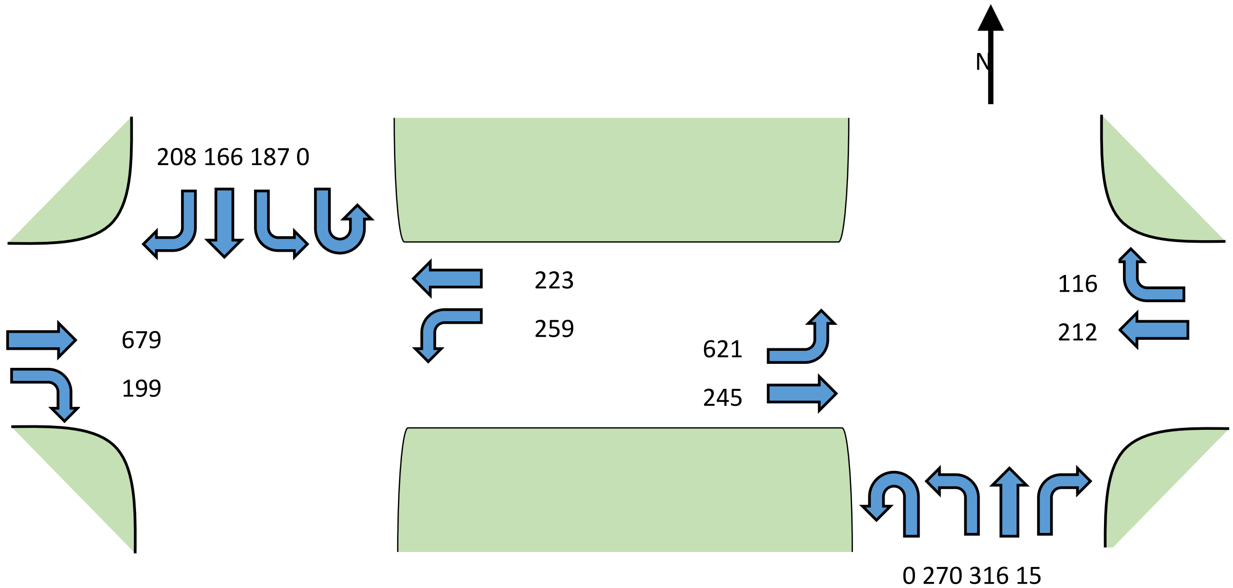

Figure 65. Ft. Worth District—I-35W @ Alsbury (AM Peak Hour).

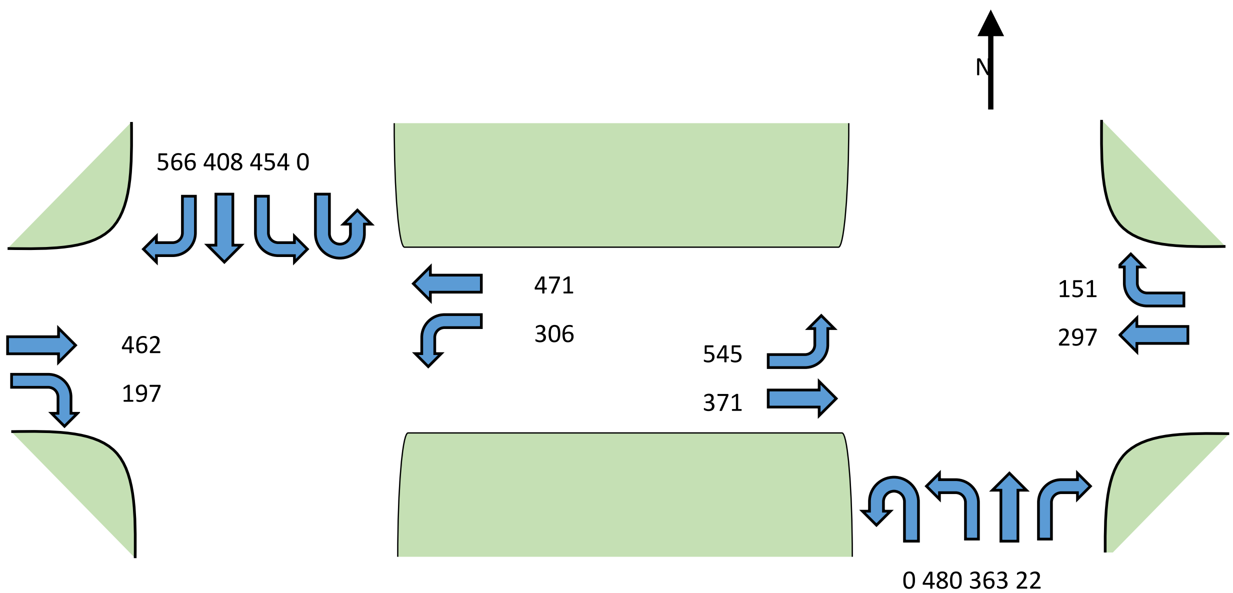

Figure 66. Ft. Worth District—I-35W @ Alsbury (PM Peak Hour).

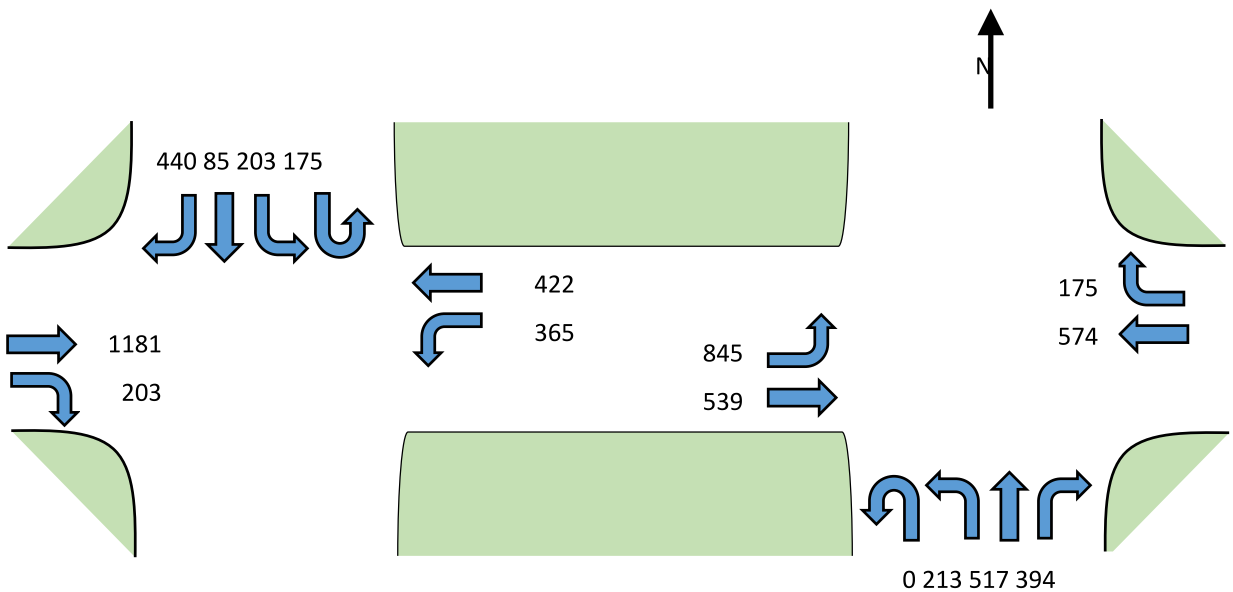

Figure 67. Ft. Worth District—I-35W @ FM 1187 (AM Peak Hour).

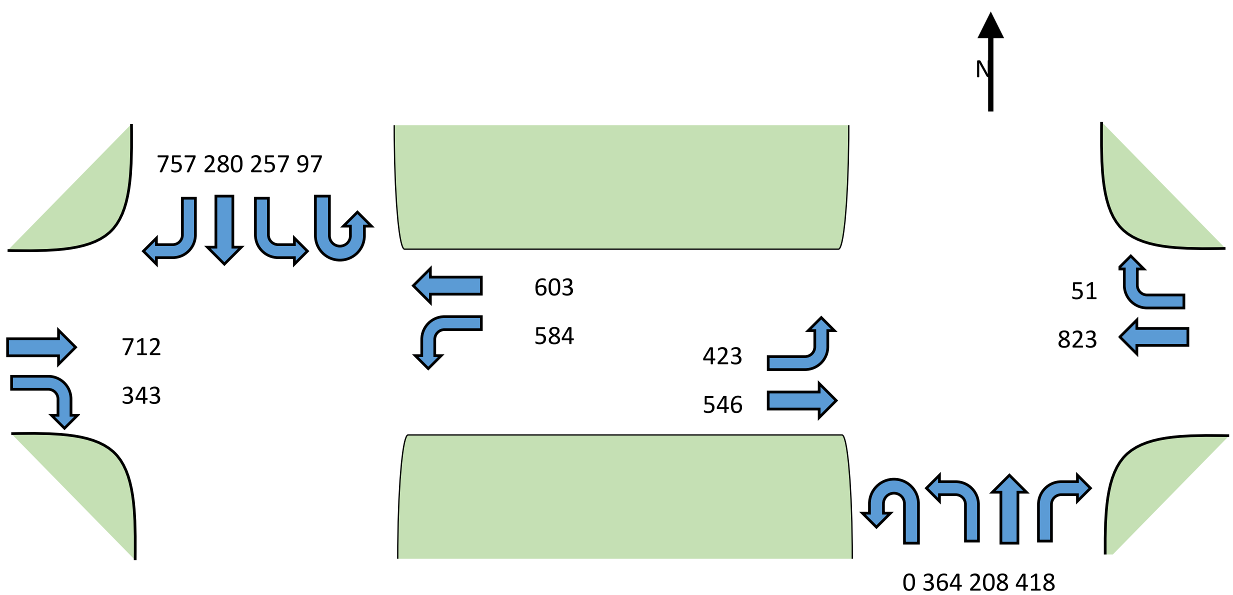

Figure 68. Ft. Worth District—I-35W @ FM 1187 (PM Peak Hour).

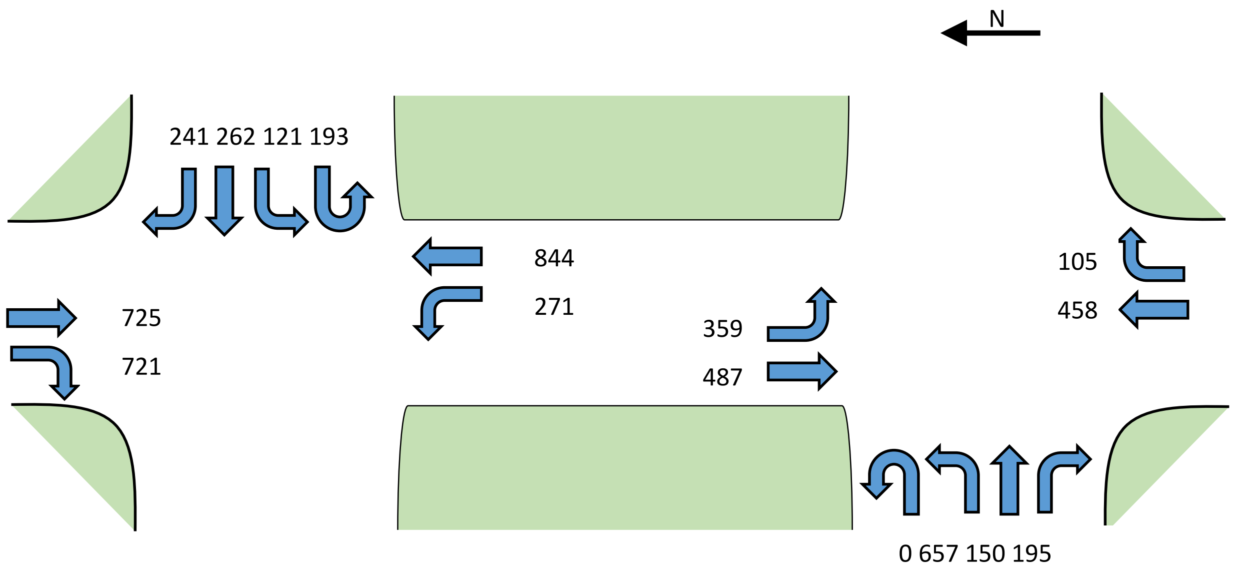

Figure 69. Ft. Worth District—I-20 @ McCart (AM Peak Hour).

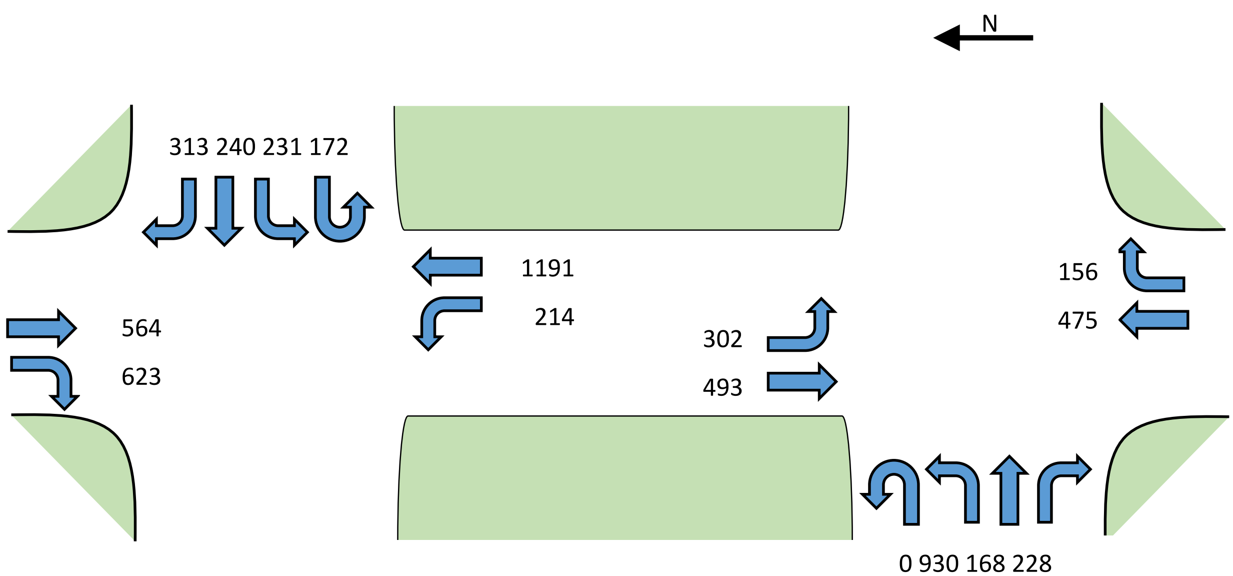

Figure 70. Ft. Worth District—I-20 @ McCart (PM Peak Hour).

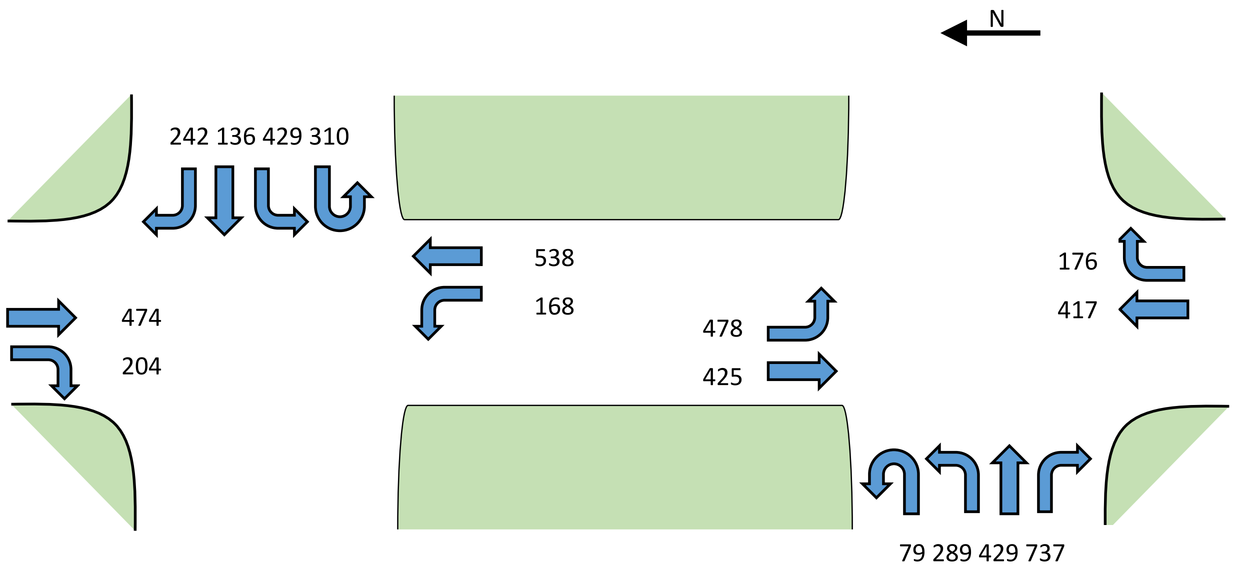

Figure 71. Ft. Worth District—I-20 @ Hulen (AM Peak Hour).

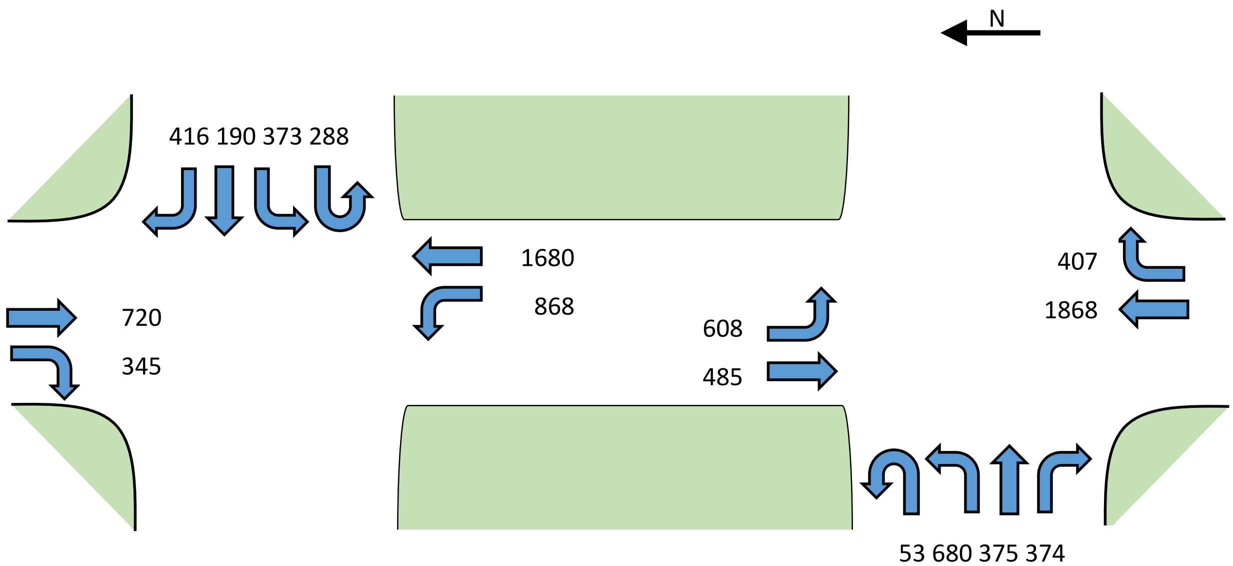

Figure 72. Ft. Worth District—I-20 @ Hulen (PM Peak Hour).

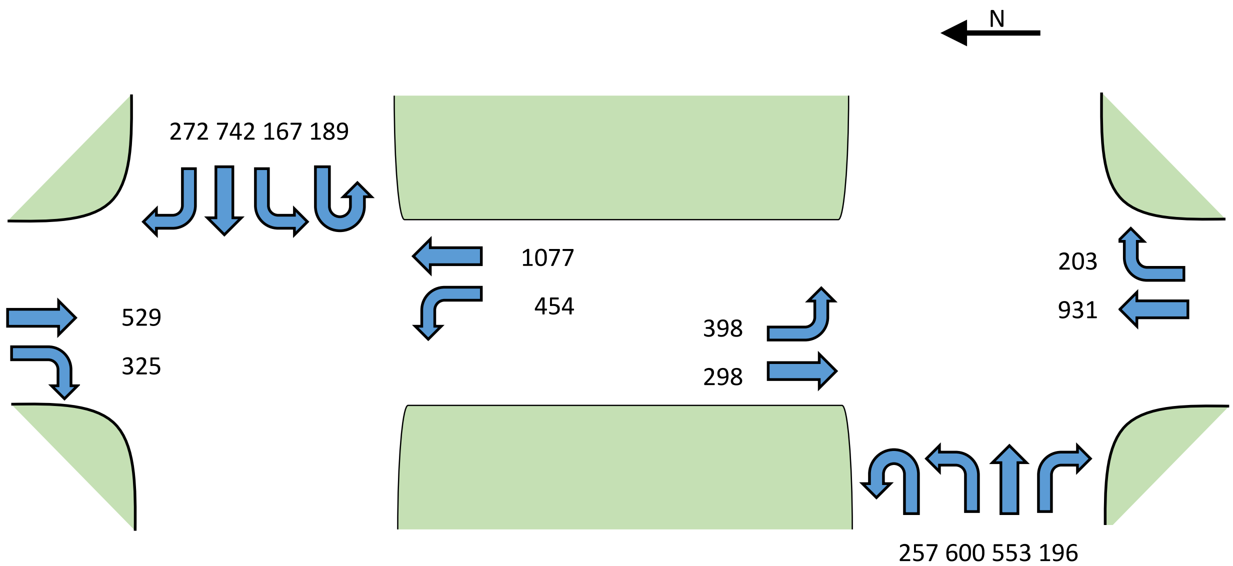

Figure 73. Houston District—I-10 @ Bunker Hill Rd. (AM Peak Hour).

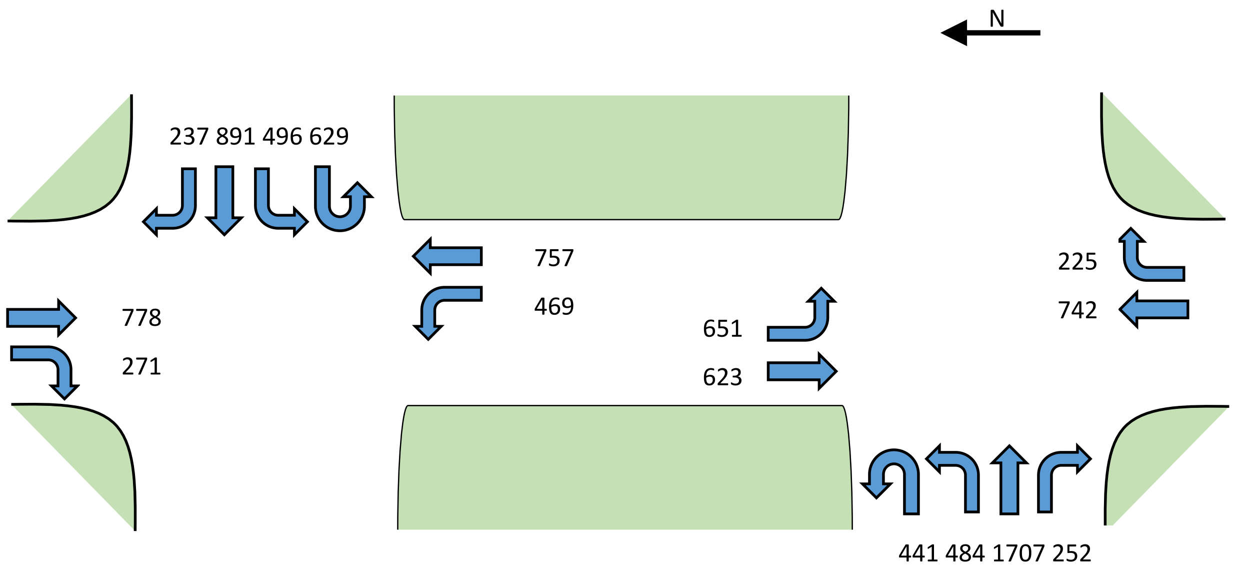

Figure 74. Houston District—I-10 @ Bunker Hill Rd. (PM Peak Hour).

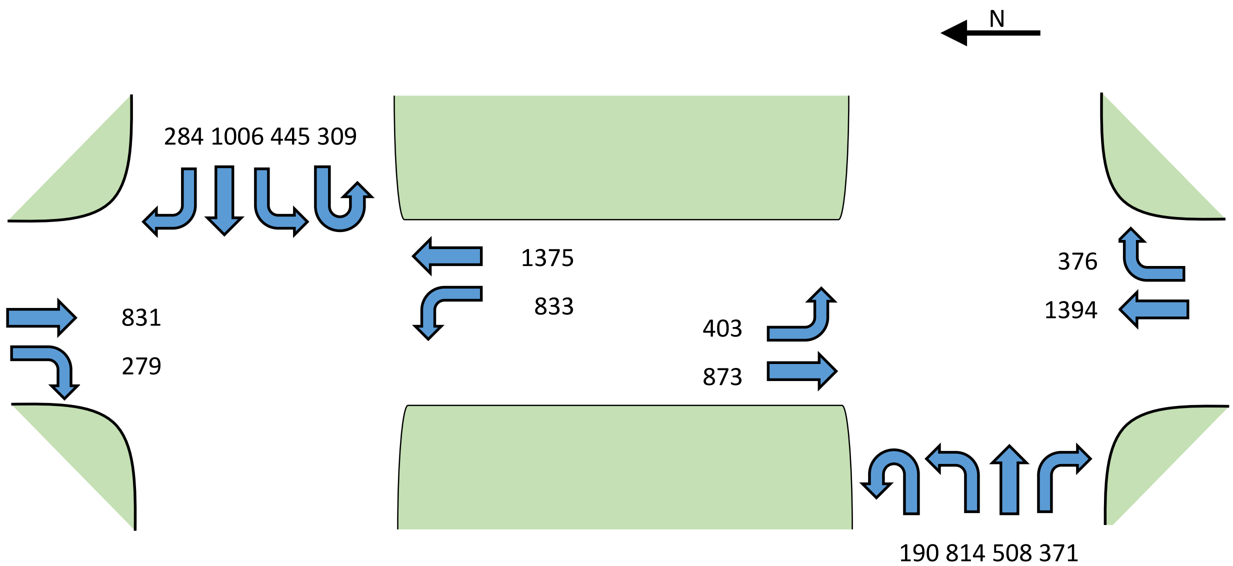

Figure 75. Houston District—I-10 @ Gessner Rd. (AM Peak Hour).

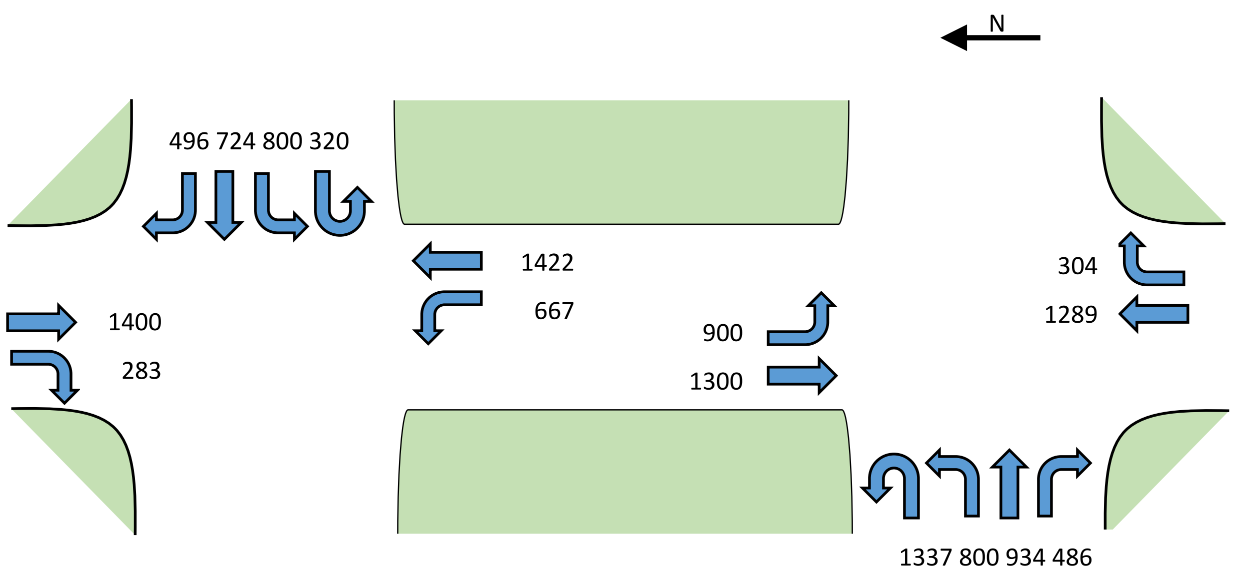

Figure 76. Houston District—I-10 @ Gessner Rd. (PM Peak Hour).

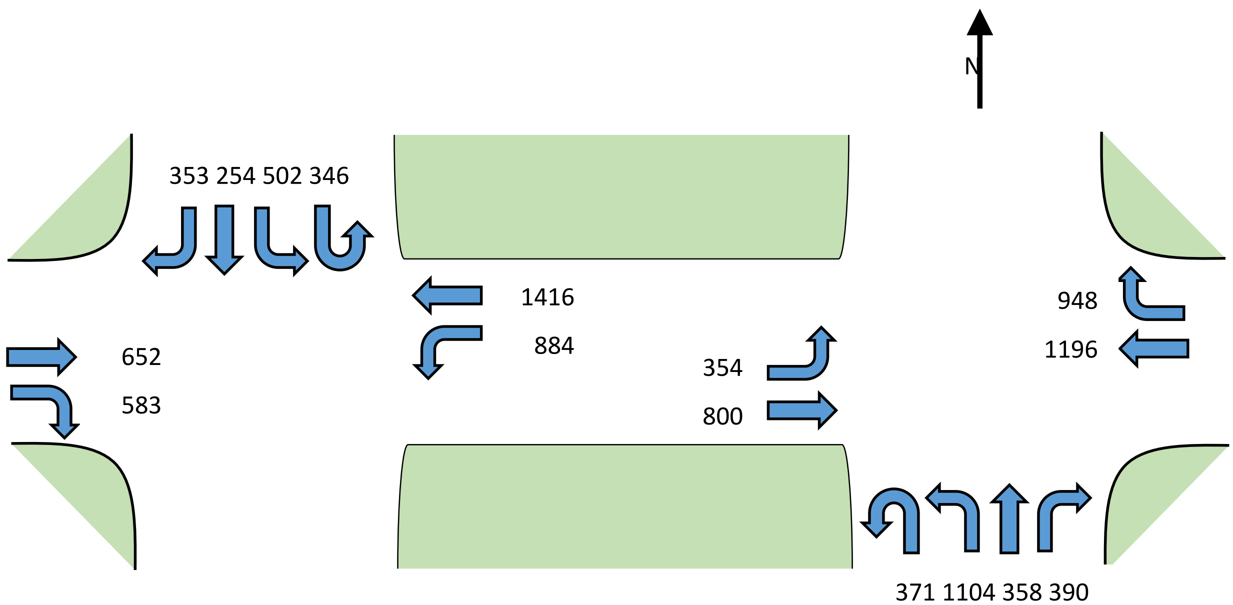

Figure 77. Houston District—I-45 @ Rayford Rd/Sawdust Rd. (AM Peak Hour).

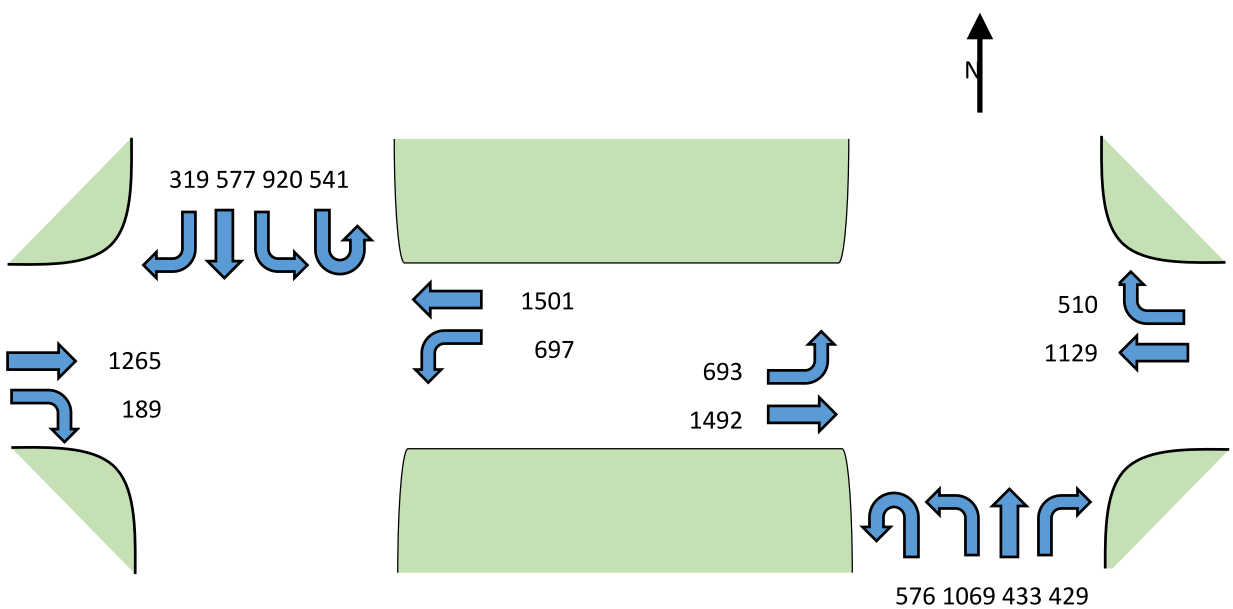

Figure 78. Houston District—I-45 @ Rayford Rd/Sawdust Rd. (PM Peak Hour).

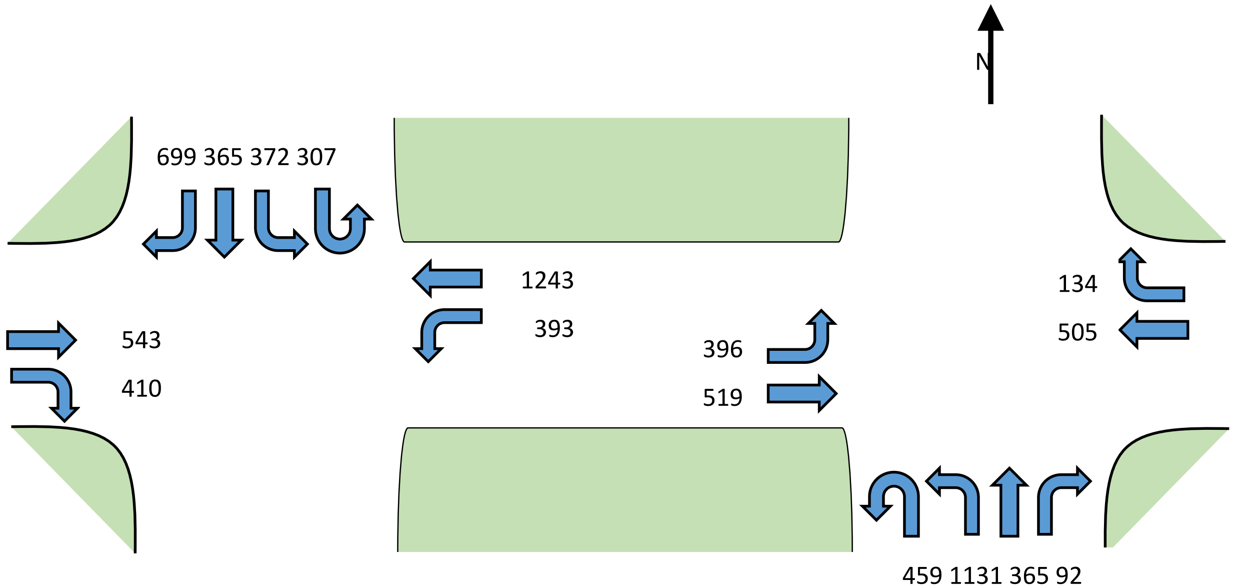

Figure 79. Houston District—I-45 @ Research Forest Dr. (AM Peak Hour).

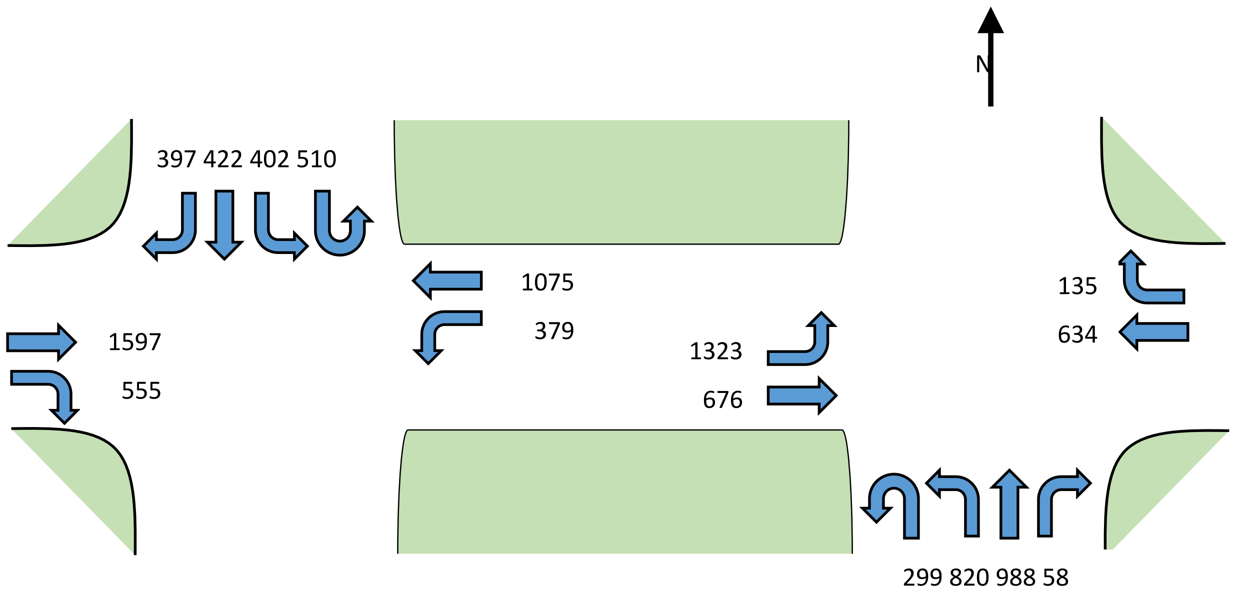

Figure 80. Houston District—I-45 @ Research Forest Dr. (PM Peak Hour).

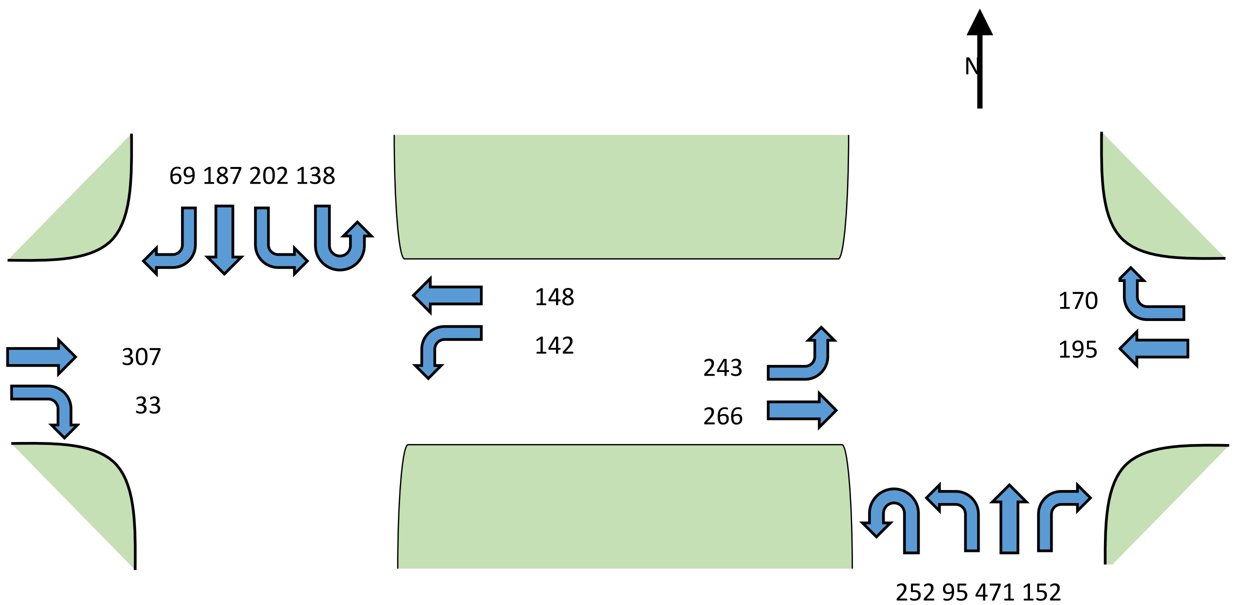

Figure 81. Laredo District—I35 @ Mann (AM Peak Hour).

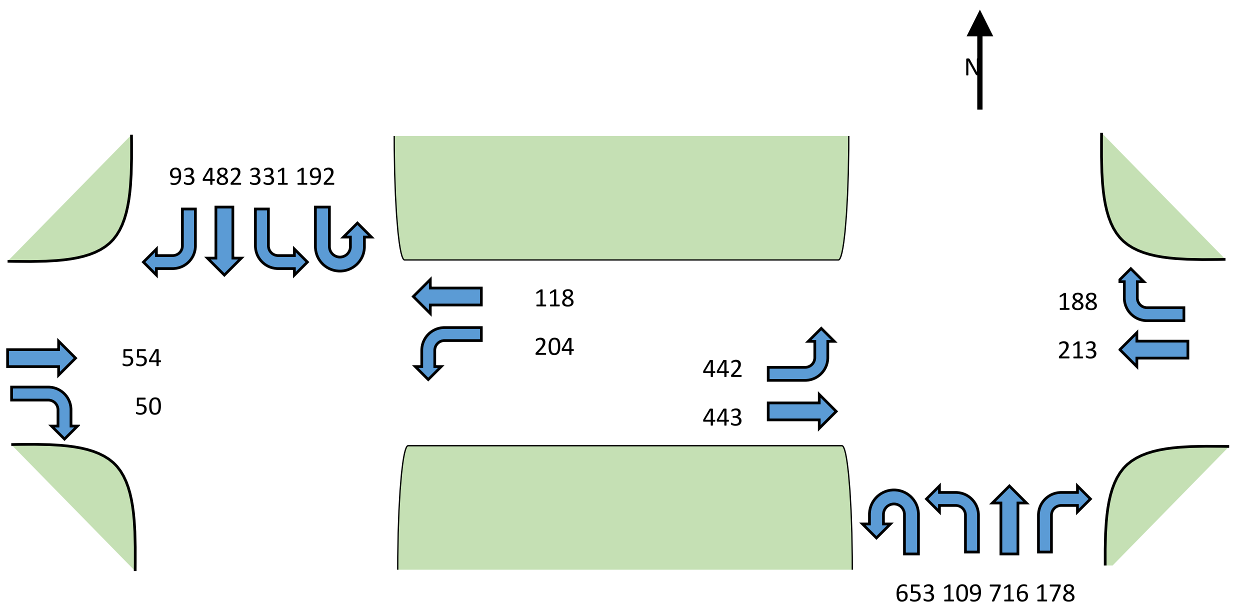

Figure 82. Laredo District—I35 @ Mann (PM Peak Hour).

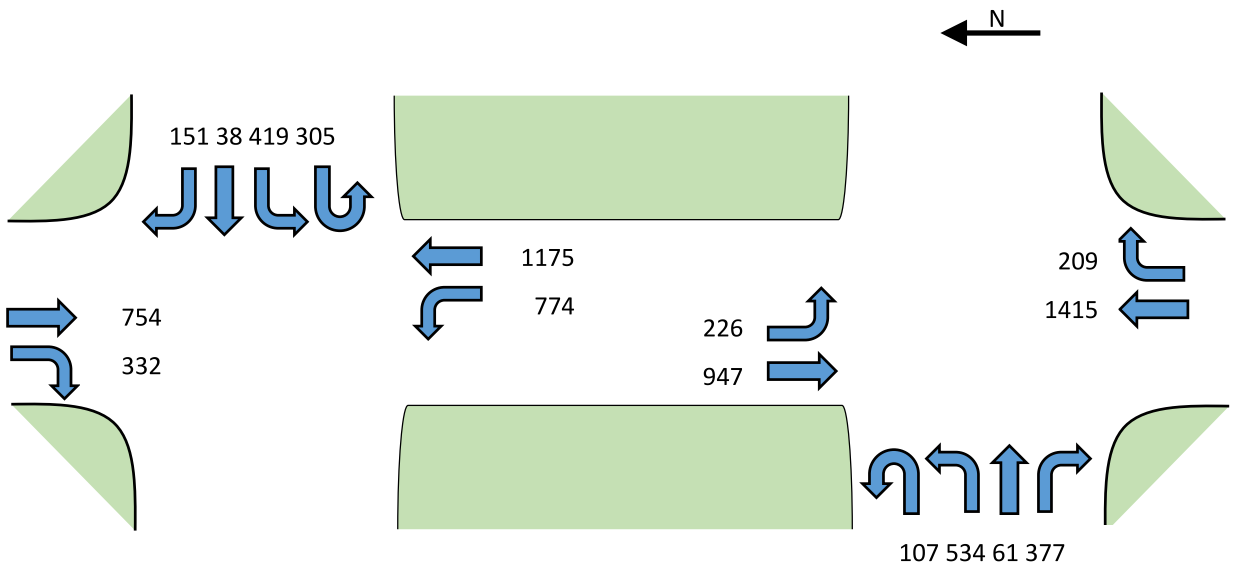

Figure 83. Pharr District—I-2 @ FM 2220 (AM Peak Hour).

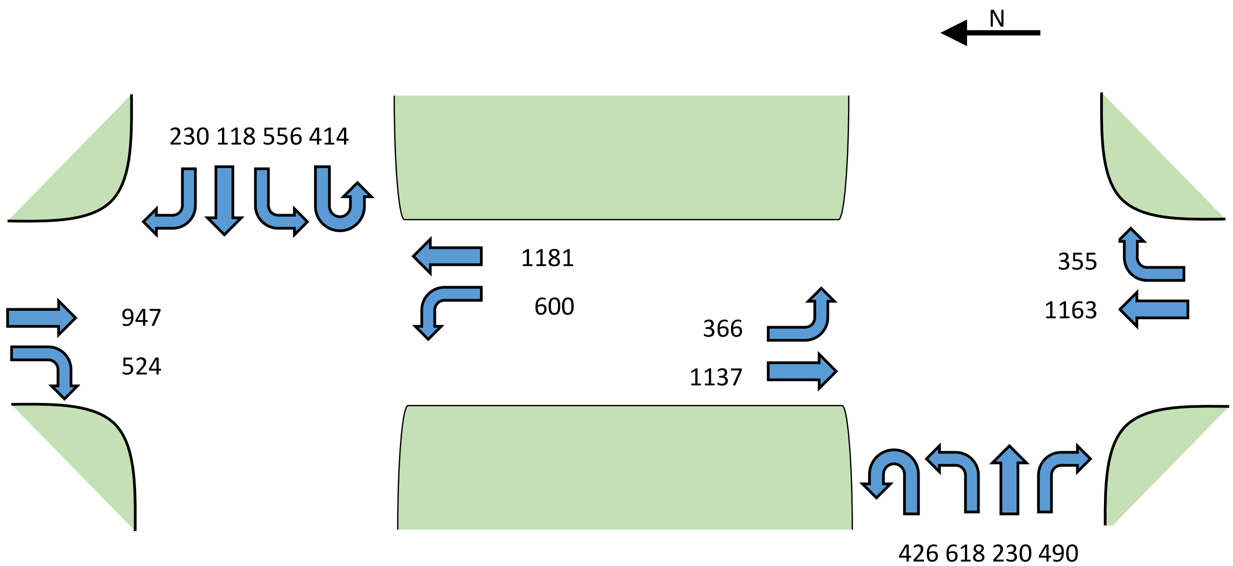

Figure 84. Pharr District—I-2 @ FM 2220 (PM Peak Hour).

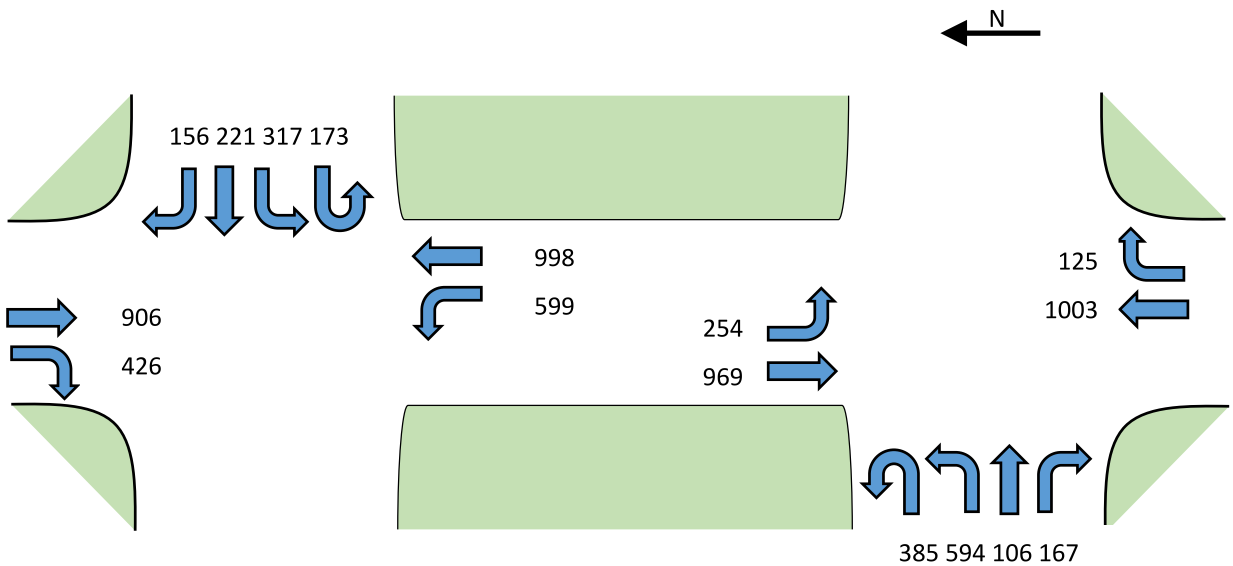

Figure 85. Pharr District—I-2 @ SH 494 (AM Peak Hour).

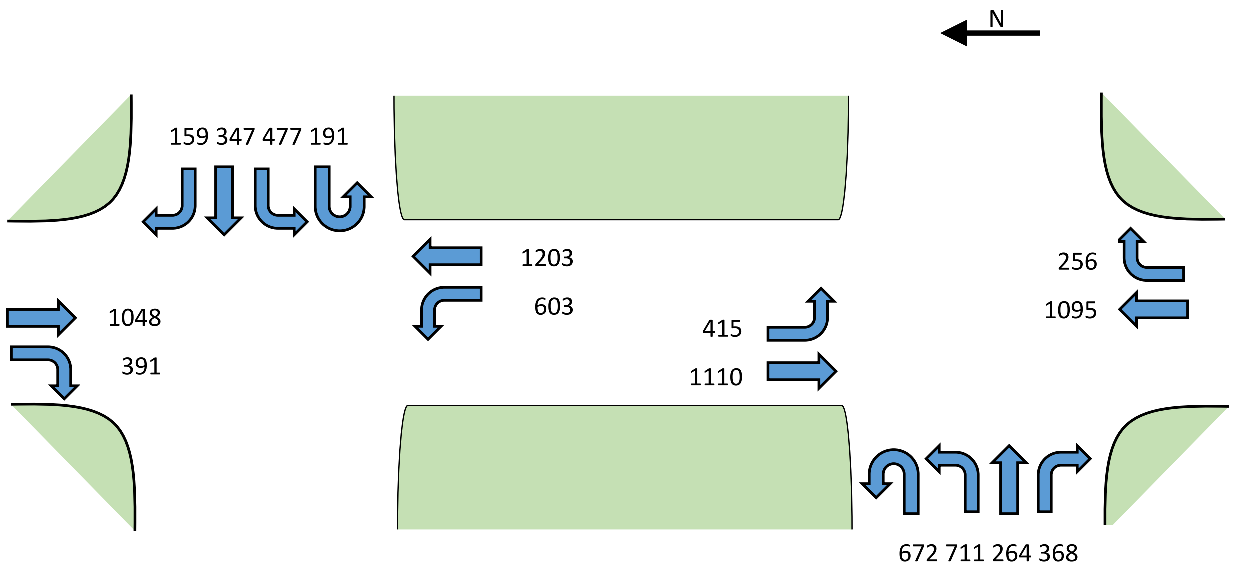

Figure 86. Pharr District—I-2 @ SH 494 (PM Peak Hour).

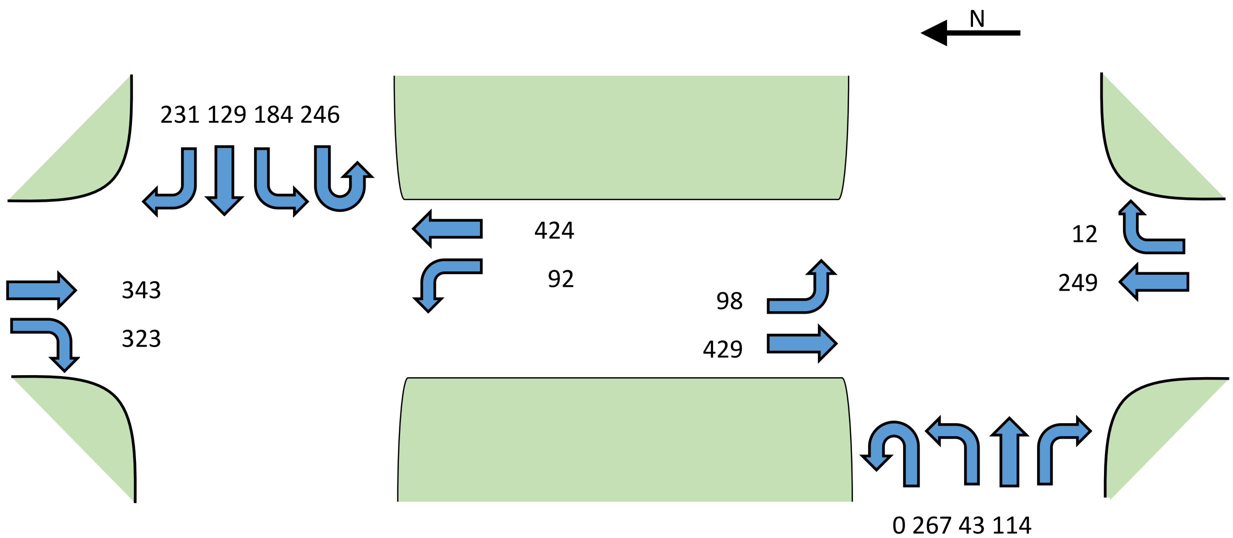

Figure 87. San Angelo District—SH 306 @ US 67 (AM Peak Hour).

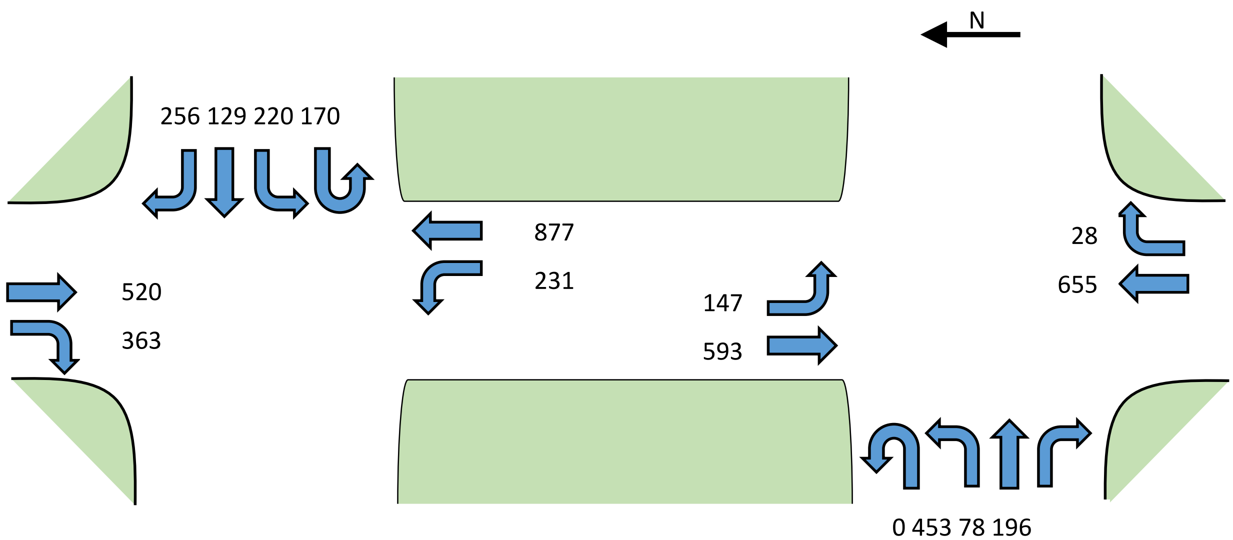

Figure 88. San Angelo District—SH 306 @ US 67 (PM Peak Hour).

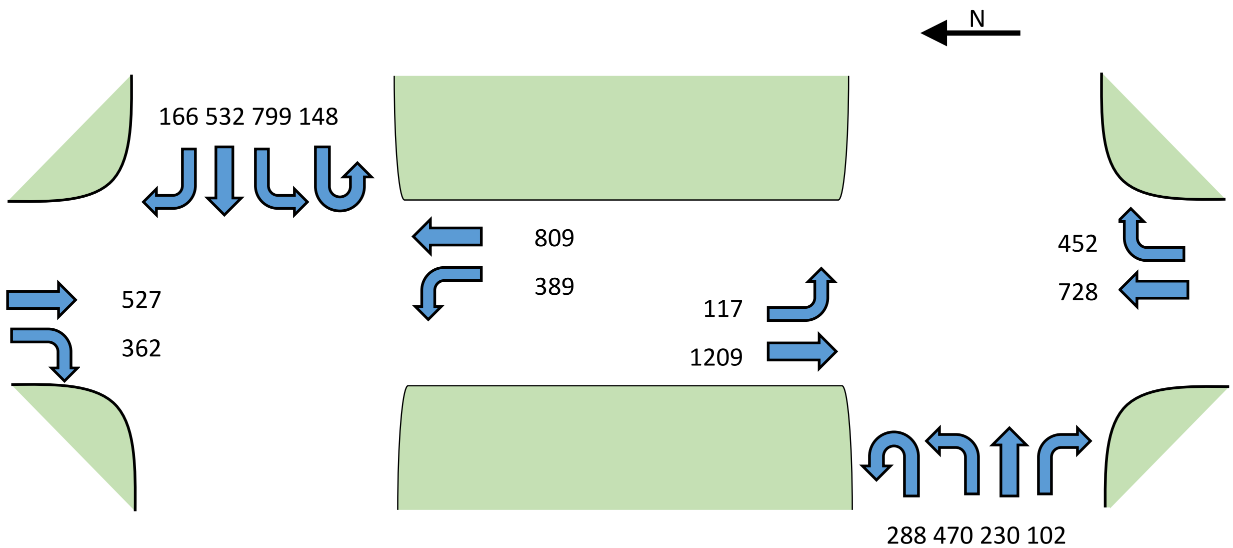

Figure 89. San Antonio District—I-410 @ Callaghan (AM Peak Hour).

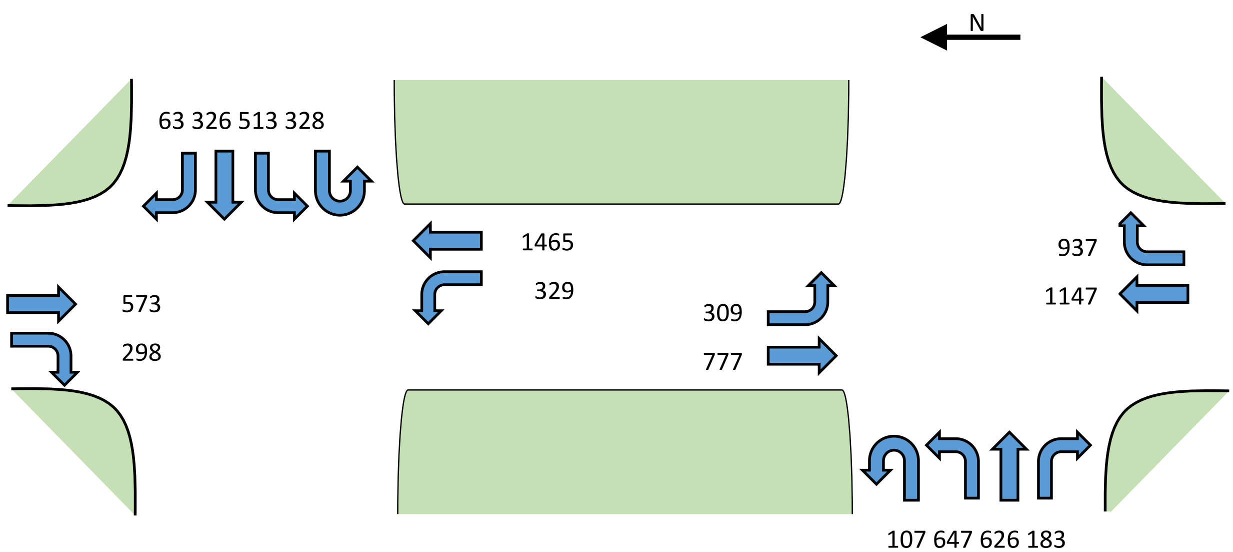

Figure 90. San Antonio District—I-410 @ Callaghan (PM Peak Hour).

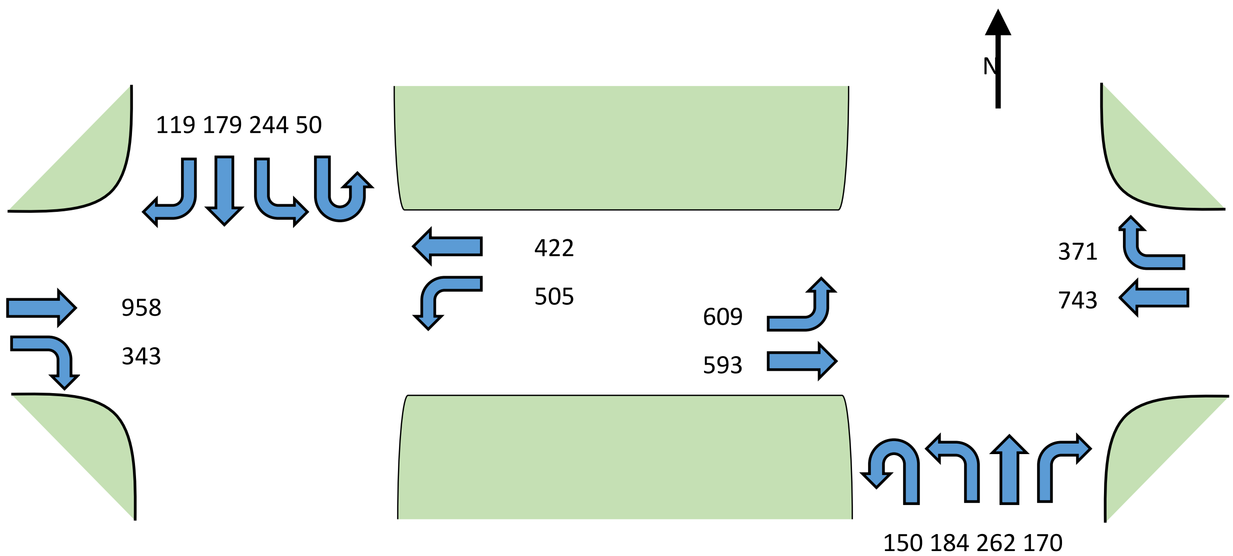

Figure 91. San Antonio District—I-410 @ Ingram (AM Peak Hour).

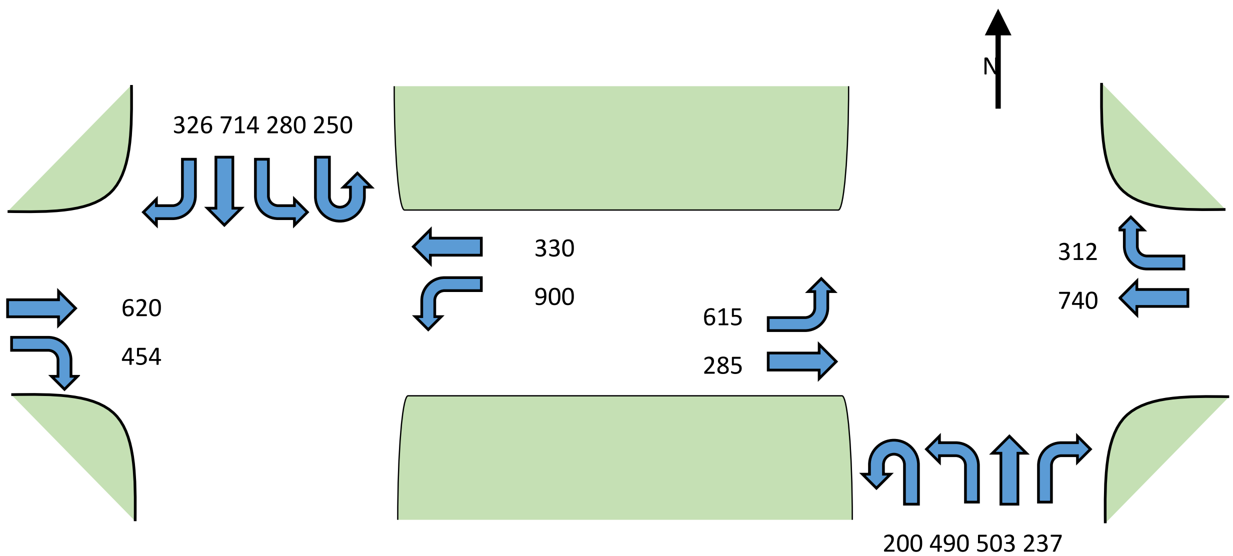

Figure 92. San Antonio District—I-410 @ Ingram (PM Peak Hour).

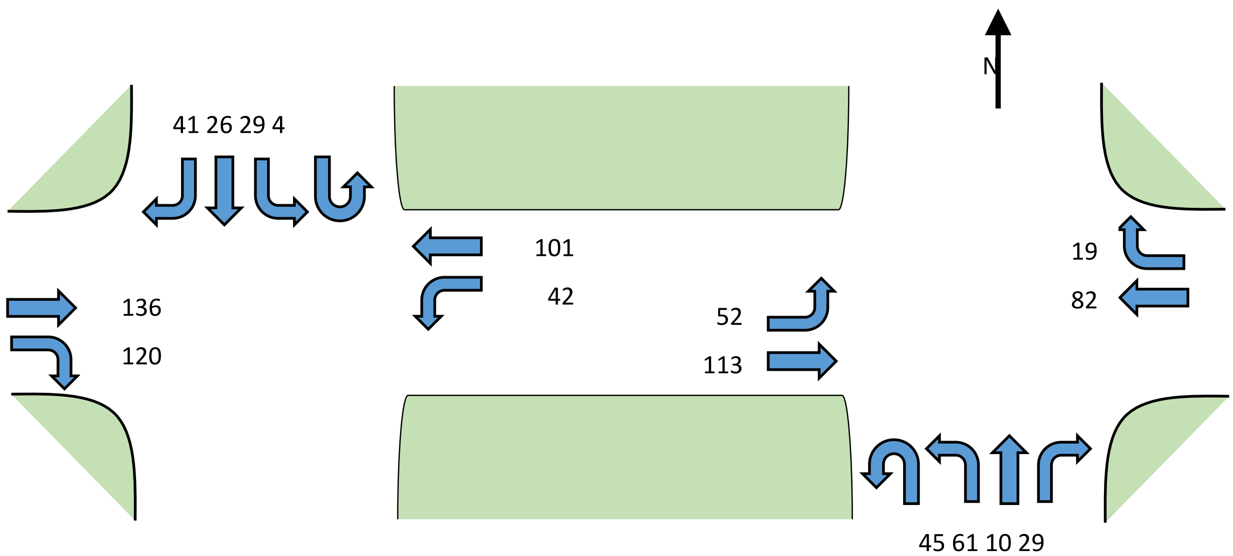

Figure 93. Waco District—I-35 @ FM 286 (AM Peak Hour).

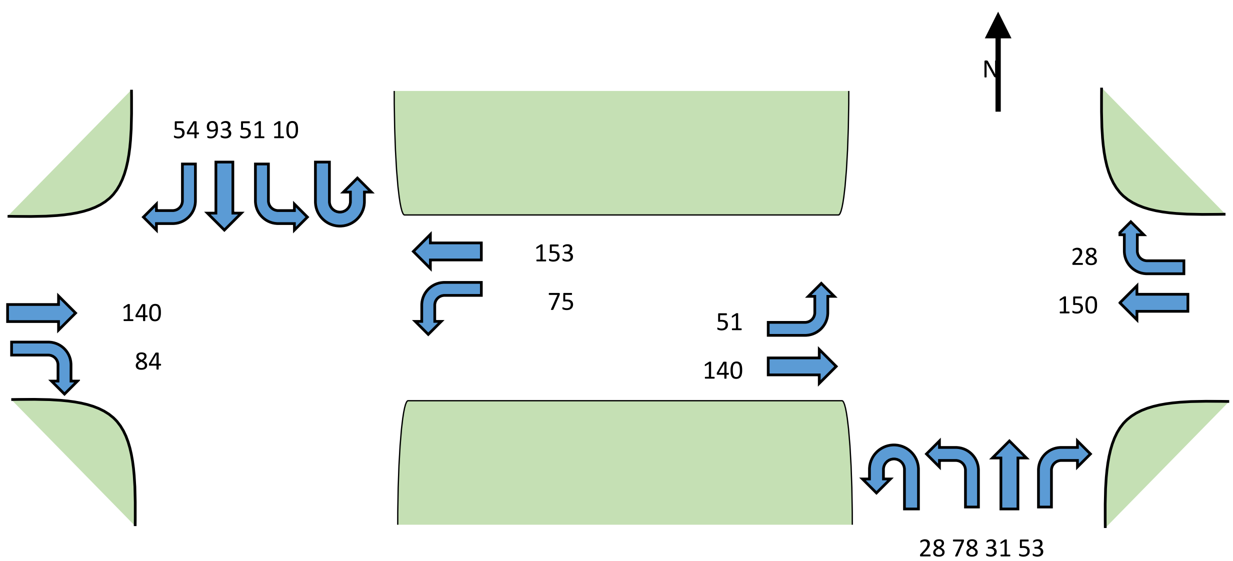

Figure 94. Waco District—I-35 @ FM 286 (PM Peak Hour).

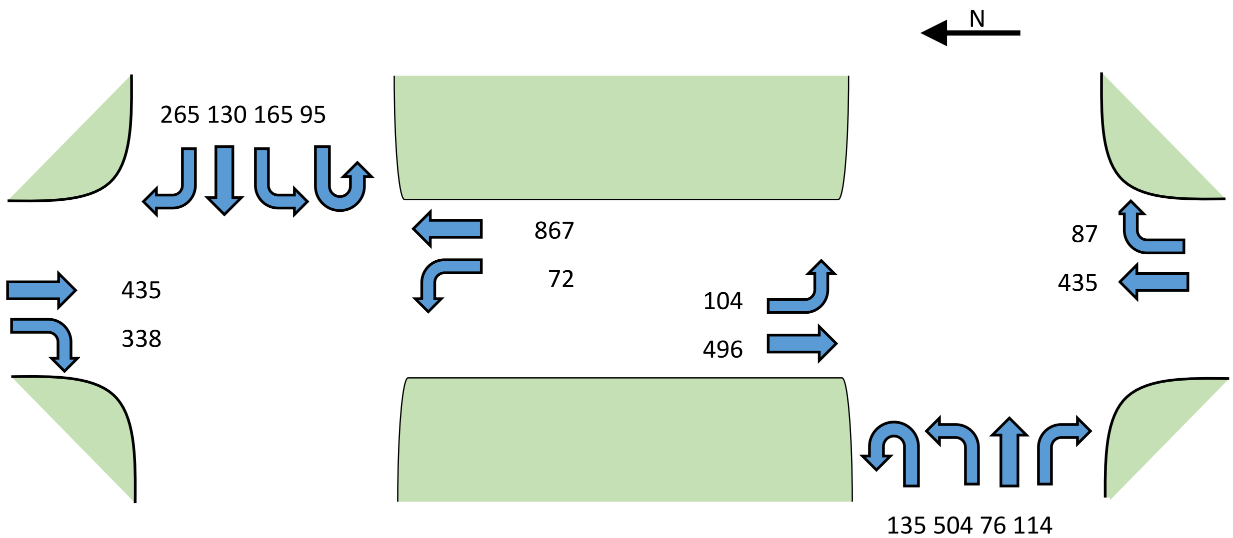

Figure 95. Wichita Falls District—US 82 @ Kemp (AM Peak Hour).

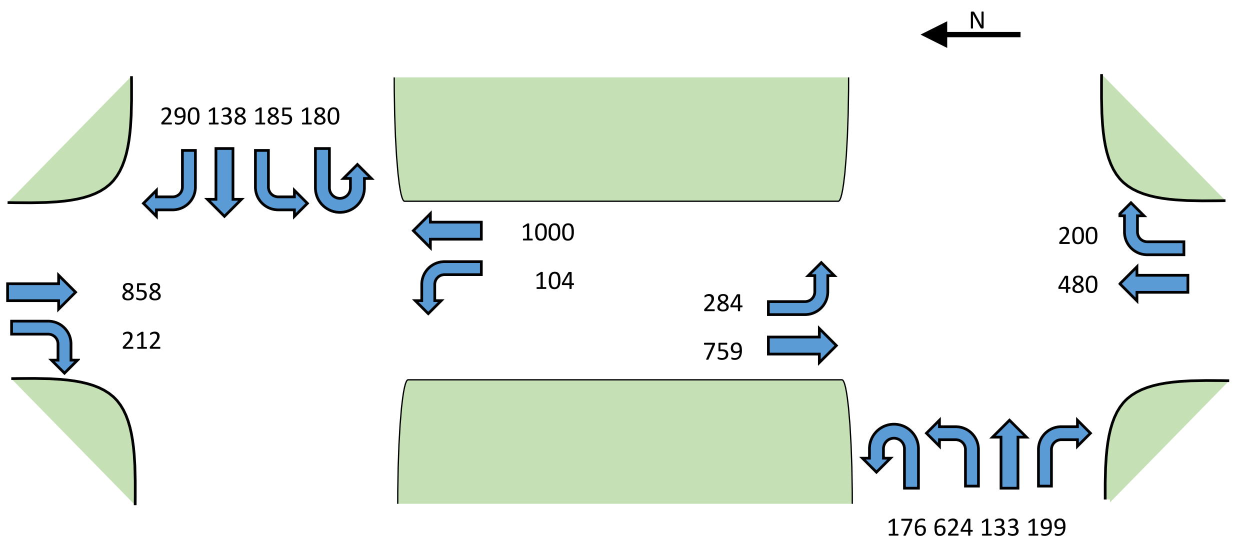

Figure 96. Wichita Falls District—US 82 @ Kemp (PM Peak Hour).

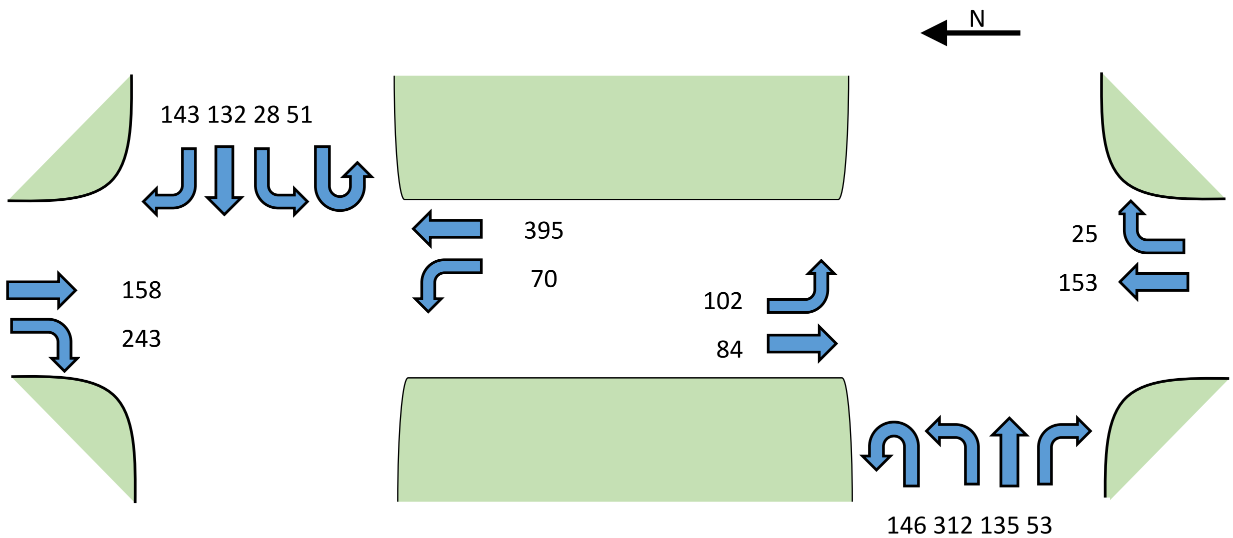

Figure 97. Wichita Falls District—US 82 @ Lawrence (AM Peak Hour).

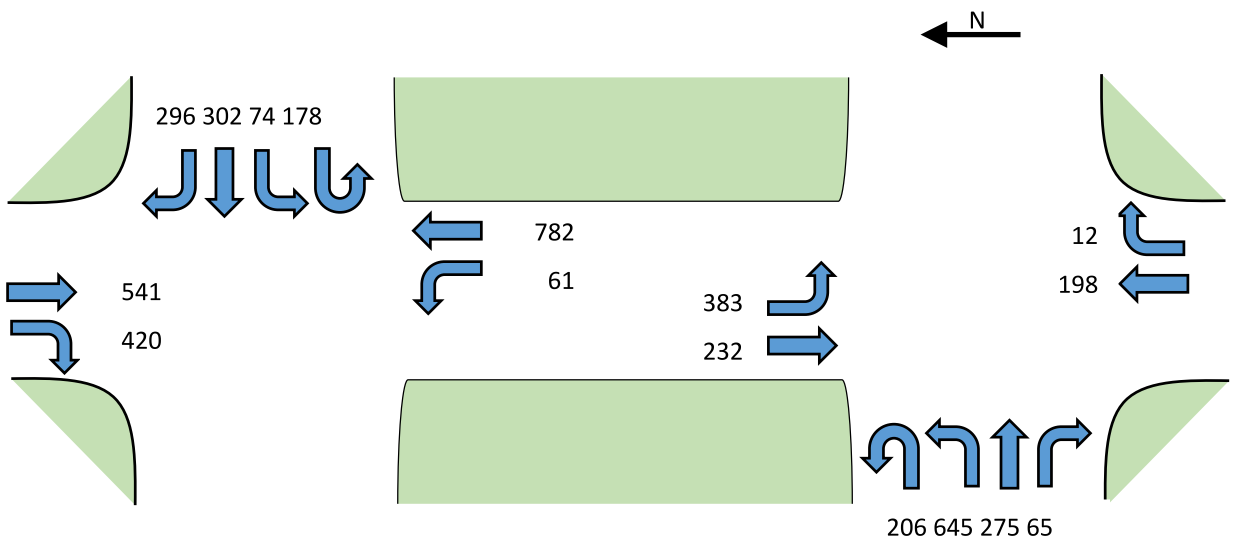

Figure 98. Wichita Falls District—US 82 @ Lawrence (PM Peak Hour).

APPENDIX C. BASE DATA FROM SIMULATION

Table 45. VISSIM Results Summary—Abilene District—I-20 @ SH 351.

Site Name: I-20 @ SH 351 in Abilene District
Time Period: AM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	81	271	55	186	373	92	29	75	75	74	77	173	7	8	**1575**
Avg. Queue Length (ft)	11	11	11	11	11	11	4	9	9	0	8	15	15	0	**6**
Max. Queue Length (ft)	104	104	104	108	108	108	82	82	82	37	135	135	135	14	**140**
Avg. Delay (sec/veh)	37.1	24.7	1.2	35.2	24.2	1.3	0.9	33.3	19.1	1.7	0.8	29.7	20.4	1.4	**22.1**
Stopped Delay (sec/veh)	25.6	12.6	0.0	24.1	12.1	0.0	0.0	20.9	11.2	0.1	0.0	17.7	12.3	0.3	**12.7**
Avg. Stops (stops/veh)	1.40	0.89	0.00	1.33	0.88	0.01	0.01	1.33	0.69	0.07	0.01	1.19	0.70	0.17	**0.83**

Site Name: I-20 @ SH 351 in Abilene District
Time Period: PM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	173	739	89	243	516	117	64	88	109	128	188	239	17	13	**2724**
Avg. Queue Length (ft)	34	34	34	20	20	20	9	17	17	1	17	34	34	0	**14**
Max. Queue Length (ft)	207	207	207	155	155	155	120	120	120	77	200	200	200	18	**217**
Avg. Delay (sec/veh)	50.3	31.3	1.4	60.0	30.2	1.6	1.2	41.5	26.6	3.6	1.3	43.2	25.2	2.2	**29.5**
Stopped Delay (sec/veh)	37.0	16.8	0.0	45.4	16.9	0.0	0.0	28.5	18.4	1.1	0.1	29.3	17.6	0.9	**18.5**
Avg. Stops (stops/veh)	1.42	1.04	0.00	1.66	1.02	0.01	0.02	1.42	0.71	0.24	0.05	1.43	0.70	0.18	**0.93**

Table 46. VISSIM Results Summary—Bryan District—SH 6 @ Boonville.

Site Name: SH 6 @ Boonville in Bryan District Time Period: AM Peak Hour														
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	292	302	489	183	692	195	102	875	162	53	41	411	160	430	**4387**
Avg. Queue Length (ft)	22	22	22	96	96	96	137	137	137	5	82	82	82	53	**66**
Max. Queue Length (ft)	172	172	172	376	376	376	760	760	760	359	334	334	334	325	**760**
Avg. Delay (sec/veh)	63.6	22.0	4.9	84.5	46.4	2.6	88.3	48.1	31.8	4.6	77.6	78.4	46.9	12.9	**41.2**
Stopped Delay (sec/veh)	52.5	13.1	0.1	66.5	33.4	0.1	68.1	26.7	22.2	1.2	62.2	58.0	36.4	6.1	**28.2**
Avg. Stops (stops/veh)	1.04	0.57	0.07	1.79	0.99	0.04	1.49	0.97	0.65	0.16	1.81	1.77	0.87	0.52	**0.87**

Site Name: SH 6 @ Boonville in Bryan District Time Period: PM Peak Hour														
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	451	690	824	320	479	278	88	746	332	168	40	376	122	174	**5087**
Avg. Queue Length (ft)	71	71	71	80	80	80	134	134	134	41	64	64	64	12	**67**
Max. Queue Length (ft)	851	851	851	339	339	339	682	682	682	572	236	236	236	229	**961**
Avg. Delay (sec/veh)	79.1	28.8	20.4	100.7	43.1	4.2	127.5	52.5	38.3	5.5	72.9	71.1	45.7	7.5	**44.6**
Stopped Delay (sec/veh)	58.4	13.7	0.4	79.6	31.3	0.6	102.9	32.3	27.2	1.5	56.5	51.8	36.9	3.9	**28.7**
Avg. Stops (stops/veh)	1.21	0.60	0.17	1.93	0.93	0.13	2.02	1.00	0.73	0.24	1.84	1.73	0.79	0.35	**0.84**

Table 47. VISSIM Results Summary—Bryan District—SH 6 @ Briarcrest.

Site Name: SH 6 @ Briarcrest in Bryan District Time Period: AM Peak Hour															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	413	359	357	411	576	106	94	817	147	238	12	116	205	612	**4463**
Avg. Queue Length (ft)	57	57	57	91	91	23	81	81	81	81	41	41	41	41	**59**
Max. Queue Length (ft)	222	222	222	352	352	239	342	342	342	342	342	342	342	342	**388**
Avg. Delay (sec/veh)	52.3	40.3	2.4	53.4	49.2	1.2	48.3	33.8	26.8	3.1	83.9	38.0	34.2	12.1	**32.3**
Stopped Delay (sec/veh)	33.0	26.2	0.2	32.5	30.5	0.0	33.1	23.1	19.3	0.6	71.9	31.9	26.6	3.2	**20.4**
Avg. Stops (stops/veh)	1.04	0.73	0.04	0.87	0.77	0.00	1.57	0.69	0.60	0.11	1.84	0.79	0.68	0.46	**0.64**
Site Name: SH 6 @ Briarcrest in Bryan District Time Period: PM Peak Hour															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	691	739	658	372	264	121	77	670	269	270	11	110	214	448	**4913**
Avg. Queue Length (ft)	105	105	105	74	74	10	93	93	93	93	40	40	40	40	**65**
Max. Queue Length (ft)	476	476	476	253	253	141	347	347	347	347	185	185	185	185	**476**
Avg. Delay (sec/veh)	64.3	25.9	4.0	65.9	60.0	0.9	73.8	42.8	35.2	4.5	97.3	75.9	43.3	6.4	**35.3**
Stopped Delay (sec/veh)	38.0	16.1	0.3	40.8	37.9	0.0	54.6	31.5	26.1	1.1	77.1	61.0	34.6	1.7	**22.8**
Avg. Stops (stops/veh)	1.28	0.56	0.05	1.37	1.35	0.00	1.79	0.78	0.70	0.20	1.95	1.62	0.78	0.27	**0.73**

Table 48. VISSIM Results Summary—Bryan District—SH 6 @ Rock Prairie.

Site Name: SH 6 @ Rock Prairie in Bryan District
Time Period: AM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	534	144	51	105	139	115	61	151	73	55	195	258	129	303	**2313**
Avg. Queue Length (ft)	76	76	76	55	55	55	12	25	25	25	26	48	48	48	**34**
Max. Queue Length (ft)	411	411	411	205	205	205	128	128	128	128	233	233	233	233	**411**
Avg. Delay (sec/veh)	26.9	21.3	22.0	47.8	50.9	49.8	0.7	33.0	37.4	2.2	4.1	42.8	31.3	2.3	**26.3**
Stopped Delay (sec/veh)	18.5	15.1	18.0	34.6	35.2	43.2	0.0	27.4	27.3	0.6	1.7	31.3	23.3	0.1	**19.1**
Avg. Stops (stops/veh)	0.68	0.56	0.66	1.13	1.15	0.97	0.02	0.89	0.84	0.15	0.26	1.14	0.67	0.04	**0.66**

Site Name: SH 6 @ Rock Prairie in Bryan District
Time Period: PM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	550	136	126	166	169	167	170	208	122	52	207	255	164	397	**2891**
Avg. Queue Length (ft)	104	104	104	102	102	102	17	34	34	34	24	44	44	44	**48**
Max. Queue Length (ft)	468	468	468	303	303	303	158	158	158	158	239	239	239	239	**468**
Avg. Delay (sec/veh)	32.6	28.0	26.2	63.3	63.1	69.0	1.5	32.4	37.0	2.1	4.5	37.6	28.3	3.4	**29.8**
Stopped Delay (sec/veh)	23.0	19.7	20.9	47.2	46.2	58.8	0.0	26.2	26.3	0.6	1.9	28.4	20.6	0.2	**21.9**
Avg. Stops (stops/veh)	0.77	0.66	0.67	1.28	1.21	1.19	0.02	0.86	0.86	0.18	0.28	1.03	0.63	0.09	**0.68**

Table 49. VISSIM Results Summary—Bryan District—SH 6 @ SH 40.

Site Name: SH 6 @ SH 40 in Bryan District Time Period: AM Peak Hour															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	454	243	56	54	169	305	13	56	54	24	134	321	54	184	**2121**
Avg. Queue Length (ft)	24	24	24	7	7	7	6	12	12	12	19	39	39	39	**14**
Max. Queue Length (ft)	248	248	248	79	79	79	66	66	66	66	172	172	172	172	**249**
Avg. Delay (sec/veh)	71.0	11.1	9.9	18.5	14.1	4.7	0.8	42.9	38.5	0.5	0.8	46.5	37.0	0.8	**29.2**
Stopped Delay (sec/veh)	56.5	6.5	7.1	11.3	9.1	1.1	0.0	31.1	29.1	0.3	0.0	30.8	27.9	0.0	**21.2**
Avg. Stops (stops/veh)	1.38	0.39	0.39	0.80	0.48	0.18	0.00	1.45	0.77	0.12	0.00	1.11	0.74	0.01	**0.68**
Site Name: SH 6 @ SH 40 in Bryan District Time Period: PM Peak Hour															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	522	572	92	56	222	337	90	228	99	57	256	407	121	491	**3551**
Avg. Queue Length (ft)	34	34	34	10	10	10	19	38	38	38	32	63	63	63	**24**
Max. Queue Length (ft)	314	314	314	102	102	102	149	149	149	149	236	236	236	236	**316**
Avg. Delay (sec/veh)	87.2	13.6	12.0	23.4	16.4	5.6	1.4	56.1	48.1	1.9	1.6	60.4	46.4	2.5	**31.2**
Stopped Delay (sec/veh)	71.4	7.9	7.8	14.8	10.8	1.6	0.0	41.0	37.6	0.9	0.0	41.5	36.3	0.2	**22.8**
Avg. Stops (stops/veh)	1.39	0.41	0.39	0.99	0.50	0.20	0.03	1.33	0.80	0.21	0.00	1.26	0.78	0.07	**0.64**

Table 50. VISSIM Results Summary—Bryan District—SH 6 @ University.

Site Name: SH 6 @ University in Bryan District **Time Period: AM Peak Hour**															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	234	267	183	15	822	108	84	748	50	218	9	89	111	392	**3330**
Avg. Queue Length (ft)	33	33	26	85	85	85	77	77	77	77	20	20	20	0	**40**
Max. Queue Length (ft)	197	197	209	367	367	367	349	349	349	349	139	139	139	2	**393**
Avg. Delay (sec/veh)	99.7	30.3	2.1	100.1	40.3	32.0	56.8	56.9	29.0	27.6	60.6	50.1	26.7	1.3	**40.0**
Stopped Delay (sec/veh)	86.2	22.7	0.2	86.6	29.4	25.1	42.9	34.9	20.5	22.6	49.1	36.6	20.2	0.0	**28.8**
Avg. Stops (stops/veh)	1.57	0.70	0.05	1.75	0.86	0.70	1.48	1.56	0.60	0.64	1.77	1.48	0.59	0.00	**0.91**
Site Name: SH 6 @ University in Bryan District **Time Period: PM Peak Hour**															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	718	490	778	165	342	79	87	777	131	194	23	238	278	342	**4642**
Avg. Queue Length (ft)	106	106	105	82	82	82	182	182	182	182	74	74	74	7	**93**
Max. Queue Length (ft)	510	510	523	311	311	311	798	798	798	798	311	311	311	162	**798**
Avg. Delay (sec/veh)	74.0	28.1	16.8	99.3	66.7	46.4	109.0	72.0	46.1	43.8	67.9	56.4	40.1	1.3	**49.4**
Stopped Delay (sec/veh)	58.0	18.5	2.1	82.0	52.3	39.5	89.3	47.7	34.7	36.7	50.3	38.6	30.6	0.0	**35.0**
Avg. Stops (stops/veh)	1.26	0.68	0.43	1.83	1.23	0.82	1.73	1.56	0.78	0.79	1.70	1.44	0.74	0.00	**0.98**

Table 51. VISSIM Results Summary—Bryan District—US 290 @ SH 36.

Site Name: US 290 @ SH 36 in Bryan District Time Period: AM Peak Hour															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	373	177	130	139	326	91	124	104	94	159	81	134	77	40	**2047**
Avg. Queue Length (ft)	40	40	0	40	40	40	5	10	10	0	5	10	10	10	**13**
Max. Queue Length (ft)	166	166	31	162	162	162	68	68	68	49	72	72	72	72	**173**
Avg. Delay (sec/veh)	39.6	41.1	1.2	39.3	45.1	2.6	1.7	19.1	18.4	1.5	1.0	18.4	19.6	1.5	**24.8**
Stopped Delay (sec/veh)	24.4	23.5	0.0	25.2	26.1	0.6	0.3	15.0	12.4	0.2	0.1	14.2	13.5	0.1	**15.1**
Avg. Stops (stops/veh)	1.14	1.23	0.01	1.40	1.19	0.09	0.07	0.71	0.59	0.06	0.04	0.66	0.61	0.07	**0.74**
Site Name: US 290 @ SH 36 in Bryan District Time Period: PM Peak Hour															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	269	257	130	151	325	156	113	186	119	181	172	273	140	83	**2554**
Avg. Queue Length (ft)	43	43	0	43	43	43	8	15	15	1	10	20	20	20	**15**
Max. Queue Length (ft)	174	174	26	179	179	179	98	98	98	60	109	109	109	109	**187**
Avg. Delay (sec/veh)	44.9	46.9	1.3	42.3	47.1	3.1	1.3	20.6	19.8	2.0	1.1	21.9	20.8	2.1	**24.4**
Stopped Delay (sec/veh)	29.6	28.1	0.0	27.9	27.9	0.3	0.2	15.9	14.2	0.3	0.1	16.9	14.3	0.4	**15.6**
Avg. Stops (stops/veh)	1.23	1.23	0.02	1.31	1.12	0.08	0.07	0.71	0.60	0.10	0.03	0.72	0.62	0.11	**0.68**

Table 52. VISSIM Results Summary—Corpus Christi District—SH 358 @ Greenwood.

Site Name: SH 358 @ Greenwood in Corpus Christi District
Time Period: AM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	166	164	164	161	168	167	322	155	162	159	204	98	101	96	**2289**
Avg. Queue Length (ft)	12	12	3	13	13	7	14	28	28	28	6	11	11	1	**8**
Max. Queue Length (ft)	86	86	94	85	85	116	170	170	170	170	102	102	102	50	**170**
Avg. Delay (sec/veh)	17.5	15.7	1.9	18.6	16.7	1.9	13.6	30.0	15.3	14.1	14.5	28.5	14.4	3.4	**14.5**
Stopped Delay (sec/veh)	9.7	9.7	0.1	11.2	10.8	0.1	7.9	17.2	8.0	9.9	8.6	17.8	7.9	0.7	**8.4**
Avg. Stops (stops/veh)	0.72	0.59	0.07	0.70	0.60	0.08	0.69	1.41	0.52	0.61	0.76	1.34	0.50	0.18	**0.63**

Site Name: SH 358 @ Greenville in Corpus Christi District
Time Period: PM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	290	352	84	445	131	177	77	167	354	168	583	203	445	382	**3857**
Avg. Queue Length (ft)	20	20	13	15	15	6	11	23	23	23	12	22	22	19	**13**
Max. Queue Length (ft)	126	126	135	107	107	128	144	144	144	144	244	181	181	238	**287**
Avg. Delay (sec/veh)	18.3	16.1	2.4	20.9	13.9	2.4	0.2	27.2	14.7	14.2	3.7	29.3	15.2	11.0	**14.0**
Stopped Delay (sec/veh)	10.0	9.6	0.3	11.1	8.0	0.3	0.0	15.6	7.3	9.7	0.1	17.4	7.3	2.8	**7.0**
Avg. Stops (stops/veh)	0.80	0.62	0.14	0.99	0.53	0.14	0.00	1.34	0.50	0.63	0.05	1.40	0.51	0.66	**0.60**

Table 53. VISSIM Results Summary—Ft. Worth District—I-35W @ Alsbury.

Site Name: I-35W @ Alsbury in Ft. Worth District
Time Period: AM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	623	57	193	207	0	115	46	228	308	14	-*	190	163	200	**2344**
Avg. Queue Length (ft)	40	40	48	16	16	10	30	30	29	26	-	19	19	5	**25**
Max. Queue Length (ft)	234	234	253	117	117	122	139	139	138	148	-	94	94	102	**253**
Avg. Delay (sec/veh)	69.8	39.2	3.4	48.3	-	1.6	36.0	35.7	24.6	13.2	-	44.2	3.5	1.4	**35.6**
Stopped Delay (sec/veh)	51.1	26.1	1.0	32.7	-	0.1	24.1	22.8	17.7	9.8	-	28.4	2.5	0.1	**24.7**
Avg. Stops (stops/veh)	1.68	1.07	0.14	1.73	-	0.06	1.43	1.30	0.67	0.53	-	1.59	0.09	0.05	**1.03**

Site Name: I-35W @ Alsbury in Ft. Worth District
Time Period: PM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	455	0	197	293	0	152	8	473	356	21	80	367	402	562	**3365**
Avg. Queue Length (ft)	33	33	41	28	28	24	39	39	39	34	45	45	45	45	**37**
Max. Queue Length (ft)	176	176	195	158	158	169	178	178	177	186	217	217	217	231	**239**
Avg. Delay (sec/veh)	78.5	-	3.8	56.6	-	2.1	43.7	40.0	21.7	11.8	51.2	47.7	7.2	5.1	**32.1**
Stopped Delay (sec/veh)	60.1	-	1.0	40.3	-	0.2	29.7	24.7	15.2	8.5	34.5	29.4	3.9	1.2	**21.6**
Avg. Stops (stops/veh)	1.70	-	0.21	1.59	-	0.08	1.78	1.53	0.61	0.48	1.81	1.79	0.23	0.22	**0.98**

*U-turn volume data not available.

Table 54. VISSIM Results Summary—Ft. Worth District—I-35W @ FM 1187.

Site Name: I-35W @ FM 1187 in Ft. Worth District
Time Period: AM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	851	327	199	368	208	179	-*	214	505	396	166	212	81	429	**4134**
Avg. Queue Length (ft)	66	66	30	59	59	59	-	57	57	57	5	32	32	32	**42**
Max. Queue Length (ft)	448	448	473	286	286	286	-	240	240	240	112	126	126	126	**473**
Avg. Delay (sec/veh)	65.7	41.6	3.5	84.9	46.0	7.7	-	40.2	33.6	3.9	8.0	46.1	39.6	3.8	**37.6**
Stopped Delay (sec/veh)	41.6	21.4	0.7	64.0	30.7	4.2	-	29.1	25.6	0.4	3.4	33.8	31.8	1.2	**25.0**
Avg. Stops (stops/veh)	1.59	1.22	0.14	1.63	0.96	0.47	-	1.00	0.72	0.15	0.55	1.07	0.78	0.04	**0.90**

Site Name: I-35W @ FM 1187 in Ft. Worth District
Time Period: PM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	428	283	331	582	239	59	-*	364	204	413	95	247	281	753	**4277**
Avg. Queue Length (ft)	59	59	38	55	55	55	-	45	45	45	1	46	46	46	**41**
Max. Queue Length (ft)	346	346	371	339	339	339	-	194	194	194	52	170	170	170	**385**
Avg. Delay (sec/veh)	75.1	42.5	5.0	69.4	40.2	2.6	-	43.9	31.1	4.0	3.0	41.7	33.9	5.3	**33.7**
Stopped Delay (sec/veh)	55.0	25.6	1.4	48.3	23.3	0.9	-	31.2	23.6	0.5	0.7	30.5	25.9	1.4	**22.8**
Avg. Stops (stops/veh)	1.57	1.05	0.26	1.49	1.05	0.15	-	1.11	0.71	0.16	0.15	0.99	0.76	0.07	**0.78**

*U-turn volume data not available.

Table 55. VISSIM Results Summary—Ft. Worth District—I-20 @ Hulen.

Site Name: I-20 @ Hulen in Fort Worth District Time Period: AM Peak Hour															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	437	28	202	160	251	173	376	382	134	243	73	293	440	718	**3909**
Avg. Queue Length (ft)	47	47	0	34	34	1	25	47	47	0	29	58	58	7	**20**
Max. Queue Length (ft)	185	185	38	125	125	72	200	200	200	27	305	305	305	260	**317**
Avg. Delay (sec/veh)	42.2	40.4	1.7	43.7	43.6	2.4	9.5	37.4	35.6	1.3	2.7	36.7	37.3	4.3	**23.4**
Stopped Delay (sec/veh)	30.6	29.6	0.0	33.2	32.1	0.6	5.6	29.7	28.0	0.0	0.2	28.8	28.4	0.8	**17.0**
Avg. Stops (stops/veh)	0.87	0.80	0.02	0.81	0.84	0.17	0.34	0.79	0.73	0.01	0.07	0.83	0.79	0.17	**0.52**
Site Name: I-20 @ Hulen in Fort Worth District Time Period: PM Peak Hour															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	625	115	337	853	969	406	334	364	189	415	50	685	375	359	**6074**
Avg. Queue Length (ft)	94	94	10	177	177	6	39	75	75	11	67	134	134	3	**51**
Max. Queue Length (ft)	280	280	184	685	685	146	252	243	243	194	392	392	392	150	**685**
Avg. Delay (sec/veh)	63.9	55.9	7.4	52.7	48.7	13.5	4.3	64.0	58.1	7.6	6.9	60.6	53.3	7.3	**41.2**
Stopped Delay (sec/veh)	48.7	43.8	3.0	37.6	36.9	6.7	0.9	55.0	49.3	2.1	2.9	49.8	42.8	3.5	**31.2**
Avg. Stops (stops/veh)	0.95	0.86	0.38	0.91	0.90	0.60	0.22	0.91	0.84	0.39	0.22	0.96	0.89	0.28	**0.75**

Table 56. VISSIM Results Summary—Ft. Worth District—I-20 @ McCart.

Site Name: I-20 @ McCart in Ft. Worth District
Time Period: AM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	360	370	718	247	211	102	195	120	262	239	29	621	148	187	**3809**
Avg. Queue Length (ft)	89	89	11	79	79	3	27	51	51	66	85	85	85	85	**48**
Max. Queue Length (ft)	514	514	392	287	287	123	187	187	187	208	322	322	322	322	**517**
Avg. Delay (sec/veh)	47.9	44.7	7.4	50.7	46.9	18.3	4.1	46.6	50.3	6.1	45.4	42.8	38.4	3.4	**31.2**
Stopped Delay (sec/veh)	34.0	33.7	2.3	39.0	36.5	13.5	1.4	40.8	42.3	2.8	36.9	34.3	30.2	0.8	**23.4**
Avg. Stops (stops/veh)	0.97	0.88	0.29	0.90	0.86	0.51	0.30	0.85	0.89	0.41	0.97	0.91	0.84	0.14	**0.68**

Site Name: I-20 @ McCart in Ft. Worth District
Time Period: PM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	303	267	612	180	281	152	166	237	234	312	29	895	160	223	**4051**
Avg. Queue Length (ft)	100	100	19	78	78	2	36	72	72	88	131	131	131	131	**61**
Max. Queue Length (ft)	498	498	430	317	317	138	237	237	237	259	810	810	810	810	**830**
Avg. Delay (sec/veh)	70.0	61.2	7.6	46.6	45.6	21.5	3.7	57.6	63.8	10.7	45.1	41.8	37.5	6.0	**35.9**
Stopped Delay (sec/veh)	54.7	49.0	2.9	35.5	35.4	16.1	0.9	50.3	54.8	5.4	34.6	31.6	27.9	1.8	**27.4**
Avg. Stops (stops/veh)	1.18	1.05	0.32	0.88	0.82	0.55	0.23	0.93	0.99	0.65	1.03	0.95	0.94	0.26	**0.76**

Table 57. VISSIM Results Summary—Houston District—I-10 @ Bunker Hill.

Site Name: I-10 @ Bunker Hill in Houston District **Time Period: AM Peak Hour**															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	412	128	319	435	445	191	182	150	649	233	263	602	544	196	**4750**
Avg. Queue Length (ft)	325	325	325	982	982	982	30	58	58	58	49	91	91	91	**244**
Max. Queue Length (ft)	747	747	747	1561	1561	1561	226	226	226	226	325	325	325	325	**1561**
Avg. Delay (sec/veh)	87.5	82.8	141.7	192.3	190.8	190.9	5.9	33.5	34.4	12.6	10.8	45.0	41.4	2.1	**80.3**
Stopped Delay (sec/veh)	67.9	66.0	112.4	139.5	139.3	139.0	1.7	25.6	26.0	8.3	2.6	36.2	32.2	0.1	**60.0**
Avg. Stops (stops/veh)	1.35	1.34	2.04	2.90	2.95	2.99	0.37	0.66	0.71	0.42	0.56	0.83	0.77	0.04	**1.33**
Site Name: I-10 @ Bunker Hill in Houston District **Time Period: PM Peak Hour**															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	584	107	345	482	263	222	573	462	825	231	444	488	1705	247	**6977**
Avg. Queue Length (ft)	322	322	322	263	263	263	1005	895	895	895	116	168	168	168	**471**
Max. Queue Length (ft)	828	828	828	824	824	824	1672	1671	1671	1671	621	497	497	497	**1672**
Avg. Delay (sec/veh)	92.3	85.1	108.7	92.9	88.2	90.4	186.3	145.	55.1	21.2	29.4	47.2	47.6	4.4	**76.2**
Stopped Delay (sec/veh)	69.5	67.2	84.7	72.3	70.2	72.7	52.0	86.9	36.4	7.8	8.8	35.2	33.7	0.3	**47.0**
Avg. Stops (stops/veh)	1.44	1.40	1.63	1.46	1.42	1.45	4.49	2.73	0.97	0.47	1.07	0.87	0.85	0.11	**1.44**

Table 58. VISSIM Results Summary—Houston District—I-10 @ Gessner.

Site Name: I-10 @ Gessner in Houston District
Time Period: AM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	405	417	275	839	555	384	321	457	1000	287	184	816	511	360	**6810**
Avg. Queue Length (ft)	58	58	58	93	93	93	51	97	97	97	198	260	260	260	**108**
Max. Queue Length (ft)	179	179	179	323	323	323	294	294	294	294	617	617	617	617	**617**
Avg. Delay (sec/veh)	44.3	41.3	26.9	45.1	35.6	14.1	6.8	41.7	44.2	7.0	6.9	92.1	56.3	2.7	**41.0**
Stopped Delay (sec/veh)	34.9	34.8	22.0	30.9	26.7	9.7	1.0	33.9	33.5	1.5	2.3	73.1	43.4	0.5	**31.0**
Avg. Stops (stops/veh)	0.82	0.80	0.63	0.83	0.72	0.31	0.42	0.79	0.80	0.08	0.19	1.34	0.94	0.14	**0.74**

Site Name: I-10 @ Gessner in Houston District
Time Period: PM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	912	480	276	674	630	307	269	720	644	415	224	618	725	366	**7261**
Avg. Queue Length (ft)	85	85	85	129	129	129	78	143	143	143	1642	1642	1642	1642	**609**
Max. Queue Length (ft)	288	288	288	379	379	379	455	455	455	455	1671	1669	1669	1669	**1671**
Avg. Delay (sec/veh)	44.1	33.1	23.6	67.9	58.6	19.9	13.4	66.3	51.4	8.5	138.1	395.8	367.7	108.7	**113.1**
Stopped Delay (sec/veh)	29.6	26.7	18.8	53.1	47.5	14.9	4.1	53.7	40.8	1.8	78.3	275.0	251.5	54.9	**78.7**
Avg. Stops (stops/veh)	0.78	0.67	0.53	1.01	0.94	0.41	1.11	0.97	0.82	0.11	1.83	5.67	5.20	1.80	**1.72**

Table 59. VISSIM Results Summary—Houston District—I-45 @ Rayford/ Sawdust.

Site Name: I-45 @ Rayford/ Sawdust in Houston District
Time Period: AM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	351	302	587	880	325	950	286	879	253	301	341	508	232	347	**6541**
Avg. Queue Length (ft)	70	70	63	109	109	110	872	1530	1530	1533	68	130	130	32	**355**
Max. Queue Length (ft)	455	455	495	745	745	801	1674	1674	1674	1674	382	381	381	400	**1674**
Avg. Delay (sec/veh)	42.4	38.3	19.4	40.9	29.3	18.5	116.8	294.1	231.4	142.0	11.4	91.6	80.6	6.6	**86.5**
Stopped Delay (sec/veh)	34.3	32.2	8.0	27.1	22.3	2.5	57.5	216.7	153.2	70.4	1.6	78.0	67.2	2.6	**58.6**
Avg. Stops (stops/veh)	0.82	0.75	0.72	0.87	0.62	0.46	2.22	3.61	3.18	2.28	0.54	1.26	1.18	0.25	**1.35**

Site Name: I-45 @ Rayford/ Sawdust in Houston District
Time Period: PM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	648	563	189	704	431	503	481	952	357	371	472	777	480	279	**7207**
Avg. Queue Length (ft)	508	508	405	120	120	103	1128	1324	1324	1333	687	830	830	751	**632**
Max. Queue Length (ft)	1081	1081	1111	359	359	415	1674	1674	1674	1674	1558	1233	1233	1270	**1674**
Avg. Delay (sec/veh)	150.9	65.9	25.3	67.0	47.4	16.6	82.9	234.9	196.5	112.3	101.5	173.6	161.9	78.6	**121.0**
Stopped Delay (sec/veh)	104.0	46.3	12.2	50.0	38.9	6.6	34.8	174.2	134.1	50.6	17.6	94.3	90.1	22.1	**73.6**
Avg. Stops (stops/veh)	2.15	1.07	0.75	1.16	0.81	0.67	1.60	2.78	2.57	2.06	2.27	2.70	2.44	1.61	**1.87**

Table 60. VISSIM Results Summary—Houston District—I-45 @ Research Forest.

Site Name: I-45 @ Research Forest in Houston District
Time Period: AM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	406	143	417	396	112	134	474	1112	362	86	308	378	362	703	**5394**
Avg. Queue Length (ft)	57	57	9	89	89	89	80	122	122	122	29	56	56	5	**56**
Max. Queue Length (ft)	206	206	188	258	258	260	605	561	561	561	215	215	215	108	**605**
Avg. Delay (sec/veh)	45.7	44.4	4.5	66.7	53.0	42.9	17.5	37.4	28.4	28.0	4.0	39.5	39.5	3.1	**29.7**
Stopped Delay (sec/veh)	34.9	33.3	0.7	49.5	39.2	35.5	3.8	26.0	20.4	21.6	0.4	32.9	30.7	0.0	**20.7**
Avg. Stops (stops/veh)	0.79	0.78	0.17	1.24	1.13	0.83	0.73	0.75	0.61	0.65	0.19	0.78	0.75	0.01	**0.61**

Site Name: I-45 @ Research Forest in Houston District
Time Period: PM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	1311	274	535	383	252	138	290	824	1004	56	512	396	416	398	**6787**
Avg. Queue Length (ft)	487	487	438	106	106	107	100	182	182	182	97	93	93	24	**166**
Max. Queue Length (ft)	1184	1184	1207	291	291	294	600	592	592	592	618	443	443	336	**1219**
Avg. Delay (sec/veh)	108.4	89.0	21.2	74.2	60.6	50.9	15.6	54.3	53.3	54.3	29.1	50.4	49.0	2.2	**57.6**
Stopped Delay (sec/veh)	69.4	53.0	3.7	55.9	45.6	42.2	5.3	42.1	39.8	42.4	9.3	42.8	39.6	0.0	**38.8**
Avg. Stops (stops/veh)	1.55	1.36	0.43	1.22	1.06	0.86	0.87	0.88	0.85	0.89	1.60	0.83	0.80	0.01	**1.01**

Table 61. VISSIM Results Summary—Laredo District—I 35 @ Mann.

Site Name: I 35 @ Mann in Laredo District
Time Period: AM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	246	66	33	139	56	163	258	91	464	151	134	202	181	69	**2253**
Avg. Queue Length (ft)	18	18	18	14	14	0	13	26	26	26	11	21	21	21	**12**
Max. Queue Length (ft)	143	143	153	164	164	40	158	158	158	158	142	142	142	142	**178**
Avg. Delay (sec/veh)	90.2	17.5	3.9	26.3	24.3	2.3	1.6	35.7	15.9	13.4	2.4	38.6	14.5	15.1	**23.8**
Stopped Delay (sec/veh)	74.2	11.1	2.1	17.8	16.9	0.6	0.0	23.5	9.2	9.7	0.6	25.9	8.5	10.6	**16.9**
Avg. Stops (stops/veh)	1.88	0.59	0.19	0.87	0.73	0.05	0.00	1.48	0.54	0.53	0.16	1.59	0.51	0.59	**0.72**

Site Name: I 35 @ Mann in Laredo District
Time Period: PM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	246	73	34	204	8	180	659	110	708	185	187	335	483	95	**3506**
Avg. Queue Length (ft)	1575	1575	1578	18	18	0	22	44	44	44	75	148	148	148	**421**
Max. Queue Length (ft)	1668	1668	1669	151	151	54	251	251	251	251	507	507	507	507	**1669**
Avg. Delay (sec/veh)	807.3	231.9	183.4	51.7	28.5	2.7	3.3	45.4	18.9	17.4	6.5	108.1	41.0	40.1	**90.7**
Stopped Delay (sec/veh)	668.3	161.9	124.7	39.3	19.7	0.8	0.1	31.9	11.3	12.6	3.1	80.9	27.7	29.5	**70.3**
Avg. Stops (stops/veh)	15.94	4.43	3.30	1.72	0.80	0.06	0.02	1.53	0.57	0.58	0.34	3.31	1.15	1.10	**2.07**

Table 62. VISSIM Results Summary—Pharr District—I-2 @ FM 2220.

Site Name: I-2 @ FM 2220 in Pharr District
Time Period: AM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	234	527	320	774	623	211	308	413	36	149	104	535	61	378	**4673**
Avg. Queue Length (ft)	26	26	3	40	40	41	10	20	20	2	14	27	27	0	**16**
Max. Queue Length (ft)	132	132	114	207	207	223	130	130	130	75	163	163	163	0	**223**
Avg. Delay (sec/veh)	35.9	18.4	3.9	42.4	30.4	8.8	1.6	33.3	17.3	3.7	1.4	24.7	16.7	2.5	**22.2**
Stopped Delay (sec/veh)	23.7	9.7	0.7	27.4	18.0	4.2	0.0	22.9	9.8	1.0	0.0	12.9	9.4	0.0	**13.2**
Avg. Stops (stops/veh)	1.80	0.55	0.24	1.70	1.09	0.41	0.02	1.22	0.54	0.26	0.01	1.12	0.56	0.01	**0.87**

Site Name: I-2 @ FM 2220 in Pharr District
Time Period: PM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	380	574	518	599	546	356	419	548	111	233	417	622	235	489	**6046**
Avg. Queue Length (ft)	32	32	7	36	36	36	17	33	33	3	18	35	35	0	**18**
Max. Queue Length (ft)	162	162	190	200	200	212	199	199	199	112	186	186	186	0	**226**
Avg. Delay (sec/veh)	43.1	20.3	5.2	37.1	27.9	8.4	3.1	38.2	19.1	4.1	2.5	23.7	17.9	3.7	**19.6**
Stopped Delay (sec/veh)	27.0	9.9	0.6	24.3	16.3	3.4	0.2	26.0	10.4	1.0	0.1	11.8	9.3	0.1	**11.0**
Avg. Stops (stops/veh)	2.13	0.60	0.25	1.47	1.01	0.43	0.11	1.37	0.60	0.27	0.03	0.97	0.56	0.03	**0.75**

Table 63. VISSIM Results Summary—Pharr District—I-2 @ SH 494.

Site Name: I-2 @ SH 494 in Pharr District
Time Period: AM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	264	655	418	592	391	123	175	309	217	152	385	580	105	167	**4533**
Avg. Queue Length (ft)	86	86	14	78	78	0	38	76	76	76	26	78	78	84	**42**
Max. Queue Length (ft)	335	335	235	274	274	37	238	238	238	238	349	349	349	364	**390**
Avg. Delay (sec/veh)	45.8	51.1	10.2	47.1	49.8	2.7	4.7	60.3	59.8	57.5	4.9	53.0	46.7	24.2	**39.8**
Stopped Delay (sec/veh)	33.6	36.1	3.6	34.3	34.9	0.3	0.0	51.1	47.6	48.9	0.5	43.2	35.6	18.2	**29.5**
Avg. Stops (stops/veh)	0.78	0.82	0.44	0.77	0.81	0.10	0.00	0.92	0.94	0.94	0.04	0.96	0.89	0.70	**0.69**

Site Name: I-2 @ SH 494 in Pharr District
Time Period: PM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	427	620	387	605	483	257	189	466	344	160	666	699	260	372	**5935**
Avg. Queue Length (ft)	98	98	14	95	95	3	55	110	110	110	42	116	116	129	**58**
Max. Queue Length (ft)	392	392	219	321	321	116	332	332	332	332	527	527	527	547	**548**
Avg. Delay (sec/veh)	49.0	54.5	12.0	52.4	56.1	5.9	5.0	67.3	66.2	65.8	9.4	50.7	47.4	32.8	**42.4**
Stopped Delay (sec/veh)	35.5	37.4	4.6	38.4	39.7	1.7	0.0	55.7	51.7	55.4	1.9	39.8	33.5	22.3	**30.6**
Avg. Stops (stops/veh)	0.83	0.90	0.53	0.83	0.86	0.29	0.01	1.03	1.05	1.03	0.15	0.94	1.00	0.95	**0.76**

Table 64. VISSIM Results Summary—San Angelo District—SH 306 @ US 67.

Site Name: SH 306 @ US 67 in San Angelo District
Time Period: AM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	98	241	315	87	156	11	-*	270	46	107	247	177	130	223	**2108**
Avg. Queue Length (ft)	15	15	0	9	9	0	-	15	15	0	10	20	20	0	**7**
Max. Queue Length (ft)	111	111	0	77	77	8	-	111	111	0	123	123	123	0	**131**
Avg. Delay (sec/veh)	36.5	23.2	0.8	41.5	23.5	0.9	-	21.9	18.2	0.5	1.3	29.2	20.2	0.6	**15.1**
Stopped Delay (sec/veh)	25.0	11.5	0.0	29.9	12.3	0.0	-	13.7	10.8	0.0	0.0	18.8	12.7	0.0	**9.0**
Avg. Stops (stops/veh)	1.31	0.88	0.00	1.42	0.89	0.00	-	0.92	0.62	0.00	0.00	1.03	0.63	0.00	**0.54**

Site Name: SH 306 @ US 67 in San Angelo District
Time Period: PM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	142	369	352	224	419	33	-*	451	79	193	161	224	130	248	**3025**
Avg. Queue Length (ft)	27	27	0	25	25	0	-	37	37	0	15	30	30	0	**13**
Max. Queue Length (ft)	171	171	0	147	147	7	-	191	191	0	154	154	154	0	**197**
Avg. Delay (sec/veh)	52.7	29.6	0.9	56.2	28.8	1.0	-	31.4	24.3	0.6	1.3	36.5	27.3	0.6	**23.7**
Stopped Delay (sec/veh)	39.7	16.9	0.0	42.5	17.2	0.0	-	21.2	15.8	0.0	0.0	25.8	19.0	0.0	**15.8**
Avg. Stops (stops/veh)	1.46	0.99	0.00	1.53	0.92	0.00	-	1.04	0.67	0.00	0.00	1.12	0.70	0.00	**0.72**

*U-turn volume data not available.

Table 65. VISSIM Results Summary—San Antonio District—I-410 @ Callaghan.

Site Name: I-410 @ Callaghan in San Antonio District
Time Period: AM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	117	410	354	384	330	442	146	791	527	140	284	471	224	104	**4726**
Avg. Queue Length (ft)	127	127	127	75	75	46	63	125	125	1	47	94	94	1	**49**
Max. Queue Length (ft)	430	430	430	273	273	327	403	403	403	71	289	289	289	61	**448**
Avg. Delay (sec/veh)	29.7	34.1	41.1	44.1	44.9	6.8	2.0	41.0	40.6	3.2	1.3	59.1	54.4	4.4	**34.3**
Stopped Delay (sec/veh)	23.2	27.2	33.1	34.0	34.6	1.9	0.2	31.5	29.9	1.0	0.0	48.8	43.4	1.5	**26.4**
Avg. Stops (stops/veh)	0.58	0.69	0.64	0.82	0.88	0.28	0.06	0.85	0.84	0.19	0.00	0.98	0.94	0.32	**0.67**

Site Name: I-410 @ Callaghan in San Antonio District
Time Period: PM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	314	260	289	286	714	790	392	601	383	78	108	636	627	185	**5663**
Avg. Queue Length (ft)	108	108	108	1528	1528	1557	201	378	378	4	63	123	123	2	**414**
Max. Queue Length (ft)	383	383	383	1670	1670	1673	779	778	778	131	415	415	415	92	**1674**
Avg. Delay (sec/veh)	39.6	41.3	53.0	101.3	112.3	242.3	39.4	146.9	115.2	14.0	1.2	43.7	43.0	4.3	**96.0**
Stopped Delay (sec/veh)	29.9	32.2	42.2	71.0	73.2	97.4	24.2	124.8	95.0	7.5	0.0	34.0	31.8	1.4	**60.5**
Avg. Stops (stops/veh)	0.78	0.81	0.95	2.66	3.73	16.63	1.38	2.14	1.82	0.61	0.00	0.88	0.84	0.26	**3.71**

Table 66. VISSIM Results Summary—San Antonio District—I-410 @ Ingram.

Site Name: I-410 @ Ingram in San Antonio District
Time Period: AM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	600	362	333	508	230	366	150	184	261	167	45	246	175	109	**3735**
Avg. Queue Length (ft)	73	73	3	60	60	4	13	25	25	2	12	23	24	0	**20**
Max. Queue Length (ft)	256	256	122	259	259	157	133	133	133	77	119	119	119	46	**285**
Avg. Delay (sec/veh)	41.7	40.7	3.2	44.6	43.6	4.9	0.9	23.2	23.2	3.3	0.6	23.2	23.5	1.9	**25.8**
Stopped Delay (sec/veh)	25.1	24.6	0.8	27.1	26.5	1.0	0.0	18.0	16.3	1.1	0.0	18.6	16.9	0.4	**16.0**
Avg. Stops (stops/veh)	1.17	1.21	0.18	1.53	1.37	0.25	0.00	0.63	0.59	0.25	0.00	0.63	0.59	0.12	**0.79**

Site Name: I-410 @ Ingram in San Antonio District
Time Period: PM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	560	621	438	354	558	215	201	496	498	241	241	285	708	323	**5738**
Avg. Queue Length (ft)	154	154	10	141	141	3	22	45	45	6	22	45	45	10	**41**
Max. Queue Length (ft)	633	633	230	400	400	142	233	233	233	127	243	243	243	161	**633**
Avg. Delay (sec/veh)	70.5	70.6	6.5	65.4	70.3	9.7	1.2	27.3	19.9	5.9	1.3	27.2	19.6	7.6	**35.0**
Stopped Delay (sec/veh)	48.5	48.9	1.7	44.0	47.0	4.2	0.0	19.6	12.8	2.9	0.0	17.9	12.6	3.2	**23.2**
Avg. Stops (stops/veh)	2.19	2.16	0.40	2.04	2.22	0.41	0.01	0.73	0.57	0.31	0.00	1.00	0.56	0.42	**1.10**

Table 67. VISSIM Results Summary—Waco District—I-35 @ FM 286.

Site Name: I-35 @ FM 286 in Waco District
Time Period: AM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	52	88	121	43	40	18	-*	59	12	26	2	28	26	44	**557**
Avg. Queue Length (ft)	3	3	0	2	2	0	-	2	2	0	1	1	1	0	**1**
Max. Queue Length (ft)	70	70	3	46	46	0	-	46	46	8	38	38	38	11	**70**
Avg. Delay (sec/veh)	16.2	18.8	1.2	15.7	18.6	0.6	-	15.6	10.9	0.7	0.5	15.5	10.5	0.7	**10.5**
Stopped Delay (sec/veh)	0.6	0.6	0.0	0.7	0.7	0.0	-	0.7	0.5	0.0	0.0	0.7	0.2	0.0	**0.4**
Avg. Stops (stops/veh)	2.21	2.20	0.00	2.22	2.24	0.00	-	2.26	1.19	0.00	0.00	2.24	1.05	0.00	**1.31**

Site Name: I-35 @ FM 286 in Waco District
Time Period: PM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	53	93	83	74	68	23	104	93	32	47	-*	46	101	51	**868**
Avg. Queue Length (ft)	4	4	0	4	4	0	2	3	3	0	-	3	3	0	**1**
Max. Queue Length (ft)	61	61	19	68	68	3	58	58	58	14	-	52	52	17	**76**
Avg. Delay (sec/veh)	16.9	19.5	1.3	17.5	19.8	0.7	0.7	16.7	11.3	0.7	-	16.4	11.2	0.8	**10.9**
Stopped Delay (sec/veh)	0.7	0.7	0.0	1.1	0.8	0.0	0.0	0.8	0.4	0.0	-	0.7	0.3	0.0	**0.5**
Avg. Stops (stops/veh)	2.30	2.31	0.00	2.38	2.34	0.00	0.00	2.38	1.14	0.00	-	2.33	1.15	0.00	**1.33**

*U-turn volume data not available.

Table 68. VISSIM Results Summary—Wichita Falls District—US 82 @ Kemp.

Site Name: US 82 @ Kemp in Wichita Falls District Time Period: AM Peak Hour															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	96	333	342	75	363	85	95	168	122	261	128	505	70	114	**2757**
Avg. Queue Length (ft)	15	15	0	12	12	0	9	17	17	0	16	33	33	0	**8**
Max. Queue Length (ft)	140	140	0	105	105	5	116	116	116	48	189	189	189	33	**189**
Avg. Delay (sec/veh)	70.3	17.9	1.5	63.5	16.8	0.9	1.9	37.9	22.6	1.1	1.4	40.8	24.9	1.5	**20.5**
Stopped Delay (sec/veh)	58.9	11.8	0.1	52.5	11.2	0.0	0.7	25.2	15.9	0.0	0.2	25.5	17.7	0.0	**13.8**
Avg. Stops (stops/veh)	1.50	0.58	0.01	1.36	0.53	0.01	0.06	1.35	0.60	0.02	0.03	1.44	0.66	0.03	**0.63**
Site Name: US 82 @ Kemp in Wichita Falls District Time Period: PM Peak Hour															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	263	574	211	100	374	200	184	185	135	285	167	616	131	197	**3621**
Avg. Queue Length (ft)	34	34	0	19	19	0	13	25	25	0	28	56	56	1	**14**
Max. Queue Length (ft)	207	207	34	167	167	28	165	165	165	50	246	246	246	52	**257**
Avg. Delay (sec/veh)	91.2	22.3	3.3	76.5	22.7	1.6	3.9	48.2	29.9	1.5	3.5	51.0	31.5	2.3	**28.9**
Stopped Delay (sec/veh)	77.0	15.0	1.1	64.3	16.4	0.2	1.6	33.2	22.4	0.1	1.2	33.3	23.3	0.2	**20.7**
Avg. Stops (stops/veh)	1.66	0.64	0.11	1.50	0.62	0.03	0.11	1.60	0.66	0.04	0.10	1.66	0.72	0.08	**0.77**

Table 69. VISSIM Results Summary—Wichita Falls District—US 82 @ Lawrence.

Site Name: US 82 @ Lawrence in Wichita Falls District
Time Period: AM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	97	55	243	70	83	25	52	28	134	138	141	311	125	54	**1556**
Avg. Queue Length (ft)	6	6	0	11	11	0	4	9	9	0	21	42	42	0	**7**
Max. Queue Length (ft)	87	87	30	92	92	5	82	82	82	45	262	262	262	11	**262**
Avg. Delay (sec/veh)	41.9	15.1	1.4	57.5	28.0	0.9	0.1	29.9	20.8	1.5	1.0	35.1	20.5	0.8	**18.7**
Stopped Delay (sec/veh)	32.1	9.7	0.0	46.0	20.6	0.0	0.0	19.4	13.9	0.1	0.0	20.1	13.6	0.0	**12.2**
Avg. Stops (stops/veh)	1.21	0.60	0.01	1.64	0.82	0.00	0.00	1.41	0.60	0.05	0.00	1.39	0.59	0.01	**0.62**

Site Name: US 82 @ Lawrence in Wichita Falls District
Time Period: PM Peak Hour

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	390	155	416	58	138	12	180	73	299	294	194	638	268	67	**3182**
Avg. Queue Length (ft)	41	41	0	19	19	0	9	17	17	2	166	333	333	0	**41**
Max. Queue Length (ft)	251	251	101	118	118	14	117	117	117	114	970	970	970	38	**970**
Avg. Delay (sec/veh)	70.3	22.0	2.6	75.9	35.2	3.1	0.3	36.3	19.6	3.1	23.5	90.1	24.1	4.1	**37.4**
Stopped Delay (sec/veh)	56.3	15.2	0.2	64.3	27.7	1.2	0.0	25.3	13.0	0.7	14.8	60.2	16.1	1.7	**26.1**
Avg. Stops (stops/veh)	1.50	0.67	0.04	1.67	0.89	0.14	0.00	1.42	0.57	0.16	0.70	2.92	0.69	0.11	**1.08**

APPENDIX D. SIMULATION RESULTS FROM THE COUNTERMEASURES

Approach: Extend Turn Bays

Simulation Results for I-10 at Gessner Site

Table 70. VISSIM Countermeasures Results—Extending Left-Turn and U-Turn Bays Performance Measures of AM Peak Hour at I-10 @ Gessner Rd.

Base Condition															
Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	405	417	275	839	555	384	321	457	1000	287	184	816	511	360	6810
Avg. Queue Length (ft)	58	58	58	93	93	93	51	97	97	97	198	260	260	260	108
Max. Queue Length (ft)	179	179	179	323	323	323	294	294	294	294	617	617	617	617	617
Avg. Delay (sec/veh)	44.3	41.3	26.9	45.1	35.6	14.1	6.8	41.7	44.2	7.0	6.9	92.1	56.3	2.7	41.0
Stopped Delay (sec/veh)	34.9	34.8	22.0	30.9	26.7	9.7	1.0	33.9	33.5	1.5	2.3	73.1	43.4	0.5	31.0
Avg. Stops (stops/veh)	0.82	0.80	0.63	0.83	0.72	0.31	0.42	0.79	0.80	0.08	0.19	1.34	0.94	0.14	0.74
Extending Left-Turn and U-Turn Bays by 100 ft in both Westbound and Eastbound															
Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	405	417	275	839	555	384	321	457	1000	286	184	812	511	360	6807
Avg. Queue Length (ft)	58	58	58	93	93	93	51	97	97	97	192	253	253	253	106
Max. Queue Length (ft)	177	177	177	315	315	315	293	293	293	293	619	619	619	619	619
Avg. Delay (sec/veh)	44.3	41.4	26.9	44.9	35.4	14.0	6.6	41.5	44.2	8.1	3.8	88.4	55.7	2.7	40.4
Stopped Delay (sec/veh)	34.9	34.8	22.0	30.9	26.6	9.7	0.9	33.7	33.4	1.4	0.7	70.3	43.0	0.5	30.6
Avg. Stops (stops/veh)	0.82	0.80	0.64	0.82	0.72	0.30	0.39	0.79	0.80	0.09	0.11	1.30	0.94	0.14	0.73

Table 71. VISSIM Countermeasures Results– Extending Left-Turn and U-Turn Bays Performance Measures of PM Peak Hour at I-10 @ Gessner Rd.

Base Condition															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	912	480	276	674	630	307	269	720	644	415	224	618	725	366	**7261**
Avg. Queue Length (ft)	85	85	85	129	129	129	78	143	143	143	1642	1642	1642	1642	**609**
Max. Queue Length (ft)	288	288	288	379	379	379	455	455	455	455	1671	1669	1669	1669	**1671**
Avg. Delay (sec/veh)	44.1	33.1	23.6	67.9	58.6	19.9	13.4	66.3	51.4	8.5	138.1	395.8	367.7	108.7	**113.1**
Stopped Delay (sec/veh)	29.6	26.7	18.8	53.1	47.5	14.9	4.1	53.7	40.8	1.8	78.3	275.0	251.5	54.9	**78.7**
Avg. Stops (stops/veh)	0.78	0.67	0.53	1.01	0.94	0.41	1.11	0.97	0.82	0.11	1.83	5.67	5.20	1.80	**1.72**
Extending Left-Turn and U-Turn Bays by 100 ft in both Westbound and Eastbound															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	912	480	276	675	630	307	269	719	645	415	229	620	724	366	**7267**
Avg. Queue Length (ft)	85	85	85	136	136	136	79	144	144	144	1641	1641	1641	1641	**610**
Max. Queue Length (ft)	289	289	289	436	436	436	453	453	453	453	1671	1669	1669	1669	**1671**
Avg. Delay (sec/veh)	44.0	33.3	23.9	70.7	61.6	20.6	13.2	66.8	51.6	9.7	110.1	402.1	364.7	83.1	**111.9**
Stopped Delay (sec/veh)	29.6	26.9	19.3	55.4	49.9	15.4	4.0	54.2	41.0	1.8	60.9	282.0	248.8	39.9	**78.3**
Avg. Stops (stops/veh)	0.78	0.68	0.48	1.04	0.98	0.43	1.09	0.98	0.82	0.10	1.51	5.67	5.25	1.55	**1.71**

Simulation Results for I-20 at Hulen Site

Table 72. VISSIM Countermeasures Results Summary: I-20 @ Hulen AM Peak Hour—Extend U-Turn Bay.

Base Condition															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	437	28	202	160	251	173	376	382	134	243	73	293	440	718	**3909**
Avg. Queue Length (ft)	47	47	0	34	34	1	25	47	47	0	29	58	58	7	**20**
Max. Queue Length (ft)	185	185	38	125	125	72	200	200	200	27	305	305	305	260	**317**
Avg. Delay (sec/veh)	42.2	40.4	1.7	43.7	43.6	2.4	9.5	37.4	35.6	1.3	2.7	36.7	37.3	4.3	**23.4**
Stopped Delay (sec/veh)	30.6	29.6	0.0	33.2	32.1	0.6	5.6	29.7	28.0	0.0	0.2	28.8	28.4	0.8	**17.0**
Avg. Stops (stops/veh)	0.87	0.80	0.02	0.81	0.84	0.17	0.34	0.79	0.73	0.01	0.07	0.83	0.79	0.17	**0.52**
Extend U-Turn Bay															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	438	28	202	158	250	172	376	383	135	243	73	292	438	717	**3905**
Avg. Queue Length (ft)	48	48	0	33	33	1	24	46	46	0	29	58	58	6	**19**
Max. Queue Length (ft)	176	176	40	117	117	65	206	206	206	34	306	306	306	230	**319**
Avg. Delay (sec/veh)	42.9	40.5	1.7	41.4	42.7	2.2	9.3	36.6	35.3	1.3	2.5	37.3	37.1	4.0	**23.2**
Stopped Delay (sec/veh)	31.3	29.8	0.0	31.1	31.4	0.5	5.5	28.9	27.7	0.0	0.2	29.5	28.4	0.7	**16.9**
Avg. Stops (stops/veh)	0.87	0.79	0.02	0.80	0.82	0.14	0.34	0.78	0.73	0.02	0.05	0.82	0.78	0.15	**0.51**

Table 73. VISSIM Countermeasures Results Summary: I-20 @ Hulen PM Peak Hour—Extend U-Turn Bay.

Base Condition*

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	625	115	337	853	969	406	334	364	189	415	50	685	375	359	**6074**
Avg. Queue Length (ft)	94	94	10	177	177	6	39	75	75	11	67	134	134	3	**51**
Max. Queue Length (ft)	280	280	184	685	685	146	252	243	243	194	392	392	392	150	**685**
Avg. Delay (sec/veh)	63.9	55.9	7.4	52.7	48.7	13.5	4.3	64.0	58.1	7.6	6.9	60.6	53.3	7.3	**41.2**
Stopped Delay (sec/veh)	48.7	43.8	3.0	37.6	36.9	6.7	0.9	55.0	49.3	2.1	2.9	49.8	42.8	3.5	**31.2**
Avg. Stops (stops/veh)	0.95	0.86	0.38	0.91	0.90	0.60	0.22	0.91	0.84	0.39	0.22	0.96	0.89	0.28	**0.75**

Extend U-Turn Bay

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	623	114	337	854	971	406	334	363	189	414	49	686	375	359	**6074**
Avg. Queue Length (ft)	94	94	13	178	178	7	39	76	76	10	67	133	133	5	**52**
Max. Queue Length (ft)	264	264	254	670	670	177	250	242	242	187	380	380	380	167	**670**
Avg. Delay (sec/veh)	64.1	57.7	7.5	53.6	48.7	14.1	4.1	64.8	56.8	7.5	6.6	59.9	53.5	7.5	**41.3**
Stopped Delay (sec/veh)	48.9	45.5	3.0	38.4	36.8	7.1	0.8	55.8	48.1	2.1	3.1	49.0	42.9	3.6	**31.3**
Avg. Stops (stops/veh)	0.96	0.87	0.38	0.92	0.90	0.63	0.22	0.92	0.84	0.38	0.19	0.97	0.90	0.30	**0.76**

Simulation Results for I-410 at Ingram Site

Table 74. VISSIM Countermeasures Results Summary: I-410 @ Ingram AM Peak Hour—Extend U-Turn Bay.

Base Condition*

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	603	363	331	120	351	443	151	189	321	167	139	244	174	116	**3713**
Avg. Queue Length (ft)	76	76	0	50	50	7	25	51	51	2	22	43	43	1	**25**
Max. Queue Length (ft)	294	294	21	208	208	183	180	180	180	82	166	166	167	49	**294**
Avg. Delay (sec/veh)	38.3	36.3	1.9	40.9	44.0	4.7	1.4	40.4	41.7	3.1	1.8	41.2	42.2	2.0	**26.7**
Stopped Delay (sec/veh)	26.2	25.9	0.0	30.3	32.6	1.2	0.0	33.1	32.4	0.9	0.1	35.1	32.6	0.4	**19.4**
Avg. Stops (stops/veh)	0.76	0.75	0.01	0.78	0.92	0.28	0.02	0.91	0.81	0.23	0.06	0.86	0.86	0.10	**0.58**

Extend U-Turn Bay

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	605	363	331	119	347	443	151	189	321	167	139	245	176	116	**3712**
Avg. Queue Length (ft)	77	77	0	52	52	8	26	52	52	2	22	43	43	1	**25**
Max. Queue Length (ft)	294	294	44	204	204	197	181	181	181	79	170	170	172	49	**294**
Avg. Delay (sec/veh)	38.7	36.6	1.9	42.1	45.6	4.8	2.0	42.3	42.1	3.4	3.2	41.8	42.1	2.0	**27.2**
Stopped Delay (sec/veh)	26.4	26.2	0.0	31.4	33.9	1.3	0.0	35.0	32.6	1.1	0.1	35.6	32.5	0.4	**19.7**
Avg. Stops (stops/veh)	0.78	0.75	0.01	0.81	0.95	0.29	0.04	0.92	0.83	0.26	0.06	0.87	0.88	0.13	**0.59**

Table 75. VISSIM Countermeasures Results Summary: I-410 @ Ingram PM Peak Hour—Extend U-Turn Bay.

Base Condition*															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	564	621	442	329	486	440	353	483	495	247	330	285	716	321	**6112**
Avg. Queue Length (ft)	227	227	5	144	144	23	78	155	155	9	110	220	220	21	**81**
Max. Queue Length (ft)	907	907	207	654	654	395	461	461	461	151	739	739	739	469	**944**
Avg. Delay (sec/veh)	75.8	73.3	7.1	66.2	65.3	11.5	3.3	68.4	68.5	11.7	2.6	62.9	82.0	25.2	**50.2**
Stopped Delay (sec/veh)	60.1	60.1	2.3	52.5	53.1	5.4	0.4	58.3	57.4	5.9	0.3	53.7	67.8	14.7	**40.1**
Avg. Stops (stops/veh)	1.11	1.09	0.28	0.95	0.97	0.62	0.25	0.93	0.93	0.62	0.04	0.91	1.05	0.90	**0.81**

Extend U-Turn Bay															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	565	622	441	327	483	441	353	491	496	247	330	284	716	321	**6116**
Avg. Queue Length (ft)	230	230	5	149	149	26	77	152	152	10	103	205	205	18	**80**
Max. Queue Length (ft)	919	919	175	683	683	466	467	467	467	178	695	695	695	332	**960**
Avg. Delay (sec/veh)	77.2	74.7	7.0	66.5	66.4	12.3	3.0	67.6	67.7	12.0	4.1	62.4	78.7	21.6	**50.0**
Stopped Delay (sec/veh)	61.6	61.5	2.3	52.8	54.2	5.8	0.3	57.6	56.9	6.0	0.4	53.3	64.7	11.9	**39.9**
Avg. Stops (stops/veh)	1.11	1.09	0.28	0.96	0.99	0.65	0.24	0.91	0.92	0.63	0.11	0.88	1.04	0.83	**0.81**

Table 76. VISSIM Countermeasures Results Summary: I-410 @ Ingram AM Peak Hour—Extend U-Turn Bay (Increased Travel Demand).

Base Condition															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	605	362	331	120	350	443	191	236	399	212	169	304	223	148	**4092**
Avg. Queue Length (ft)	82	82	0	55	55	8	33	66	66	3	28	56	55	1	**30**
Max. Queue Length (ft)	300	300	47	204	204	194	213	213	213	101	189	189	193	70	**300**
Avg. Delay (sec/veh)	40.8	38.0	1.9	44.3	47.5	5.1	1.6	45.1	44.2	3.8	1.94	44.9	46.3	2.5	**28.7**
Stopped Delay (sec/veh)	28.3	27.3	0	33.7	36.0	1.4	0	37.1	34.5	1.2	0.16	38.1	35.9	0.6	**21.2**
Avg. Stops (stops/veh)	0.80	0.76	0.01	0.80	0.94	0.29	0.03	0.92	0.83	0.26	0.07	0.89	0.90	0.15	**0.59**
Extend U-Turn Bay															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	603	361	331	120	349	443	191	234	396	212	169	306	223	148	**4086**
Avg. Queue Length (ft)	81	81	0	53	53	8	34	67	67	3	27	54	54	1	**29**
Max. Queue Length (ft)	292	292	49	212	212	215	229	229	229	109	195	195	191	57	**297**
Avg. Delay (sec/veh)	40.3	38.2	1.9	42.6	46.0	5.0	2.1	44.0	46.0	4.0	3.5	44.2	44.3	2.6	**28.5**
Stopped Delay (sec/veh)	27.8	27.4	0	32.0	34.6	1.4	0	35.9	36.1	1.3	0.2	37.5	34.2	0.6	**20.9**
Avg. Stops (stops/veh)	0.80	0.76	0.02	0.78	0.91	0.30	0.04	0.93	0.85	0.28	0.11	0.88	0.88	0.17	**0.60**

Table 77. VISSIM Countermeasures Results Summary: I-410 @ Ingram PM Peak Hour—Extend U-Turn Bay (Increased Travel Demand).

Base Condition															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	569	626	441	327	483	440	406	572	585	292	364	318	803	353	**6579**
Avg. Queue Length (ft)	236	236	6	150	150	30	473	934	934	557	666	1330	1330	1168	**443**
Max. Queue Length (ft)	959	959	205	673	673	427	1497	1497	1497	1286	1666	1666	1666	1674	**1674**
Avg. Delay (sec/veh)	77.7	76.3	8.6	66.8	67.0	14.0	76.6	198.2	202.8	133.7	80.8	198.9	275.9	207.1	**128.6**
Stopped Delay (sec/veh)	62.0	62.8	3.3	53.2	54.5	7.1	55.3	169.2	172.5	103.0	50.0	156.0	225.0	157.9	**103.0**
Avg. Stops (stops/veh)	1.11	1.11	0.34	0.97	1.01	0.75	1.81	2.47	2.55	2.80	1.85	3.35	4.13	4.22	**2.07**
Extend U-Turn Bay															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	568	625	442	327	481	441	418	578	591	292	386	324	822	370	**6662**
Avg. Queue Length (ft)	232	232	8	148	148	30	368	724	724	484	630	1258	1258	1083	**397**
Max. Queue Length (ft)	912	912	220	664	664	438	1368	1368	1368	1325	1671	1671	1671	1670	**1671**
Avg. Delay (sec/veh)	77.1	75.4	8.5	67.2	66.0	13.6	50.8	194.9	196.3	121.4	34.1	170.8	278.8	217.1	**122.8**
Stopped Delay (sec/veh)	61.4	61.9	3.2	53.4	53.5	6.9	35.8	167.3	168.4	92.8	16.5	140.0	233.4	171.4	**100.1**
Avg. Stops (stops/veh)	1.11	1.11	0.36	0.96	0.95	0.70	1.21	2.47	2.45	2.64	0.66	2.23	3.64	4.15	**1.84**

U-Turn Lane: Dual U-Turn Lane

Simulation Results for I-410 at Ingram Site

Table 78. VISSIM Countermeasures Results Summary: I-410 @ Ingram AM Peak Hour—Dual U-Turn Lane.

Base Condition															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	603	363	331	120	351	443	151	189	321	167	139	244	174	116	**3713**
Avg. Queue Length (ft)	76	76	0	50	50	7	25	51	51	2	22	43	43	1	**25**
Max. Queue Length (ft)	294	294	21	208	208	183	180	180	180	82	166	166	167	49	**294**
Avg. Delay (sec/veh)	38.3	36.3	1.9	40.9	44.0	4.7	1.4	40.4	41.7	3.1	1.8	41.2	42.2	2.0	**26.7**
Stopped Delay (sec/veh)	26.2	25.9	0.0	30.3	32.6	1.2	0.0	33.1	32.4	0.9	0.1	35.1	32.6	0.4	**19.4**
Avg. Stops (stops/veh)	0.76	0.75	0.01	0.78	0.92	0.28	0.02	0.91	0.81	0.23	0.06	0.86	0.86	0.10	**0.58**
Dual U-Turn Lane															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	603	363	331	120	351	443	151	189	321	167	139	243	174	116	**3713**
Avg. Queue Length (ft)	77	77	0	50	50	8	26	51	51	2	22	43	43	1	**25**
Max. Queue Length (ft)	294	294	21	208	208	182	181	181	181	79	166	166	167	49	**294**
Avg. Delay (sec/veh)	38.4	36.3	1.8	40.1	43.9	4.8	0.9	40.7	42.0	3.1	1.3	41.1	41.8	2.0	**26.6**
Stopped Delay (sec/veh)	26.3	25.8	0.0	29.7	32.5	1.3	0.0	33.4	32.6	0.9	0.1	35.1	32.2	0.4	**19.4**
Avg. Stops (stops/veh)	0.76	0.76	0.01	0.78	0.92	0.30	0.00	0.91	0.82	0.23	0.02	0.85	0.85	0.13	**0.58**

Table 79. VISSIM Countermeasures Results Summary: I-410 @ Ingram PM Peak Hour—Dual U-Turn Lane.

Base Condition															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	564	621	442	329	486	440	353	483	495	247	330	285	716	321	**6112**
Avg. Queue Length (ft)	227	227	5	144	144	23	78	155	155	9	110	220	220	21	**81**
Max. Queue Length (ft)	907	907	207	654	654	395	461	461	461	151	739	739	739	469	**944**
Avg. Delay (sec/veh)	75.8	73.3	7.1	66.2	65.3	11.5	3.3	68.4	68.5	11.7	2.6	62.9	82.0	25.2	**50.2**
Stopped Delay (sec/veh)	60.1	60.1	2.3	52.5	53.1	5.4	0.4	58.3	57.4	5.9	0.3	53.7	67.8	14.7	**40.1**
Avg. Stops (stops/veh)	1.11	1.09	0.28	0.95	0.97	0.62	0.25	0.93	0.93	0.62	0.04	0.91	1.05	0.90	**0.81**
Dual U-Turn Lane															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	563	621	442	330	487	440	353	484	494	247	330	286	721	323	**6121**
Avg. Queue Length (ft)	223	223	5	142	142	23	78	155	155	9	104	208	208	19	**79**
Max. Queue Length (ft)	847	847	206	662	662	395	453	453	453	156	694	694	694	382	**887**
Avg. Delay (sec/veh)	75.5	73.2	6.9	65.9	64.8	11.6	1.7	68.9	68.4	11.7	1.9	62.5	78.8	23.2	**49.5**
Stopped Delay (sec/veh)	59.9	60.1	2.2	52.3	52.7	5.3	0.1	58.7	57.4	6.1	0.1	53.4	65.2	13.1	**39.6**
Avg. Stops (stops/veh)	1.10	1.10	0.28	0.95	0.97	0.66	0.04	0.93	0.92	0.61	0.02	0.89	1.03	0.89	**0.80**

Table 80. U-Turn Departure Side Results: I-410 @ Ingram Base Scenario and Dual U-Turn Lane Improvement.

Measure of Effectiveness	**AM Peak Hour**		**PM Peak Hour**	
	Base	**Improvement**	**Base**	**Improvement**
Southbound U-Turn Departure End				
Number of Vehicles	139	139	330	330
Avg. Queue Length (ft)	0.41	0.18	0.4	0.28
Max. Queue Length (ft)	52	37	65	36
Avg. Queue Stops (stops)	13	3	9	4
Northbound U-Turn Departure End				
Number of Vehicles	151	151	353	353
Avg. Queue Length (ft)	0.14	0.02	2.39	0.29
Max. Queue Length (ft)	41	13	142	35
Avg. Queue Stops (stops)	7	1	42	6

U-Turn Lane: Add U-Turn Lane for Sites without One

Simulation Results for I-20 at McCart Site

Table 81. VISSIM Countermeasures Results Summary: I-20 @ McCart AM Peak Hour—Add U-Turn (Westbound).

Base Condition*															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	360	370	718	247	211	102	195	120	262	239	29	621	148	187	**3809**
Avg. Queue Length (ft)	89	89	11	79	79	3	27	51	51	66	85	85	85	85	**48**
Max. Queue Length (ft)	514	514	392	287	287	123	187	187	187	208	322	322	322	322	**517**
Avg. Delay (sec/veh)	47.9	44.7	7.4	50.7	46.9	18.3	4.1	46.6	50.3	6.1	45.4	42.8	38.4	3.4	**31.2**
Stopped Delay (sec/veh)	34.0	33.7	2.3	39.0	36.5	13.5	1.4	40.8	42.3	2.8	36.9	34.3	30.2	0.8	**23.4**
Avg. Stops (stops/veh)	0.97	0.88	0.29	0.90	0.86	0.51	0.30	0.85	0.89	0.41	0.97	0.91	0.84	0.14	**0.68**
Add U-Turn (Westbound)															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	359	371	718	246	209	102	195	117	258	239	30	633	150	187	**3812**
Avg. Queue Length (ft)	89	89	11	78	78	2	27	51	51	66	40	80	80	80	**42**
Max. Queue Length (ft)	522	522	370	292	292	95	202	202	202	224	283	283	283	283	**523**
Avg. Delay (sec/veh)	48.1	43.6	7.2	49.9	47.4	17.8	4.1	47.8	49.6	5.7	6.2	42.3	39.4	3.7	**30.6**
Stopped Delay (sec/veh)	34.4	32.6	2.1	38.1	36.9	13.2	1.4	42.0	41.7	2.6	0.6	33.9	31.0	0.9	**22.9**
Avg. Stops (stops/veh)	0.96	0.88	0.31	0.90	0.86	0.47	0.31	0.85	0.89	0.38	0.19	0.91	0.85	0.17	**0.67**

Table 82. VISSIM Countermeasures Results Summary: I-20 @ McCart PM Peak Hour—Add U-Turn (Westbound).

Base Condition*

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	303	267	612	180	281	152	166	237	234	312	29	895	160	223	**4051**
Avg. Queue Length (ft)	100	100	19	78	78	2	36	72	72	88	131	131	131	131	**61**
Max. Queue Length (ft)	498	498	430	317	317	138	237	237	237	259	810	810	810	810	**830**
Avg. Delay (sec/veh)	70.0	61.2	7.6	46.6	45.6	21.5	3.7	57.6	63.8	10.7	45.1	41.8	37.5	6.0	**35.9**
Stopped Delay (sec/veh)	54.7	49.0	2.9	35.5	35.4	16.1	0.9	50.3	54.8	5.4	34.6	31.6	27.9	1.8	**27.4**
Avg. Stops (stops/veh)	1.18	1.05	0.32	0.88	0.82	0.55	0.23	0.93	0.99	0.65	1.03	0.95	0.94	0.26	**0.76**

Add U-Turn (Westbound)

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	302	266	612	182	284	153	166	238	234	311	28	891	161	223	**4051**
Avg. Queue Length (ft)	98	98	19	78	78	3	35	69	69	84	58	115	115	115	**52**
Max. Queue Length (ft)	492	492	393	327	327	123	215	215	215	237	729	729	729	729	**755**
Avg. Delay (sec/veh)	69.0	61.6	7.2	45.3	47.0	21.7	3.5	57.9	60.3	8.9	7.7	40.0	38.0	5.6	**34.9**
Stopped Delay (sec/veh)	53.8	49.6	2.9	34.4	36.9	16.4	0.7	50.5	51.6	4.3	1.7	30.3	28.6	1.7	**26.7**
Avg. Stops (stops/veh)	1.15	1.02	0.29	0.84	0.82	0.53	0.17	0.94	0.96	0.57	0.32	0.92	0.91	0.24	**0.73**

Simulation Results for University at Briarcrest Site

Table 83. VISSIM Countermeasures Results– Adding Northbound U-Turn Lane: Performance Measures of AM Peak Hour at SH 6 @ Briarcrest Dr.

Base Condition*															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	414	359	357	411	576	106	94	818	147	238	12	116	205	613	**4466**
Avg. Queue Length (ft)	57	57	57	91	91	0	81	81	81	81	41	41	41	41	**54**
Max. Queue Length (ft)	222	222	222	342	342	31	340	340	340	340	339	339	339	339	**387**
Avg. Delay (sec/veh)	51.7	40.2	2.4	53.8	49.3	1.6	49.0	33.7	26.8	3.1	83.2	37.9	34.2	12.2	**32.3**
Stopped Delay (sec/veh)	32.6	26.2	0.2	32.5	30.4	0.0	33.4	23.0	19.3	0.6	71.3	31.8	26.5	3.3	**20.3**
Avg. Stops (stops/veh)	1.02	0.73	0.04	0.88	0.77	0.02	1.61	0.69	0.60	0.12	1.84	0.79	0.68	0.47	**0.65**
Add U-Turn Lane for Northbound U-Turn															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	414	359	358	411	576	106	94	817	147	238	12	116	205	611	**4464**
Avg. Queue Length (ft)	56	56	56	91	91	0	76	76	76	76	41	41	41	41	**53**
Max. Queue Length (ft)	222	222	222	345	345	69	340	340	340	340	359	359	359	359	**389**
Avg. Delay (sec/veh)	51.9	40.3	2.4	52.7	49.2	1.6	17.0	32.7	26.4	3.0	83.5	37.9	34.2	11.8	**31.3**
Stopped Delay (sec/veh)	32.7	26.2	0.2	32.2	30.5	0.0	6.1	22.5	19.1	0.5	71.6	31.8	26.5	3.2	**19.6**
Avg. Stops (stops/veh)	1.01	0.73	0.04	0.86	0.77	0.02	0.35	0.68	0.60	0.12	1.81	0.79	0.68	0.45	**0.61**

Departure: Adding Lanes/Turn Bays

Simulation Results for I-10 at Gessner Site

Table 84. VISSIM Countermeasures Results—Adding U-Turn Lanes for Departure: Performance Measures at I-10 @ Gessner Rd.

Measure of Effectiveness	AM Peak Hour		PM Peak Hour	
	Base	Add U-Turn Lane	Base	Add U-Turn Lane
Eastbound U-Turn Departure Traffic				
Number of Vehicles	321	321	269	268
Avg. Queue Length (ft)	6.9	4.9	16.5	12.2
Max. Queue Length (ft)	157.1	117.2	179.7	204.0
Avg. Queue Stops (stops)	148	115	191	160
Southbound Right-Turn Traffic				
Number of Vehicles	384	384	307	307
Avg. Queue Length (ft)	93	93	129	130
Max. Queue Length (ft)	323	317	379	390
Avg. Delay (sec/veh)	14.1	13.8	19.9	19.8
Stopped Delay (sec/veh)	9.7	9.5	14.9	14.8
Avg. Stops (stops/veh)	0.31	0.31	0.41	0.41
Westbound Through Traffic				
Number of Vehicles	511	510	725	716
Avg. Queue Length (ft)	260	264	1642	1644
Max. Queue Length (ft)	617	643	1669	1669
Avg. Delay (sec/veh)	56.3	55.7	367.7	384.9
Stopped Delay (sec/veh)	43.4	43.0	251.5	265.1
Avg. Stops (stops/veh)	0.94	0.93	5.20	5.41
Northbound Left-Turn Traffic				
Number of Vehicles	405	405	912	307
Avg. Queue Length (ft)	58	58	86	130
Max. Queue Length (ft)	179	177	300	390
Avg. Delay (sec/veh)	44.3	44.0	42.7	19.8
Stopped Delay (sec/veh)	34.9	35.0	29.7	14.8
Avg. Stops (stops/veh)	0.82	0.82	0.77	0.41

Simulation Results for I-10 at Bunker Hill Site

Table 85. VISSIM Countermeasures Results—Adding U-Turn Lanes for Departure: Performance Measures at I-10 @ Bunker Hill Rd.

Measure of Effectiveness	AM Peak Hour		PM Peak Hour	
	Base	Add U-Turn Lane	Base	Add U-Turn Lane
Eastbound U-Turn Departure Traffic				
Number of Vehicles	182	182	573	574
Avg. Queue Length (ft)	1.5	1.3	1110.7	1116.9
Max. Queue Length (ft)	84.6	76.7	1668.7	1671.2
Avg. Queue Stops (stops)	51	48	3536	3540
Southbound Right-Turn Traffic				
Number of Vehicles	191	189	222	222
Avg. Queue Length (ft)	982	1018	263	232
Max. Queue Length (ft)	1561	1617	824	761
Avg. Delay (sec/veh)	190.9	196.4	90.4	83.4
Stopped Delay (sec/veh)	139.0	143.0	72.7	67.6
Avg. Stops (stops/veh)	2.99	3.11	1.45	1.36
Westbound Through Traffic				
Number of Vehicles	544	544	1705	1708
Avg. Queue Length (ft)	91	91	168	179
Max. Queue Length (ft)	325	327	497	576
Avg. Delay (sec/veh)	41.4	41.5	47.6	49.5
Stopped Delay (sec/veh)	32.2	32.3	33.7	34.9
Avg. Stops (stops/veh)	0.77	0.77	0.85	0.88
Northbound Left-Turn Traffic				
Number of Vehicles	412	412	584	580
Avg. Queue Length (ft)	325	322	322	355
Max. Queue Length (ft)	747	767	828	860
Avg. Delay (sec/veh)	87.5	86.6	92.3	99.4
Stopped Delay (sec/veh)	67.9	67.3	69.5	75.1
Avg. Stops (stops/veh)	1.35	1.34	1.44	1.52

Departure: Separation from the Conflicted Traffic

Simulation Results for I-45 at Rayford Site

Table 86. VISSIM Countermeasures Results—Separation from Conflicted Traffic: Performance Measures of Southbound U-Turn Departure End at I-45 @ Rayford Rd.

AM Peak Hour						
Measure of Effectiveness	Base	Change (1)	Change (2)	Change (3) with Compliance Rate		
				0%	50%	100%
Number of Vehicles	341	345	344	345	346	345
Avg. Queue Length (ft)	2.15	2.68	100.18	0.01	0.05	0.04
Max. Queue Length (ft)	145.56	133.33	544.38	18.64	28.13	18.78
Avg. Queue Stops (stops)	36	35	338	1	1	1
PM Peak Hour						
Measure of Effectiveness	Base	Change (1)	Change (2)	Change (3) with Compliance Rate		
				0%	50%	100%
Number of Vehicles	472	471	466	451	469	472
Avg. Queue Length (ft)	523.6	310.72	488.12	1.19	2.11	2.49
Max. Queue Length (ft)	1524.97	1524.45	1525.51	119.9	176.87	246.74
Avg. Queue Stops (stops)	1936	1215	1840	16	26	26

Table 87. VISSIM Countermeasures Results—Separation from Conflicted Traffic: Performance Measures of Westbound Right Turn at I-45 @ Rayford Rd.

AM Peak Hour						
Measure of Effectiveness	Base	Change (1)	Change (2)	Change (3) with Compliance Rate		
				0%	50%	100%
Number of Vehicles	950	950	959	949	949	949
Avg. Queue Length (ft)	110	108	107	53	52	52
Max. Queue Length (ft)	801	748	802	228	234	231
Avg. Delay (sec/veh)	18.5	18.5	17.8	3.9	4.0	4.0
Stopped Delay (sec/veh)	2.5	2.5	2.1	0.0	0.0	0.0
Avg. Stops (stops/veh)	0.46	0.46	0.45	0.00	0.00	0.00
PM Peak Hour						
Measure of Effectiveness	Base	Change (1)	Change (2)	Change (3) with Compliance Rate		
				0%	50%	100%
Number of Vehicles	503	503	506	510	510	510
Avg. Queue Length (ft)	103	105	100	69	71	68
Max. Queue Length (ft)	415	434	400	251	264	265
Avg. Delay (sec/veh)	16.6	16.7	21.0	2.6	2.6	2.6
Stopped Delay (sec/veh)	6.6	6.8	7.5	0.1	0.1	0.0
Avg. Stops (stops/veh)	0.67	0.66	0.83	0.01	0.01	0.01

Table 88. VISSIM Countermeasures Results—Separation from Conflicted Traffic: Performance Measures of Northbound Through at I-45 @ Rayford Rd.

AM Peak Hour						
Measure of Effectiveness	Base	Change (1)	Change (2)	Change (3) with Compliance Rate		
				0%	50%	100%
Number of Vehicles	253	280	283	281	278	278
Avg. Queue Length (ft)	1530	1530	1533	1539	1534	1530
Max. Queue Length (ft)	1674	1674	1674	1674	1674	1674
Avg. Delay (sec/veh)	231.4	229.0	229.3	226.7	228.9	228.1
Stopped Delay (sec/veh)	153.2	151.5	152.2	150.2	152.3	150.7
Avg. Stops (stops/veh)	3.18	3.14	3.19	3.10	3.15	3.14
PM Peak Hour						
Measure of Effectiveness	Base	Change (1)	Change (2)	Change (3) with Compliance Rate		
				0%	50%	100%
Number of Vehicles	357	384	385	386	385	385
Avg. Queue Length (ft)	1324	1309	1311	1316	1324	1322
Max. Queue Length (ft)	1674	1674	1674	1674	1673	1673
Avg. Delay (sec/veh)	196.5	194.4	194.8	196.8	198.8	195.7
Stopped Delay (sec/veh)	134.1	132.1	132.9	133.9	135.9	133.1
Avg. Stops (stops/veh)	2.57	2.54	2.55	2.53	2.60	2.59

Table 89. VISSIM Countermeasures Results—Separation from Conflicted Traffic: Performance Measures of Eastbound Left Turn at I-45 @ Rayford Rd.

AM Peak Hour						
Measure of Effectiveness	Base	Change (1)	Change (2)	Change (3) with Compliance Rate		
				0%	50%	100%
Number of Vehicles	351	359	359	359	361	360
Avg. Queue Length (ft)	70	71	70	70	70	70
Max. Queue Length (ft)	455	418	412	439	411	435
Avg. Delay (sec/veh)	42.4	42.3	42.3	42.5	42.6	42.6
Stopped Delay (sec/veh)	34.3	34.2	34.2	34.2	34.2	34.3
Avg. Stops (stops/veh)	0.82	0.82	0.82	0.82	0.82	0.82
PM Peak Hour						
Measure of Effectiveness	Base	Change (1)	Change (2)	Change (3) with Compliance Rate		
				0%	50%	100%
Number of Vehicles	648	667	681	673	677	675
Avg. Queue Length (ft)	508	478	321	415	344	350
Max. Queue Length (ft)	1081	1058	911	996	948	968
Avg. Delay (sec/veh)	150.9	147.9	111.7	136.9	123.5	123.2
Stopped Delay (sec/veh)	104.0	102.6	80.6	95.8	87.9	87.1
Avg. Stops (stops/veh)	2.15	2.13	1.72	1.96	1.79	1.83

Signal System and Timing: Signal Timing Changes

Simulation Results for University at Briarcrest Site

Table 90. VISSIM Countermeasures Results—Interior Left-Turn Operations: Performance Measures of AM Peak Hour at SH 6 @ Briarcrest Dr.

Base Condition with Protected-Permissive Left Turn for Interior Left-Turn Traffic															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	414	359	357	411	576	106	94	818	147	238	12	116	205	613	**4466**
Avg. Queue Length (ft)	57	57	57	91	91	0	81	81	81	81	41	41	41	41	**54**
Max. Queue Length (ft)	222	222	222	342	342	31	340	340	340	340	339	339	339	339	**387**
Avg. Delay (sec/veh)	51.7	40.2	2.4	53.8	49.3	1.6	49.0	33.7	26.8	3.1	83.2	37.9	34.2	12.2	**32.3**
Stopped Delay (sec/veh)	32.6	26.2	0.2	32.5	30.4	0.0	33.4	23.0	19.3	0.6	71.3	31.8	26.5	3.3	**20.3**
Avg. Stops (stops/veh)	1.02	0.73	0.04	0.88	0.77	0.02	1.61	0.69	0.60	0.12	1.84	0.79	0.68	0.47	**0.65**
Protected-Only Left Turn for Interior Left-Turn Traffic															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	416	359	357	412	576	106	93	818	147	238	12	116	205	612	**4468**
Avg. Queue Length (ft)	57	57	57	91	91	1	81	81	81	81	44	44	44	44	**55**
Max. Queue Length (ft)	222	222	222	335	335	104	338	338	338	338	359	359	359	359	**375**
Avg. Delay (sec/veh)	78.4	41.5	2.4	62.9	50.1	1.6	95.0	35.0	26.8	3.4	129.2	38.7	34.2	13.3	**37.4**
Stopped Delay (sec/veh)	53.1	26.5	0.2	37.6	30.5	0.1	79.0	23.6	19.4	0.7	113.0	32.3	26.5	4.0	**24.0**
Avg. Stops (stops/veh)	1.32	0.75	0.04	1.07	0.79	0.02	1.59	0.71	0.61	0.14	1.89	0.79	0.68	0.52	**0.71**

Signal System and Timing: Signalized U-Turn

Simulation Results for I-410 at Ingram Site

Table 91. VISSIM Countermeasures Results Summary: I-410 @ Ingram AM Peak Hour—Signalized Control U-Turn.

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Base Condition															
Number of Vehicles	603	363	331	120	351	443	151	189	321	167	139	244	174	116	**3713**
Avg. Queue Length (ft)	76	76	0	50	50	7	25	51	51	2	22	43	43	1	**25**
Max. Queue Length (ft)	294	294	21	208	208	183	180	180	180	82	166	166	167	49	**294**
Avg. Delay (sec/veh)	38.3	36.3	1.9	40.9	44.0	4.7	1.4	40.4	41.7	3.1	1.8	41.2	42.2	2.0	**26.7**
Stopped Delay (sec/veh)	26.2	25.9	0.0	30.3	32.6	1.2	0.0	33.1	32.4	0.9	0.1	35.1	32.6	0.4	**19.4**
Avg. Stops (stops/veh)	0.76	0.75	0.01	0.78	0.92	0.28	0.02	0.91	0.81	0.23	0.06	0.86	0.86	0.10	**0.58**
Signalized Control U-Turn															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	603	363	331	120	351	443	151	189	321	167	140	244	174	116	**3713**
Avg. Queue Length (ft)	76	76	0	50	50	7	38	51	51	2	36	43	43	1	**30**
Max. Queue Length (ft)	294	294	21	208	208	183	190	182	182	82	177	166	167	50	**294**
Avg. Delay (sec/veh)	38.3	36.4	1.8	40.9	44.0	4.7	31.4	40.5	41.7	3.1	37.7	41.2	42.1	2.0	**29.2**
Stopped Delay (sec/veh)	26.2	25.9	0.0	30.3	32.6	1.2	26.1	33.2	32.4	0.9	32.4	35.1	32.5	0.4	**21.7**
Avg. Stops (stops/veh)	0.76	0.76	0.01	0.78	0.92	0.29	0.73	0.91	0.81	0.24	0.83	0.86	0.86	0.13	**0.63**

Table 92. VISSIM Countermeasures Results Summary: I-410 @ Ingram PM Peak Hour—Signalized Control U-Turn.

Base Condition														
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	564	621	442	329	486	440	353	483	495	247	330	285	716	321	**6112**
Avg. Queue Length (ft)	227	227	5	144	144	23	78	155	155	9	110	220	220	21	**81**
Max. Queue Length (ft)	907	907	207	654	654	395	461	461	461	151	739	739	739	469	**944**
Avg. Delay (sec/veh)	75.8	73.3	7.1	66.2	65.3	11.5	3.3	68.4	68.5	11.7	2.6	62.9	82.0	25.2	**50.2**
Stopped Delay (sec/veh)	60.1	60.1	2.3	52.5	53.1	5.4	0.4	58.3	57.4	5.9	0.3	53.7	67.8	14.7	**40.1**
Avg. Stops (stops/veh)	1.11	1.09	0.28	0.95	0.97	0.62	0.25	0.93	0.93	0.62	0.04	0.91	1.05	0.90	**0.81**
Signalized Control U-Turn															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	EB			**WB**			**NB**				**SB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	566	623	442	329	486	441	349	483	494	247	329	287	721	322	**6119**
Avg. Queue Length (ft)	223	223	5	141	141	22	168	160	160	9	192	217	217	21	**114**
Max. Queue Length (ft)	847	847	217	628	628	423	579	481	481	153	738	720	720	412	**901**
Avg. Delay (sec/veh)	76.1	73.4	6.8	65.9	64.6	11.0	76.2	69.3	68.8	12.0	78.4	62.9	80.9	24.1	**58.4**
Stopped Delay (sec/veh)	60.5	60.3	2.1	52.3	52.5	4.9	69.1	59.1	57.8	6.1	71.4	53.9	66.9	13.8	**47.8**
Avg. Stops (stops/veh)	1.10	1.09	0.28	0.95	0.96	0.62	1.05	0.94	0.93	0.63	1.05	0.89	1.05	0.91	**0.91**

Signs and Markings: Added Lane Sign for U-Turn Lane

Simulation Results for I-410 at Ingram Site

Table 93. VISSIM Countermeasures Results Summary: I-410 @ Ingram AM Peak Hour—Added Lane Sign for U-Turn Lane.

Base Condition

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	603	363	331	120	351	443	151	189	321	167	139	244	174	116	**3713**
Avg. Queue Length (ft)	76	76	0	50	50	7	25	51	51	2	22	43	43	1	**25**
Max. Queue Length (ft)	294	294	21	208	208	183	180	180	180	82	166	166	167	49	**294**
Avg. Delay (sec/veh)	38.3	36.3	1.9	40.9	44.0	4.7	1.4	40.4	41.7	3.1	1.8	41.2	42.2	2.0	**26.7**
Stopped Delay (sec/veh)	26.2	25.9	0.0	30.3	32.6	1.2	0.0	33.1	32.4	0.9	0.1	35.1	32.6	0.4	**19.4**
Avg. Stops (stops/veh)	0.76	0.75	0.01	0.78	0.92	0.28	0.02	0.91	0.81	0.23	0.06	0.86	0.86	0.10	**0.58**

Added Lane Sign for U-Turn Lane (100% Compliance)

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	603	363	331	120	351	443	151	189	321	167	139	244	174	116	**3713**
Avg. Queue Length (ft)	76	76	0	50	50	7	25	51	51	2	22	43	43	1	**25**
Max. Queue Length (ft)	294	294	21	208	208	178	180	180	180	82	166	166	167	50	**294**
Avg. Delay (sec/veh)	38.3	36.3	1.8	40.9	44.0	4.7	1.0	40.4	41.7	3.1	1.0	41.2	42.2	2.0	**26.6**
Stopped Delay (sec/veh)	26.2	25.9	0.0	30.3	32.6	1.2	0.0	33.1	32.4	0.9	0.0	35.1	32.6	0.4	**19.4**
Avg. Stops (stops/veh)	0.76	0.75	0.01	0.78	0.92	0.28	0.00	0.91	0.81	0.24	0.00	0.86	0.86	0.13	**0.57**

Added Lane Sign for U-Turn Lane (50% Compliance)

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	603	361	331	120	349	442	155	185	327	167	140	249	175	114	**3718**
Avg. Queue Length (ft)	79	79	0	51	51	7	26	51	51	2	21	43	42	1	**25**
Max. Queue Length (ft)	274	274	31	200	200	165	187	187	187	76	160	160	158	51	**274**
Avg. Delay (sec/veh)	39.4	36.6	1.8	43.7	45.3	4.6	2.4	42.3	42.1	2.9	2.1	41.0	42.9	2.0	**27.3**
Stopped Delay (sec/veh)	27.1	26.0	0.0	32.7	33.7	1.2	0.0	34.9	32.7	0.9	0.1	34.9	33.1	0.4	**19.8**
Avg. Stops (stops/veh)	0.79	0.78	0.01	0.83	0.95	0.25	0.00	0.91	0.82	0.20	0.01	0.86	0.87	0.13	**0.58**

Table 94. VISSIM Countermeasures Results Summary: I-410 @ Ingram PM Peak Hour—Added Lane Sign for U-Turn Lane.

Base Condition

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	564	621	442	329	486	440	353	483	495	247	330	285	716	321	**6112**
Avg. Queue Length (ft)	227	227	5	144	144	23	78	155	155	9	110	220	220	21	**81**
Max. Queue Length (ft)	907	907	207	654	654	395	461	461	461	151	739	739	739	469	**944**
Avg. Delay (sec/veh)	75.8	73.3	7.1	66.2	65.3	11.5	3.3	68.4	68.5	11.7	2.6	62.9	82.0	25.2	**50.2**
Stopped Delay (sec/veh)	60.1	60.1	2.3	52.5	53.1	5.4	0.4	58.3	57.4	5.9	0.3	53.7	67.8	14.7	**40.1**
Avg. Stops (stops/veh)	1.11	1.09	0.28	0.95	0.97	0.62	0.25	0.93	0.93	0.62	0.04	0.91	1.05	0.90	**0.81**

Added Lane Sign for U-Turn Lane (100% Compliance)

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	565	621	442	329	486	440	353	483	495	247	330	287	719	321	**6119**
Avg. Queue Length (ft)	227	227	6	143	143	22	79	154	154	9	114	207	207	20	**81**
Max. Queue Length (ft)	889	889	215	651	651	394	450	450	450	153	703	703	703	402	**929**
Avg. Delay (sec/veh)	76.0	73.6	7.3	66.1	64.9	11.4	1.9	68.1	68.1	11.2	1.9	62.4	78.5	22.8	**49.5**
Stopped Delay (sec/veh)	60.3	60.3	2.4	52.4	52.8	5.3	0.0	58.1	57.1	5.6	0.0	53.3	64.7	12.8	**39.5**
Avg. Stops (stops/veh)	1.11	1.11	0.30	0.95	0.97	0.61	0.01	0.93	0.92	0.60	0.01	0.89	1.02	0.85	**0.79**

Added Lane Sign for U-Turn Lane (50% Compliance)

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	566	623	442	326	480	441	341	496	502	244	328	289	711	330	**6119**
Avg. Queue Length (ft)	230	230	6	145	145	19	77	153	153	9	96	192	192	20	**77**
Max. Queue Length (ft)	924	924	243	645	645	369	479	479	479	144	649	649	649	400	**970**
Avg. Delay (sec/veh)	77.6	74.9	7.8	64.8	65.7	11.9	4.4	68.0	65.6	11.7	4.3	61.2	76.2	20.1	**49.5**
Stopped Delay (sec/veh)	62.0	61.8	2.8	51.4	53.6	5.6	0.1	57.8	54.8	5.9	0.3	52.3	62.8	10.5	**39.4**
Avg. Stops (stops/veh)	1.09	1.09	0.31	0.94	0.97	0.94	0.05	0.92	0.92	0.59	0.04	0.88	1.01	0.84	**0.79**

Table 95. U-Turn Departure Side Results: I-410 @ Ingram Base Scenario and Added Lane Sign for U-Turn Lane Improvement.

Measure of Effectiveness	AM Peak Hour			PM Peak Hour		
	Base	Compliance Rate		Base	Compliance Rate	
		50%	100%		50%	100%
Southbound U-Turn Departure End						
Number of Vehicles	139	140	139	330	328	330
Avg. Queue Length (ft)	0.41	0.13	0	0.40	0.64	0
Max. Queue Length (ft)	52	37	0	65	73	0
Avg. Queue Stops (stops)	13	2	0	9	9	0
Northbound U-Turn Departure End						
Number of Vehicles	151	155	151	353	341	353
Avg. Queue Length (ft)	0.14	0.01	0	2.39	0.66	0
Max. Queue Length (ft)	41	21	0	142	82	0
Avg. Queue Stops (stops)	7	1	0	42	10	0

Signs and Markings: Direct Left-Turn Traffic to Alternate Receiving Lanes

Simulation Results for I-45 at Research Forest Site

Table 96. VISSIM Countermeasures Results—Direct Vehicles to Alternate Receiving Lanes Performance Measures of Southbound U-Turn Traffic at I-45 @ Research Forest Dr.

Measure of Effectiveness	AM Peak Hour			PM Peak Hour		
	Base	Compliance Rate		Base	Compliance Rate	
		50%	100%		50%	100%
Number of Vehicles	308	308	308	512	510	510
Avg. Queue Length (ft)	0.86	0.65	0.75	97.5	89.8	88.7
Max. Queue Length (ft)	90.0	72.6	84.1	612.5	557.7	566.3
Avg. Queue Stops (stops)	24	23	24	513	505	486

Restrictions: No RTOR from Cross Street

Simulation Results for I-45 at Research Forest Site

Table 97. VISSIM Countermeasures Results—No RTOR from Cross-Street Performance Measures at Houston District I-45 @ Research Forest Dr.

Measure of Effectiveness	AM Peak Hour			PM Peak Hour		
	Base	Compliance Rate		Base	Compliance Rate	
		50%	100%		50%	100%
Southbound U-Turn Departure End						
Number of Vehicles	308	308	308	512	510	510
Avg. Queue Length (ft)	0.86	0.80	0.85	97.5	95.8	102.6
Max. Queue Length (ft)	90.0	84.8	89.6	612.5	562.8	597.2
Avg. Queue Stops (stops)	24	26	25	513	520	538
Westbound Right Turn						
Number of Vehicles	134	134	134	138	137	137
Avg. Queue Length (ft)	89	90	91	107	109	108
Max. Queue Length (ft)	260	255	254	294	297	296
Avg. Delay (sec/veh)	42.9	45.6	47.9	50.9	53.4	53.9
Stopped Delay (sec/veh)	35.5	37.9	40.1	42.2	44.5	45.0
Avg. Stops (stops/veh)	0.83	0.86	0.88	0.86	0.87	0.87

Simulation Results for I-20 at McCart Site

Table 98. VISSIM Countermeasures Results Summary: I-20 @ McCart AM Peak Hour—No RTOR from Cross Street (Southbound Only).

Base Condition															
Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	360	370	718	247	211	102	195	120	262	239	29	621	148	187	**3809**
Avg. Queue Length (ft)	89	89	11	79	79	3	27	51	51	66	85	85	85	85	**48**
Max. Queue Length (ft)	514	514	392	287	287	123	187	187	187	208	322	322	322	322	**517**
Avg. Delay (sec/veh)	47.9	44.7	7.4	50.7	46.9	18.3	4.1	46.6	50.3	6.1	45.4	42.8	38.4	3.4	**31.2**
Stopped Delay (sec/veh)	34.0	33.7	2.3	39.0	36.5	13.5	1.4	40.8	42.3	2.8	36.9	34.3	30.2	0.8	**23.4**
Avg. Stops (stops/veh)	0.97	0.88	0.29	0.90	0.86	0.51	0.30	0.85	0.89	0.41	0.97	0.91	0.84	0.14	**0.68**

NO RTOR (Southbound Only) (100% Compliance)															
Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	360	372	718	246	210	102	195	119	262	239	29	626	149	187	**3812**
Avg. Queue Length (ft)	89	89	9	80	80	83	27	52	52	67	85	85	85	85	**59**
Max. Queue Length (ft)	494	494	345	298	298	307	192	192	192	214	332	332	332	332	**496**
Avg. Delay (sec/veh)	47.9	44.6	7.2	49.7	46.5	47.1	4.0	47.0	50.6	6.4	42.7	43.3	39.2	3.6	**32.0**
Stopped Delay (sec/veh)	34.3	33.6	2.2	38.0	36.0	40.8	1.4	41.2	42.7	3.1	34.0	34.6	31.0	0.9	**24.1**
Avg. Stops (stops/veh)	0.95	0.88	0.28	0.89	0.86	0.90	0.27	0.84	0.89	0.42	0.97	0.92	0.83	0.15	**0.68**

NO RTOR (Southbound Only) (50% Compliance)															
Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	360	372	718	245	210	103	195	119	261	239	29	623	148	187	**3808**
Avg. Queue Length (ft)	89	89	9	81	81	80	27	51	51	66	85	85	85	85	**58**
Max. Queue Length (ft)	503	503	370	307	307	312	195	195	195	216	294	294	294	294	**503**
Avg. Delay (sec/veh)	48.3	45.2	7.2	50.6	47.8	35.2	4.3	45.6	50.6	6.6	43.1	43.1	39.4	3.5	**31.8**
Stopped Delay (sec/veh)	34.4	33.9	2.1	39.0	37.4	29.4	1.6	39.8	42.6	3.2	34.5	34.5	31.2	0.8	**23.9**
Avg. Stops (stops/veh)	0.97	0.90	0.27	0.88	0.86	0.74	0.33	0.84	0.91	0.44	0.97	0.92	0.85	0.15	**0.69**

Table 99. VISSIM Countermeasures Results Summary: I-20 @ McCart PM Peak Hour— No RTOR from Cross Street (Southbound Only).

Base Condition															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	303	267	612	180	281	152	166	237	234	312	29	895	160	223	**4051**
Avg. Queue Length (ft)	100	100	19	78	78	2	36	72	72	88	131	131	131	131	**61**
Max. Queue Length (ft)	498	498	430	317	317	138	237	237	237	259	810	810	810	810	**830**
Avg. Delay (sec/veh)	70.0	61.2	7.6	46.6	45.6	21.5	3.7	57.6	63.8	10.7	45.1	41.8	37.5	6.0	**35.9**
Stopped Delay (sec/veh)	54.7	49.0	2.9	35.5	35.4	16.1	0.9	50.3	54.8	5.4	34.6	31.6	27.9	1.8	**27.4**
Avg. Stops (stops/veh)	1.18	1.05	0.32	0.88	0.82	0.55	0.23	0.93	0.99	0.65	1.03	0.95	0.94	0.26	**0.76**
NO RTOR (Southbound Only) (100% Compliance)															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	302	267	612	181	282	152	166	236	233	311	29	897	161	223	**4050**
Avg. Queue Length (ft)	105	105	22	82	82	82	36	70	70	86	129	129	129	129	**72**
Max. Queue Length (ft)	463	463	440	295	295	298	217	217	217	239	827	827	827	827	**852**
Avg. Delay (sec/veh)	74.5	63.0	7.8	46.2	45.8	42.7	4.1	57.6	62.2	10.2	43.3	41.7	39.2	6.0	**37.1**
Stopped Delay (sec/veh)	58.9	50.8	3.1	35.0	35.6	36.0	1.1	50.2	53.3	5.2	33.1	31.5	29.3	1.8	**28.5**
Avg. Stops (stops/veh)	1.21	1.04	0.32	0.87	0.84	0.85	0.26	0.93	1.00	0.63	1.03	0.94	0.98	0.27	**0.78**
NO RTOR (Southbound Only) (50% Compliance)															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	302	269	612	180	282	152	166	236	232	312	29	898	161	222	**4051**
Avg. Queue Length (ft)	103	103	22	80	80	78	36	71	71	87	132	132	132	132	**72**
Max. Queue Length (ft)	467	467	395	298	298	300	229	229	229	251	901	901	901	901	**928**
Avg. Delay (sec/veh)	72.2	62.2	7.7	46.2	44.6	36.3	3.7	58.9	62.9	10.6	41.8	41.8	38.3	6.2	**36.7**
Stopped Delay (sec/veh)	56.8	49.9	3.0	35.3	34.6	30.0	0.9	51.5	54.0	5.2	31.5	31.8	28.7	2.0	**28.2**
Avg. Stops (stops/veh)	1.19	1.02	0.33	0.85	0.82	0.76	0.22	0.93	0.99	0.67	1.01	0.95	0.97	0.27	**0.77**

Table 100. U-Turn Departure Side Results: I-20 @ McCart Base Scenario and No RTOR from Cross-Street Improvement.

Measure of Effectiveness	AM Peak Hour			PM Peak Hour		
	Base	Compliance Rate		Base	Compliance Rate	
		50%	100%		50%	100%
Eastbound U-Turn Departure End (SB No RTOR)						
Number of Vehicles	195	195	195	166	166	166
Avg. Queue Length (ft)	2.56	2.74	2.43	1.58	1.55	1.87
Max. Queue Length (ft)	106	105	98	72	69	82
Avg. Queue Stops (stops)	46	45	44	30	29	33

Simulation Results for I-20 at Hulen Site

Table 101. VISSIM Countermeasures Results Summary: I-20 @ Hulen AM Peak Hour—No RTOR from Cross Street.

Base Condition															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	437	28	202	160	251	173	376	382	134	243	73	293	440	718	**3909**
Avg. Queue Length (ft)	47	47	0	34	34	1	25	47	47	0	29	58	58	7	**20**
Max. Queue Length (ft)	185	185	38	125	125	72	200	200	200	27	305	305	305	260	**317**
Avg. Delay (sec/veh)	42.2	40.4	1.7	43.7	43.6	2.4	9.5	37.4	35.6	1.3	2.7	36.7	37.3	4.3	**23.4**
Stopped Delay (sec/veh)	30.6	29.6	0.0	33.2	32.1	0.6	5.6	29.7	28.0	0.0	0.2	28.8	28.4	0.8	**17.0**
Avg. Stops (stops/veh)	0.87	0.80	0.02	0.81	0.84	0.17	0.34	0.79	0.73	0.01	0.07	0.83	0.79	0.17	**0.52**
NO RTOR (100% Compliance)															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	443	28	204	159	251	173	374	381	134	243	73	292	438	718	**3911**
Avg. Queue Length (ft)	54	54	44	34	34	48	25	47	47	0	29	57	57	7	**29**
Max. Queue Length (ft)	214	214	220	125	125	214	205	205	205	40	288	288	288	253	**308**
Avg. Delay (sec/veh)	42.8	39.2	40.4	41.9	43.6	48.1	9.5	37.5	35.3	1.3	2.4	37.7	36.9	4.1	**27.5**
Stopped Delay (sec/veh)	31.1	28.4	32.9	31.6	32.1	42.6	5.6	29.8	27.9	0.0	0.1	29.9	28.1	0.7	**20.6**
Avg. Stops (stops/veh)	0.85	0.77	0.83	0.80	0.83	0.97	0.36	0.79	0.72	0.02	0.05	0.81	0.79	0.15	**0.59**
NO RTOR (50% Compliance)															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	440	28	203	160	252	173	375	383	135	243	73	294	440	718	**3915**
Avg. Queue Length (ft)	53	53	33	34	34	32	25	47	47	0	28	56	56	7	**26**
Max. Queue Length (ft)	213	213	218	125	125	197	211	211	211	34	250	250	250	249	**275**
Avg. Delay (sec/veh)	43.6	40.7	30.9	41.9	43.5	32.7	9.6	37.7	34.3	1.4	2.5	37.6	36.7	3.7	**26.3**
Stopped Delay (sec/veh)	32.0	29.5	24.5	31.6	32.1	28.3	5.7	30.0	26.9	0.0	0.1	29.8	28.1	0.5	**19.6**
Avg. Stops (stops/veh)	0.84	0.82	0.68	0.78	0.81	0.74	0.36	0.79	0.72	0.03	0.05	0.82	0.77	0.14	**0.57**

**Table 102. VISSIM Countermeasures Results Summary: I-20 @ Hulen PM Peak Hour—
No RTOR from Cross Street.**

Base Condition

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	625	115	337	853	969	406	334	364	189	415	50	685	375	359	**6074**
Avg. Queue Length (ft)	94	94	10	177	177	6	39	75	75	11	67	134	134	3	**51**
Max. Queue Length (ft)	280	280	184	685	685	146	252	243	243	194	392	392	392	150	**685**
Avg. Delay (sec/veh)	63.9	55.9	7.4	52.7	48.7	13.5	4.3	64.0	58.1	7.6	6.9	60.6	53.3	7.3	**41.2**
Stopped Delay (sec/veh)	48.7	43.8	3.0	37.6	36.9	6.7	0.9	55.0	49.3	2.1	2.9	49.8	42.8	3.5	**31.2**
Avg. Stops (stops/veh)	0.95	0.86	0.38	0.91	0.90	0.60	0.22	0.91	0.84	0.39	0.22	0.96	0.89	0.28	**0.75**

NO RTOR (100% Compliance)

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	598	109	297	858	975	403	334	361	187	414	49	687	377	359	**6008**
Avg. Queue Length (ft)	392	392	395	212	212	193	38	74	74	10	65	130	130	3	**141**
Max. Queue Length (ft)	619	619	623	817	817	820	235	235	235	190	388	388	388	171	**832**
Avg. Delay (sec/veh)	115	138	242	56.3	52.0	58.6	3.9	63.6	57.0	7.3	5.6	59.7	53.3	7.3	**63.4**
Stopped Delay (sec/veh)	91.2	110	208	40.7	39.6	49.7	0.8	54.7	48.3	2.1	1.8	49.0	42.8	3.5	**50.5**
Avg. Stops (stops/veh)	1.83	2.44	3.46	0.99	0.96	1.06	0.17	0.91	0.83	0.37	0.18	0.95	0.89	0.27	**1.06**

NO RTOR (50% Compliance)

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	609	111	317	858	977	403	334	362	187	415	49	688	377	360	**6045**
Avg. Queue Length (ft)	250	250	252	208	208	166	38	74	74	10	67	135	135	2	**110**
Max. Queue Length (ft)	510	510	513	817	817	794	236	236	236	174	401	401	401	135	**840**
Avg. Delay (sec/veh)	83.3	89.7	142	55.9	52.0	51.5	4.0	63.3	56.8	7.3	7.4	61.2	53.3	7.2	**54.4**
Stopped Delay (sec/veh)	65.4	71.1	120	40.4	39.5	42.9	0.8	54.4	48.1	2.0	3.1	50.2	42.8	3.5	**42.8**
Avg. Stops (stops/veh)	1.22	1.49	2.11	0.98	0.96	1.00	0.18	0.90	0.82	0.38	0.26	0.97	0.90	0.25	**0.92**

Table 103. U-Turn Departure Side Results: I-20 @ Hulen Base Scenario and No RTOR from Cross-Street Improvement.

Measure of Effectiveness	AM Peak Hour			PM Peak Hour		
	Base	Compliance Rate		Base	Compliance Rate	
		50%	100%		50%	100%
Eastbound U-Turn Departure End (SB No RTOR)						
Number of Vehicles	376	375	374	334	334	334
Avg. Queue Length (ft)	4.69	4.93	5.25	4.03	3.43	3.37
Max. Queue Length (ft)	116	132	139	175	151	142
Avg. Queue Stops (stops)	98	101	102	70	63	60
Westbound U-Turn Departure End (NB No RTOR)						
Number of Vehicles	73	73	73	50	49	49
Avg. Queue Length (ft)	0.14	0.10	0.10	0.23	0.17	0.20
Max. Queue Length (ft)	29	28	26	33	27	38
Avg. Queue Stops (stops)	7	5	6	4	3	5

Simulation Results for I-410 at Ingram Site

Table 104. VISSIM Countermeasures Results Summary: I-410 @ Ingram AM Peak Hour—No RTOR from Cross Street.

Base Condition

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	603	363	331	120	351	443	151	189	321	167	139	244	174	116	**3713**
Avg. Queue Length (ft)	76	76	0	50	50	7	25	51	51	2	22	43	43	1	**25**
Max. Queue Length (ft)	294	294	21	208	208	183	180	180	180	82	166	166	167	49	**294**
Avg. Delay (sec/veh)	38.3	36.3	1.9	40.9	44.0	4.7	1.4	40.4	41.7	3.1	1.8	41.2	42.2	2.0	**26.7**
Stopped Delay (sec/veh)	26.2	25.9	0.0	30.3	32.6	1.2	0.0	33.1	32.4	0.9	0.1	35.1	32.6	0.4	**19.4**
Avg. Stops (stops/veh)	0.76	0.75	0.01	0.78	0.92	0.28	0.02	0.91	0.81	0.23	0.06	0.86	0.86	0.10	**0.58**

NO RTOR (100% Compliance)

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	605	361	330	86	252	306	151	187	322	167	139	245	176	116	**3444**
Avg. Queue Length (ft)	71	71	69	1104	1104	1447	24	48	48	2	21	43	42	0	**257**
Max. Queue Length (ft)	273	273	335	1502	1502	1671	177	177	177	82	164	164	161	46	**1674**
Avg. Delay (sec/veh)	35.7	34.1	35.7	371.4	373.2	474.1	1.3	40.5	39.4	3.3	1.7	40.9	42.5	1.8	**102.1**
Stopped Delay (sec/veh)	23.6	23.8	29.3	289.3	290.5	387.3	0.0	31.5	30.2	1.1	0.1	34.7	32.8	0.4	**80.0**
Avg. Stops (stops/veh)	0.76	0.73	0.83	8.73	8.92	10.27	0.02	1.05	0.81	0.23	0.06	0.87	0.87	0.10	**2.31**

NO RTOR (50% Compliance)

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	609	364	333	99	284	350	151	190	322	167	139	245	175	116	**3544**
Avg. Queue Length (ft)	76	76	56	1011	1011	1238	25	49	49	2	21	42	42	0	**229**
Max. Queue Length (ft)	286	286	321	1638	1638	1665	179	179	179	89	168	168	165	38	**1670**
Avg. Delay (sec/veh)	37.8	36.2	29.9	285.9	286.4	362.4	1.3	41.3	41.0	3.4	1.6	40.6	41.7	1.9	**90.5**
Stopped Delay (sec/veh)	25.6	25.7	24.0	220.9	220.0	293.5	0.0	32.5	31.7	1.1	0.1	34.5	31.9	0.4	**70.4**
Avg. Stops (stops/veh)	0.78	0.76	0.73	6.70	6.85	7.92	0.02	1.04	0.82	0.25	0.05	0.86	0.87	0.09	**2.04**

Table 105. VISSIM Countermeasures Results Summary: I-410 @ Ingram PM Peak Hour—No RTOR from Cross Street.

Base Condition

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	564	621	442	329	486	440	353	483	495	247	330	285	716	321	**6112**
Avg. Queue Length (ft)	227	227	5	144	144	23	78	155	155	9	110	220	220	21	**81**
Max. Queue Length (ft)	907	907	207	654	654	395	461	461	461	151	739	739	739	469	**944**
Avg. Delay (sec/veh)	75.8	73.3	7.1	66.2	65.3	11.5	3.3	68.4	68.5	11.7	2.6	62.9	82.0	25.2	**50.2**
Stopped Delay (sec/veh)	60.1	60.1	2.3	52.5	53.1	5.4	0.4	58.3	57.4	5.9	0.3	53.7	67.8	14.7	**40.1**
Avg. Stops (stops/veh)	1.11	1.09	0.28	0.95	0.97	0.62	0.25	0.93	0.93	0.62	0.04	0.91	1.05	0.90	**0.81**

NO RTOR (100% Compliance)

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	527	580	400	296	437	374	353	484	493	247	330	287	729	321	**5856**
Avg. Queue Length (ft)	1170	1170	1184	1107	1107	1102	76	150	150	9	94	188	188	18	**493**
Max. Queue Length (ft)	1662	1662	1668	1669	1669	1666	494	494	494	151	672	672	672	372	**1674**
Avg. Delay (sec/veh)	176.9	179.9	208.0	184.2	195.5	241.3	3.1	66.5	65.7	10.8	2.4	59.3	72.7	18.8	**112.1**
Stopped Delay (sec/veh)	139.1	143.9	174.3	144.0	154.6	202.0	0.3	56.3	54.7	5.4	0.1	50.3	59.5	9.8	**90.4**
Avg. Stops (stops/veh)	3.54	3.63	3.85	3.48	3.96	4.42	0.22	0.95	0.93	0.58	0.04	0.87	0.99	0.79	**2.10**

NO RTOR (50% Compliance)

Measure of Effectiveness	Arterial						Frontage Road								Total
	EB			WB			NB				SB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	539	594	416	302	450	389	353	482	492	247	329	288	725	321	**5928**
Avg. Queue Length (ft)	929	929	948	882	882	906	75	148	148	9	100	199	199	15	**404**
Max. Queue Length (ft)	1503	1503	1472	1561	1561	1537	463	463	463	150	671	671	671	304	**1673**
Avg. Delay (sec/veh)	151.8	155.4	174.7	167.1	177.2	198.3	3.0	65.9	65.1	11.1	2.6	61.6	75.2	20.4	**101.0**
Stopped Delay (sec/veh)	118.2	123.3	145.2	131.4	140.6	166.0	0.2	55.8	54.2	5.6	0.3	52.7	61.9	11.3	**81.2**
Avg. Stops (stops/veh)	2.93	3.10	3.27	3.06	3.46	3.64	0.20	0.94	0.92	0.58	0.05	0.89	1.00	0.79	**1.85**

Table 106. U-Turn Departure Side Results: I-410 @ Ingram Base Scenario and No RTOR from Cross-Street Improvement.

Measure of Effectiveness	AM Peak Hour			PM Peak Hour		
	Base	Compliance Rate		Base	Compliance Rate	
		50%	100%		50%	100%
Southbound U-Turn Departure End (WB No RTOT)						
Number of Vehicles	139	139	139	330	329	330
Avg. Queue Length (ft)	0.41	0.29	0.32	0.40	0.46	0.51
Max. Queue Length (ft)	52	43	49	65	71	91
Avg. Queue Stops (stops)	13	12	13	9	9	10
Northbound U-Turn Departure End (EB No RTOR)						
Number of Vehicles	151	151	151	353	353	353
Avg. Queue Length (ft)	0.14	0.10	0.10	2.39	1.64	1.83
Max. Queue Length (ft)	41	42	37	142	104	130
Avg. Queue Stops (stops)	7	5	4	42	34	37

Restrictions: No RTOR Except from Right Lane Sign from Cross Street

Simulation Results for I-10 at Gessner Site

Table 107. VISSIM Countermeasures Results—No RTOR Except from Right Lane Sign Performance Measures at I-10 @ Gessner Rd.

Measure of Effectiveness	AM Peak Hour			PM Peak Hour		
	Base	Compliance Rate		Base	Compliance Rate	
		50%	100%		50%	100%
Eastbound U-Turn Departure End						
Number of Vehicles	321	321	321	269	269	269
Avg. Queue Length (ft)	6.9	6.7	6.5	16.5	16.2	17.1
Max. Queue Length (ft)	157.1	146.0	140.2	179.7	189.6	208.3
Avg. Queue Stops (stops)	148	147	140	191	184	189
Southbound Right Turn						
Number of Vehicles	384	384	384	307	307	307
Avg. Queue Length (ft)	93	93	93	129	132	130
Max. Queue Length (ft)	323	315	317	379	382	376
Avg. Delay (sec/veh)	14.1	14.1	14.1	19.9	20.1	20.5
Stopped Delay (sec/veh)	9.7	9.8	9.8	14.9	15.1	15.5
Avg. Stops (stops/veh)	0.31	0.32	0.31	0.41	0.42	0.42
Westbound Through						
Number of Vehicles	511	508	505	725	705	736
Avg. Queue Length (ft)	260	289	300	1642	1644	1642
Max. Queue Length (ft)	617	666	671	1669	1668	1666
Avg. Delay (sec/veh)	56.3	60.5	63.1	367.7	378.4	367.5
Stopped Delay (sec/veh)	43.4	46.5	48.6	251.5	259.3	251.0
Avg. Stops (stops/veh)	0.94	0.99	1.02	5.20	5.43	5.19
Northbound Left Turn						
Number of Vehicles	405	405	405	912	912	912
Avg. Queue Length (ft)	58	59	59	86	85	85
Max. Queue Length (ft)	179	177	181	300	302	293
Avg. Delay (sec/veh)	44.3	44.3	44.3	42.7	44.0	44.2
Stopped Delay (sec/veh)	34.9	34.9	35.0	29.7	29.5	29.7
Avg. Stops (stops/veh)	0.82	0.81	0.82	0.77	0.79	0.79

Restrictions: Driveways Closed to U-Turn Traffic

Simulation Results for I-10 at Gessner Site

Table 108. VISSIM Countermeasures Results—Eastbound Driveway Closure to U-Turn Performance Measures at I-10 @ Gessner Rd.

Measure of Effectiveness	AM Peak Hour			PM Peak Hour		
	Base	Driveway Closure		Base	Driveway Closure	
		1st	1st & 2nd		1st	1st & 2nd
Eastbound U-Turn Departure End						
Number of Vehicles	184	184	184	224	220	226
Avg. Queue Length (ft)	0.31	0	0	0.72	0	0
Max. Queue Length (ft)	71.75	0	0	144.45	0	0
Avg. Queue Stops (stops)	4	0	0	12	0	0
Northbound Right Turn						
Number of Vehicles	275	275	275	276	276	276
Avg. Queue Length (ft)	58	58	58	85	85	85
Max. Queue Length (ft)	179	177	182	288	295	286
Avg. Delay (sec/veh)	26.9	26.3	26.0	23.6	22.9	22.9
Stopped Delay (sec/veh)	22.0	21.5	21.3	18.8	18.3	18.3
Avg. Stops (stops/veh)	0.63	0.61	0.61	0.53	0.52	0.52
Eastbound Through						
Number of Vehicles	1000	1000	1000	644	645	644
Avg. Queue Length (ft)	97	97	97	143	141	158
Max. Queue Length (ft)	294	295	304	455	442	471
Avg. Delay (sec/veh)	44.2	43.3	43.1	51.4	50.3	50.9
Stopped Delay (sec/veh)	33.5	33.3	33.2	40.8	40.6	41.3
Avg. Stops (stops/veh)	0.80	0.80	0.79	0.82	0.81	0.82
Southbound Left Turn						
Number of Vehicles	839	838	839	674	676	676
Avg. Queue Length (ft)	93	92	93	129	134	134
Max. Queue Length (ft)	323	316	328	379	406	413
Avg. Delay (sec/veh)	45.1	44.4	44.7	67.9	69.9	70.3
Stopped Delay (sec/veh)	30.9	30.6	30.8	53.1	55.2	55.3
Avg. Stops (stops/veh)	0.83	0.82	0.82	1.01	1.04	1.04

Simulation Results for I-45 at Research Forest Site

Table 109. VISSIM Countermeasures Results—No RTOR from Cross-Street Measure of Effectiveness of at I-45 @ Research Forest Dr.

Measure of Effectiveness	AM Peak Hour			PM Peak Hour		
	Base	Compliance Rate		Base	Compliance Rate	
		50%	100%		50%	100%
Southbound U-Turn Departure End						
Number of Vehicles	308	308	308	512	510	510
Avg. Queue Length (ft)	0.86	0.78	0.8	97.5	91.0	101.4
Max. Queue Length (ft)	90.0	77.64	76.7	612.5	582.8	601.3
Avg. Queue Stops (stops)	24	25	25	513	498	537
Westbound Right Turn						
Number of Vehicles	134	134	134	138	137	137
Avg. Queue Length (ft)	89	89	89	107	108	108
Max. Queue Length (ft)	260	260	260	294	298	292
Avg. Delay (sec/veh)	42.9	42.9	42.7	50.9	52.0	51.5
Stopped Delay (sec/veh)	35.5	35.5	35.3	42.2	43.2	42.8
Avg. Stops (stops/veh)	0.83	0.83	0.82	0.86	0.86	0.87
Northbound Through						
Number of Vehicles	362	362	362	1004	1004	1003
Avg. Queue Length (ft)	122	121	120	182	182	181
Max. Queue Length (ft)	561	566	552	592	582	587
Avg. Delay (sec/veh)	28.4	28.7	28.6	53.3	52.9	52.8
Stopped Delay (sec/veh)	20.4	20.6	20.6	39.8	39.5	39.4
Avg. Stops (stops/veh)	0.61	0.61	0.61	0.85	0.85	0.85
Eastbound Left Turn						
Number of Vehicles	406	406	406	1311	1318	1312
Avg. Queue Length (ft)	57	57	57	487	407	384
Max. Queue Length (ft)	206	204	203	1184	1000	1023
Avg. Delay (sec/veh)	45.7	45.7	45.8	108.4	95.3	92.4
Stopped Delay (sec/veh)	34.9	35.0	35.0	69.4	61.6	60.8
Avg. Stops (stops/veh)	0.79	0.79	0.79	1.55	1.38	1.32

Simulation Results for I-20 at McCart Site

Table 110. VISSIM Countermeasures Results Summary: I-20 @ McCart AM Peak Hour—Driveway Closure (Westbound First Driveway).

Base Condition

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	360	370	718	247	211	102	195	120	262	239	29	621	148	187	**3809**
Avg. Queue Length (ft)	89	89	11	79	79	3	27	51	51	66	85	85	85	85	**48**
Max. Queue Length (ft)	514	514	392	287	287	123	187	187	187	208	322	322	322	322	**517**
Avg. Delay (sec/veh)	47.9	44.7	7.4	50.7	46.9	18.3	4.1	46.6	50.3	6.1	45.4	42.8	38.4	3.4	**31.2**
Stopped Delay (sec/veh)	34.0	33.7	2.3	39.0	36.5	13.5	1.4	40.8	42.3	2.8	36.9	34.3	30.2	0.8	**23.4**
Avg. Stops (stops/veh)	0.97	0.88	0.29	0.90	0.86	0.51	0.30	0.85	0.89	0.41	0.97	0.91	0.84	0.14	**0.68**

Driveway Closure (Westbound First Driveway) (100% Compliance)

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	359	372	718	245	210	103	198	119	257	239	29	629	148	188	**3813**
Avg. Queue Length (ft)	85	85	11	80	80	2	26	51	51	67	84	84	84	84	**48**
Max. Queue Length (ft)	474	474	418	277	277	120	184	184	184	206	391	391	391	391	**503**
Avg. Delay (sec/veh)	47.2	43.7	6.6	50.8	48.0	18.5	3.2	47.1	51.9	5.5	45.1	42.5	37.4	3.4	**30.8**
Stopped Delay (sec/veh)	33.6	32.7	1.7	39.0	37.4	13.5	1.0	41.3	44.0	2.5	36.3	34.0	29.2	0.8	**23.1**
Avg. Stops (stops/veh)	0.94	0.89	0.27	0.91	0.87	0.51	0.23	0.84	0.89	0.35	1.04	0.91	0.84	0.16	**0.66**

Driveway Closure (Westbound First Driveway) (50% Compliance)

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	361	372	719	246	210	102	197	118	259	239	29	626	149	188	**3815**
Avg. Queue Length (ft)	87	87	8	80	80	4	27	52	52	67	86	86	86	86	**48**
Max. Queue Length (ft)	517	517	342	305	305	141	204	204	204	226	380	380	380	380	**541**
Avg. Delay (sec/veh)	48.2	43.9	6.6	50.4	47.1	18.3	3.6	47.3	51.5	6.5	45.6	43.5	36.4	3.8	**31.1**
Stopped Delay (sec/veh)	34.4	33.0	1.7	38.7	36.7	13.6	1.3	41.4	43.5	3.2	36.6	35.0	28.4	1.0	**23.4**
Avg. Stops (stops/veh)	0.96	0.88	0.26	0.90	0.86	0.48	0.25	0.85	0.89	0.40	1.08	0.92	0.83	0.18	**0.67**

Table 111. VISSIM Countermeasures Results Summary: I-20 @ McCart PM Peak Hour—Driveway Closure (Westbound First Driveway).

Base Condition															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	303	267	612	180	281	152	166	237	234	312	29	895	160	223	**4051**
Avg. Queue Length (ft)	100	100	19	78	78	2	36	72	72	88	131	131	131	131	**61**
Max. Queue Length (ft)	498	498	430	317	317	138	237	237	237	259	810	810	810	810	**830**
Avg. Delay (sec/veh)	70.0	61.2	7.6	46.6	45.6	21.5	3.7	57.6	63.8	10.7	45.1	41.8	37.5	6.0	**35.9**
Stopped Delay (sec/veh)	54.7	49.0	2.9	35.5	35.4	16.1	0.9	50.3	54.8	5.4	34.6	31.6	27.9	1.8	**27.4**
Avg. Stops (stops/veh)	1.18	1.05	0.32	0.88	0.82	0.55	0.23	0.93	0.99	0.65	1.03	0.95	0.94	0.26	**0.76**
Driveway Closure (Westbound First Driveway) (100% Compliance)															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	301	266	612	182	284	152	173	235	234	305	29	897	161	224	**4057**
Avg. Queue Length (ft)	103	103	17	77	77	5	35	70	70	86	130	130	130	130	**61**
Max. Queue Length (ft)	432	432	338	309	309	179	232	232	232	254	814	814	814	814	**838**
Avg. Delay (sec/veh)	72.9	62.5	7.7	46.4	44.9	20.9	3.0	56.6	61.7	10.8	42.8	41.5	37.9	6.3	**35.9**
Stopped Delay (sec/veh)	57.5	50.2	3.0	35.3	34.8	15.6	0.8	49.4	52.9	5.5	32.6	31.3	28.4	2.0	**27.4**
Avg. Stops (stops/veh)	1.19	1.03	0.33	0.88	0.82	0.55	0.27	0.92	0.98	0.68	0.99	0.96	0.93	0.27	**0.77**
Driveway Closure (Westbound First Driveway) (50% Compliance)															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	303	267	611	180	281	151	168	236	232	309	29	897	160	224	**4048**
Avg. Queue Length (ft)	109	109	29	77	77	4	35	70	70	85	132	132	132	132	**63**
Max. Queue Length (ft)	517	517	463	303	303	203	223	223	223	245	813	813	813	813	**862**
Avg. Delay (sec/veh)	73.2	64.3	8.7	46.7	45.0	19.8	3.5	58.5	60.6	9.8	43.4	42.7	38.9	6.4	**36.4**
Stopped Delay (sec/veh)	57.7	51.9	3.7	35.5	34.9	14.6	0.9	51.2	51.8	5.0	33.1	32.4	29.2	1.9	**27.9**
Avg. Stops (stops/veh)	1.21	1.06	0.36	0.89	0.83	0.53	0.25	0.94	0.98	0.59	1.05	0.96	0.93	0.27	**0.77**

Table 112. U-Turn Departure Side Results: I-20 @ McCart Base Scenario and Driveway Closure Improvement.

Measure of Effectiveness	AM Peak Hour			PM Peak Hour		
	Base	Compliance Rate		Base	Compliance Rate	
		50%	100%		50%	100%
Eastbound U-Turn Departure End						
Number of Vehicles	195	197	198	166	168	173
Avg. Queue Length (ft)	2.56	2.21	2.01	1.58	1.62	1.59
Max. Queue Length (ft)	106	89	89	72	86	85
Avg. Queue Stops (stops)	46	41	38	30	32	33

Simulation Results for I-20 at Hulen Site

Table 113. VISSIM Countermeasures Results Summary: I-20 @ Hulen AM Peak Hour—Driveway Closure (Westbound Only).

Base Condition

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	437	28	202	160	251	173	376	382	134	243	73	293	440	718	**3909**
Avg. Queue Length (ft)	47	47	0	34	34	1	25	47	47	0	29	58	58	7	**20**
Max. Queue Length (ft)	185	185	38	125	125	72	200	200	200	27	305	305	305	260	**317**
Avg. Delay (sec/veh)	42.2	40.4	1.7	43.7	43.6	2.4	9.5	37.4	35.6	1.3	2.7	36.7	37.3	4.3	**23.4**
Stopped Delay (sec/veh)	30.6	29.6	0.0	33.2	32.1	0.6	5.6	29.7	28.0	0.0	0.2	28.8	28.4	0.8	**17.0**
Avg. Stops (stops/veh)	0.87	0.80	0.02	0.81	0.84	0.17	0.34	0.79	0.73	0.01	0.07	0.83	0.79	0.17	**0.52**

Driveway Closure (WB First Driveway) (100% Compliance)

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	440	28	202	158	250	173	375	386	134	243	73	291	435	718	**3905**
Avg. Queue Length (ft)	47	47	0	34	34	1	25	48	48	0	30	60	60	7	**20**
Max. Queue Length (ft)	178	178	44	124	124	61	211	211	211	44	306	306	306	242	**314**
Avg. Delay (sec/veh)	42.9	41.6	1.7	41.8	44.6	2.3	9.2	38.2	33.9	1.4	2.5	38.3	37.2	4.6	**23.7**
Stopped Delay (sec/veh)	31.4	30.7	0.0	31.5	33.3	0.5	5.7	30.4	26.7	0.1	0.1	30.4	28.5	1.0	**17.3**
Avg. Stops (stops/veh)	0.84	0.79	0.02	0.79	0.83	0.16	0.35	0.79	0.71	0.03	0.04	0.82	0.78	0.18	**0.51**

Driveway Closure (WB First Driveway) (50% Compliance)

Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	437	28	202	160	251	173	375	380	133	243	73	293	441	718	**3908**
Avg. Queue Length (ft)	48	48	0	34	34	1	25	47	47	0	29	58	58	8	**20**
Max. Queue Length (ft)	172	172	48	123	123	60	215	215	215	27	290	290	290	291	**311**
Avg. Delay (sec/veh)	42.6	40.2	1.7	42.8	43.9	2.4	9.8	37.8	34.8	1.3	2.4	37.9	37.2	4.3	**23.6**
Stopped Delay (sec/veh)	31.0	29.4	0.0	32.4	32.4	0.5	6.1	30.2	27.3	0.0	0.1	29.9	28.5	0.8	**17.2**
Avg. Stops (stops/veh)	0.85	0.78	0.03	0.81	0.85	0.18	0.36	0.78	0.73	0.02	0.05	0.83	0.77	0.17	**0.52**

Table 113. VISSIM Countermeasures Results Summary: I-20 @ Hulen AM Peak Hour—Driveway Closure (Westbound Only). (Continued).

Driveway Closure (WB First and Second Driveway) (100% Compliance)															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	440	28	202	158	250	173	375	386	134	243	73	291	435	718	**3905**
Avg. Queue Length (ft)	47	47	0	34	34	1	25	48	48	0	30	60	60	7	**20**
Max. Queue Length (ft)	181	181	47	124	124	61	210	210	210	44	306	306	306	239	**314**
Avg. Delay (sec/veh)	42.9	42.8	1.7	41.7	44.3	2.3	9.2	38.2	34.2	1.4	2.5	38.2	37.2	4.6	**23.7**
Stopped Delay (sec/veh)	31.4	31.8	0.0	31.4	32.9	0.5	5.7	30.5	27.0	0.1	0.1	30.4	28.4	1.0	**17.3**
Avg. Stops (stops/veh)	0.84	0.81	0.02	0.79	0.83	0.15	0.35	0.80	0.71	0.02	0.04	0.83	0.78	0.18	**0.51**
Driveway Closure (WB First and Second Driveway) (50% Compliance)															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	440	28	202	158	250	173	375	386	134	243	73	291	435	718	**3905**
Avg. Queue Length (ft)	47	47	0	34	34	1	25	48	48	0	30	60	60	7	**20**
Max. Queue Length (ft)	178	178	44	124	124	61	211	211	211	44	306	306	306	242	**314**
Avg. Delay (sec/veh)	42.9	41.6	1.7	41.8	44.6	2.3	9.2	38.2	33.9	1.4	2.5	38.3	37.2	4.6	**23.7**
Stopped Delay (sec/veh)	31.4	30.7	0.0	31.5	33.3	0.5	5.7	30.4	26.7	0.1	0.1	30.4	28.5	1.0	**17.3**
Avg. Stops (stops/veh)	0.84	0.79	0.02	0.79	0.83	0.16	0.34	0.79	0.71	0.03	0.04	0.82	0.78	0.18	**0.51**

Table 114. VISSIM Countermeasures Results Summary: I-20 @ Hulen PM Peak Hour—Driveway Closure (Westbound Only).

Base Condition															
Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	625	115	337	853	969	406	334	364	189	415	50	685	375	359	**6074**
Avg. Queue Length (ft)	94	94	10	177	177	6	39	75	75	11	67	134	134	3	**51**
Max. Queue Length (ft)	280	280	184	685	685	146	252	243	243	194	392	392	392	150	**685**
Avg. Delay (sec/veh)	63.9	55.9	7.4	52.7	48.7	13.5	4.3	64.0	58.1	7.6	6.9	60.6	53.3	7.3	**41.2**
Stopped Delay (sec/veh)	48.7	43.8	3.0	37.6	36.9	6.7	0.9	55.0	49.3	2.1	2.9	49.8	42.8	3.5	**31.2**
Avg. Stops (stops/veh)	0.95	0.86	0.38	0.91	0.90	0.60	0.22	0.91	0.84	0.39	0.22	0.96	0.89	0.28	**0.75**

Driveway Closure (WB First Driveway) (100% Compliance)															
Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	623	112	337	853	970	406	336	372	190	414	49	684	376	360	**6082**
Avg. Queue Length (ft)	93	93	13	176	176	8	37	73	73	10	67	134	134	2	**51**
Max. Queue Length (ft)	269	269	278	642	642	173	244	244	244	204	381	381	381	128	**642**
Avg. Delay (sec/veh)	64.0	52.7	7.4	52.9	48.3	13.8	2.5	61.5	56.7	7.2	6.9	61.0	52.8	7.4	**40.8**
Stopped Delay (sec/veh)	48.7	40.7	2.8	37.6	36.4	7.0	0.2	52.6	47.8	2.1	2.8	50.1	42.2	3.6	**30.8**
Avg. Stops (stops/veh)	0.95	0.83	0.38	0.91	0.90	0.67	0.13	0.91	0.84	0.38	0.24	0.97	0.90	0.26	**0.75**

Driveway Closure (WB First Driveway) (50% Compliance)															
Measure of Effectiveness	Arterial						Frontage Road								Total
	NB			SB			EB				WB				
	LT	Th	RT	LT	Th	RT	UT	LT	Th	RT	UT	LT	Th	RT	
Number of Vehicles	623	115	338	853	972	406	336	365	189	413	50	683	374	359	**6075**
Avg. Queue Length (ft)	95	95	13	181	181	10	37	73	73	10	68	136	136	3	**52**
Max. Queue Length (ft)	268	268	264	707	707	311	249	248	248	220	395	395	395	182	**707**
Avg. Delay (sec/veh)	64.5	57.6	7.7	53.7	49.3	14.0	3.5	61.5	57.9	7.2	7.1	61.3	54.2	7.3	**41.4**
Stopped Delay (sec/veh)	49.3	45.3	3.0	38.4	37.4	7.1	0.6	52.6	48.9	2.1	2.9	50.3	43.6	3.4	**31.4**
Avg. Stops (stops/veh)	0.95	0.87	0.40	0.93	0.91	0.64	0.18	0.90	0.85	0.36	0.24	0.99	0.90	0.28	**0.76**

Table 114. VISSIM Countermeasures Results Summary: I-20 @ Hulen PM Peak Hour—Driveway Closure (Westbound Only). (Continued).

Driveway Closure (WB First and Second Driveway) (100% Compliance)														
Measure of Effectiveness	**Arterial**						**Frontage Road**							**Total**
	NB			**SB**			**EB**				**WB**			
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	625	113	337	853	969	406	336	372	190	414	49	685	376	360	**6084**
Avg. Queue Length (ft)	93	93	13	174	174	8	37	73	73	10	67	134	134	2	**51**
Max. Queue Length (ft)	265	265	270	619	619	195	239	239	239	204	391	391	391	122	**619**
Avg. Delay (sec/veh)	63.9	52.6	7.4	52.7	48.2	13.5	2.5	61.4	56.3	7.3	7.4	61.2	52.8	7.6	**40.7**
Stopped Delay (sec/veh)	48.6	40.7	2.8	37.6	36.4	6.8	0.2	52.5	47.4	2.1	3.3	50.2	42.3	3.7	**30.8**
Avg. Stops (stops/veh)	0.96	0.84	0.38	0.91	0.89	0.65	0.14	0.91	0.84	0.39	0.23	0.98	0.89	0.28	**0.75**
Driveway Closure (WB First and Second Driveway) (50% Compliance)															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	620	112	337	853	972	406	343	367	189	409	49	686	375	360	**6078**
Avg. Queue Length (ft)	94	94	10	172	172	7	36	72	72	10	67	135	135	3	**50**
Max. Queue Length (ft)	265	265	232	637	637	203	243	243	243	213	394	394	394	156	**637**
Avg. Delay (sec/veh)	63.5	53.9	7.2	52.1	47.4	13.0	2.6	60.5	56.6	7.4	6.8	60.7	52.5	7.7	**40.3**
Stopped Delay (sec/veh)	48.4	41.9	2.7	37.1	35.8	6.4	0.2	51.6	47.7	2.2	2.7	49.9	42.1	3.7	**30.4**
Avg. Stops (stops/veh)	0.95	0.82	0.38	0.90	0.88	0.63	0.14	0.90	0.85	0.38	0.21	0.97	0.90	0.29	**0.74**

Table 115. U-Turn Departure Side Results: I-20 @ Hulen Base Scenario and Driveway Closure Improvement.

Measure of Effectiveness	AM Peak Hour			PM Peak Hour		
	Base	Compliance Rate		Base	Compliance Rate	
		50%	100%		50%	100%
Eastbound U-Turn Departure End (WB First Driveway Closure)						
Number of Vehicles	376	375	375	334	336	336
Avg. Queue Length (ft)	4.69	4.80	4.61	4.03	2.93	1.43
Max. Queue Length (ft)	116	139	132	175	141	96
Avg. Queue Stops (stops)	98	102	96	70	65	50
Eastbound U-Turn Departure End (WB First and Second Driveways Closure)						
Number of Vehicles	376	375	375	334	343	336
Avg. Queue Length (ft)	4.69	4.60	4.62	4.03	1.58	1.45
Max. Queue Length (ft)	116	132	132	175	98	108
Avg. Queue Stops (stops)	98	96	95	70	55	51

Restrictions: RTOR Yield to U-Turn Traffic

Simulation Results for I-20 at Hulen Site

Table 116. VISSIM Countermeasures Results Summary: I-20 @ Hulen AM Peak Hour—RTOR Yield to U-Turn Traffic.

Base Condition															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	437	28	202	160	251	173	376	382	134	243	73	293	440	718	**3909**
Avg. Queue Length (ft)	47	47	0	34	34	1	25	47	47	0	29	58	58	7	**20**
Max. Queue Length (ft)	185	185	38	125	125	72	200	200	200	27	305	305	305	260	**317**
Avg. Delay (sec/veh)	42.2	40.4	1.7	43.7	43.6	2.4	9.5	37.4	35.6	1.3	2.7	36.7	37.3	4.3	**23.4**
Stopped Delay (sec/veh)	30.6	29.6	0.0	33.2	32.1	0.6	5.6	29.7	28.0	0.0	0.2	28.8	28.4	0.8	**17.0**
Avg. Stops (stops/veh)	0.87	0.80	0.02	0.81	0.84	0.17	0.34	0.79	0.73	0.01	0.07	0.83	0.79	0.17	**0.52**
RTOR Yield to U-Turn Traffic															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	439	28	202	159	251	173	375	381	133	243	73	295	441	718	**3910**
Avg. Queue Length (ft)	47	47	0	34	34	3	25	47	47	0	29	58	58	8	**20**
Max. Queue Length (ft)	183	183	40	125	125	91	205	205	205	41	300	300	300	261	**318**
Avg. Delay (sec/veh)	42.5	40.2	2.0	41.6	43.8	5.9	9.5	37.0	35.1	1.3	2.6	37.5	37.8	4.3	**23.6**
Stopped Delay (sec/veh)	30.8	29.4	0.1	31.2	32.5	2.6	5.7	29.3	27.5	0.0	0.2	29.5	28.9	0.8	**17.1**
Avg. Stops (stops/veh)	0.87	0.79	0.04	0.78	0.82	0.45	0.34	0.78	0.72	0.02	0.06	0.84	0.79	0.17	**0.53**

Table 117. VISSIM Countermeasures Results Summary: I-20 @ Hulen PM Peak Hour—RTOR Yield to U-Turn Traffic.

Base Condition															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	625	115	337	853	969	406	334	364	189	415	50	685	375	359	**6074**
Avg. Queue Length (ft)	94	94	10	177	177	6	39	75	75	11	67	134	134	3	**51**
Max. Queue Length (ft)	280	280	184	685	685	146	252	243	243	194	392	392	392	150	**685**
Avg. Delay (sec/veh)	63.9	55.9	7.4	52.7	48.7	13.5	4.3	64.0	58.1	7.6	6.9	60.6	53.3	7.3	**41.2**
Stopped Delay (sec/veh)	48.7	43.8	3.0	37.6	36.9	6.7	0.9	55.0	49.3	2.1	2.9	49.8	42.8	3.5	**31.2**
Avg. Stops (stops/veh)	0.95	0.86	0.38	0.91	0.90	0.60	0.22	0.91	0.84	0.39	0.22	0.96	0.89	0.28	**0.75**
RTOR Yield to U-Turn Traffic															
Measure of Effectiveness	**Arterial**						**Frontage Road**								**Total**
	NB			**SB**			**EB**				**WB**				
	LT	**Th**	**RT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	**UT**	**LT**	**Th**	**RT**	
Number of Vehicles	620	113	337	856	974	407	334	366	189	414	49	683	374	359	**6077**
Avg. Queue Length (ft)	95	95	4	175	175	13	38	75	75	11	69	138	138	3	**51**
Max. Queue Length (ft)	286	286	189	668	668	405	240	239	239	185	399	399	399	155	**668**
Avg. Delay (sec/veh)	64.1	55.5	7.7	52.4	48.3	15.8	3.8	63.8	57.9	7.6	6.9	62.1	54.3	7.7	**41.4**
Stopped Delay (sec/veh)	48.9	43.5	3.0	37.4	36.5	7.9	0.6	54.8	49.2	2.1	2.9	51.0	43.6	3.8	**31.3**
Avg. Stops (stops/veh)	0.94	0.85	0.39	0.91	0.90	0.81	0.17	0.91	0.83	0.41	0.22	0.98	0.91	0.28	**0.76**

Table 118. U-Turn Departure Side Results: I-20 @ Hulen Base Scenario and RTOR Yield to U-Turn Traffic Improvement.

Measure of Effectiveness	**AM Peak Hour**		**PM Peak Hour**	
	Base	**Improvement**	**Base**	**Improvement**
Westbound U-Turn Departure End				
Number of Vehicles	73	73	50	49
Avg. Queue Length (ft)	0.14	0.09	0.23	0.2
Max. Queue Length (ft)	29	28	33	29
Avg. Queue Stops (stops)	7	5	4	4
Eastbound U-Turn Departure End				
Number of Vehicles	376	375	334	334
Avg. Queue Length (ft)	4.69	4.62	4.03	2.86
Max. Queue Length (ft)	116	126	175	126
Avg. Queue Stops (stops)	98	95	70	63

APPENDIX E. RESEARCH FOREST VOLUME DATA FOR SIGNAL TIMING ANALYSIS

Table 119. Data from Six Pines Intersection.

Time Begin	Northbound Six Pines			Southbound Six Pines			Eastbound Research Forest			Westbound Research Forest		
	L-turn	Thru	Right	L-turn	Thru	Right	Left	Thru	Right	Left	Thru	Right
7:00 AM	11	0	15	3	0	2	1	225	23	30	329	4
7:15 AM	4	3	22	0	3	2	1	235	23	32	451	3
7:30 AM	12	5	13	5	4	1	3	273	36	48	407	10
7:45 AM	26	5	31	3	2	2	4	239	40	42	541	3
8:00 AM	22	2	26	10	2	2	7	273	38	48	461	4
8:15 AM	13	4	26	6	7	7	4	303	36	43	460	5
8:30 AM	23	1	23	8	1	1	5	300	40	32	347	2
8:45 AM	13	0	25	8	4	4	6	273	26	53	359	5
4:00 PM	29	1	93	4	11	9	4	485	31	37	330	5
4:15 PM	68	5	62	12	4	3	8	394	31	38	270	6
4:30 PM	64	3	67	5	3	1	6	429	37	34	338	2
4:45 PM	50	3	55	3	4	5	3	393	41	34	361	0
5:00 PM	60	1	86	9	3	5	5	377	28	39	352	4
5:15 PM	63	3	57	4	2	4	3	392	36	54	342	3
5:30 PM	50	2	48	3	1	3	3	356	45	46	357	1
5:45 PM	51	1	41	5	8	2	9	317	38	40	314	4
AM Peak	104	20	124	40	28	28	28	1212	160	212	1840	40
PM Peak	272	20	372	48	44	36	36	1940	180	216	1444	24

Table 120. Data from Holly Hill Intersection.

Time Begin	Northbound Holly Hill			Southbound Holly Hill			Eastbound Research Forest			Westbound Research Forest		
	L-turn	Thru	Right	L-turn	Thru	Right	Left	Thru	Right	Left	Thru	Right
7:00 AM	0	1	1	6	0	8	1	223	1	2	331	6
7:15 AM	0	0	0	9	1	6	5	230	6	1	455	5
7:30 AM	0	1	0	8	2	13	2	249	0	10	429	6
7:45 AM	0	0	0	6	1	12	5	273	2	7	521	6
8:00 AM	0	0	0	11	1	14	13	256	1	6	476	4
8:15 AM	0	0	0	4	2	12	6	318	0	6	460	4
8:30 AM	0	0	0	8	1	6	4	295	5	5	366	5
8:45 AM	1	0	2	5	0	13	11	289	5	5	388	6
4:00 PM	2	3	16	7	2	5	11	513	6	7	337	8
4:15 PM	5	5	16	3	1	7	10	399	14	10	308	12
4:30 PM	5	1	19	5	3	8	7	476	6	11	345	22
4:45 PM	4	2	13	6	3	14	12	446	5	7	370	16
5:00 PM	8	7	25	4	1	11	11	390	3	5	355	17
5:15 PM	8	1	20	6	4	14	15	413	8	4	351	16
5:30 PM	8	8	12	7	0	10	22	363	8	16	369	14
5:45 PM	6	3	11	4	1	6	20	304	3	12	332	11
AM Peak	4	4	8	44	8	56	52	1272	24	40	2084	24
PM Peak	32	28	100	28	16	56	88	2052	56	64	1480	88

Table 121. Data from Pinecroft Intersection.

	Pinecroft			Pinecroft			Research Forest			Research Forest		
Time Begin	L-turn	Thru	Right	L-turn	Thru	Right	Left	Thru	Right	Left	Thru	Right
7:00 AM	10	0	15	0	0	0	1	197	30	24	329	0
7:15 AM	11	0	38	0	0	0	0	239	22	27	444	1
7:30 AM	9	0	31	0	0	1	4	208	39	45	418	2
7:45 AM	13	0	27	0	0	0	4	270	37	48	540	0
8:00 AM	15	0	36	0	1	0	2	232	41	40	486	2
8:15 AM	17	1	29	0	0	0	7	273	43	38	465	2
8:30 AM	29	1	49	1	0	0	4	225	52	72	333	1
8:45 AM	23	1	28	0	2	0	3	349	59	46	369	2
4:00 PM	46	0	92	0	0	0	2	530	27	24	295	0
4:15 PM	63	0	132	5	0	0	2	421	27	25	271	2
4:30 PM	67	0	102	1	0	0	0	490	30	20	340	0
4:45 PM	69	0	113	1	0	0	2	455	25	32	354	0
5:00 PM	70	0	121	2	0	0	0	407	6	27	316	0
5:15 PM	59	1	93	5	1	0	1	450	23	23	330	5
5:30 PM	65	2	116	0	1	0	1	413	26	39	362	1
5:45 PM	53	1	107	4	0	0	1	323	25	38	331	5
AM Peak	116	4	196	4	8			1396	236	288	2160	8
PM Peak	280	8	528	20	4			2120	120	156	1416	10

Table 122. Data from I-45—Northbound Frontage Road Intersection.

Time Begin	Northbound I-45				Eastbound Research Forest			Westbound Research Forest		
	U-Turn	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
7:00 AM	151	228	63	24	67	145	-	-	142	31
7:15 AM	155	262	71	17	127	99	-	-	119	41
7:30 AM	129	303	105	24	88	142	-	-	130	27
7:45 AM	127	323	131	19	115	136	-	-	150	29
8:00 AM	135	316	157	27	114	95	-	-	145	26
8:15 AM	127	346	151	26	102	78	-	-	103	23
8:30 AM	106	292	92	43	107	85	-	-	100	11
8:45 AM	97	239	95	28	145	93	-	-	117	8
4:00 PM	85	226	184	11	312	123	-	-	200	27
4:15 PM	87	163	215	18	294	155	-	-	183	32
4:30 PM	86	208	200	13	304	116	-	-	248	30
4:45 PM	63	209	207	11	290	146	-	-	216	21
5:00 PM	74	222	203	12	302	140	-	-	257	30
5:15 PM	67	205	218	19	292	154	-	-	201	38
5:30 PM	48	242	222	14	287	174	-	-	193	34
5:45 PM	76	205	189	12	272	172	-	-	178	19
AM Peak	620	1292	628	172	580	580			600	116
PM Peak	348	968	888	76	1248	696			1028	152

Table 123. Data from I-45—Southbound Frontage Road Intersection.

Time Begin	Southbound I-45				Eastbound Research Forest			Westbound Research Forest		
	U-Turn	Left	Thru	Right	Left	Thru	Right	Left	Thru	Right
7:00 AM	49	121	114	87	-	86	80	100	237	-
7:15 AM	53	128	132	93	-	81	75	86	264	-
7:30 AM	62	98	152	142	-	92	69	78	296	-
7:45 AM	78	93	165	160	-	105	77	73	331	-
8:00 AM	83	81	175	168	-	120	82	70	359	-
8:15 AM	86	75	163	148	-	113	73	74	348	-
8:30 AM	86	67	149	133	-	119	65	69	327	-
8:45 AM	86	56	137	135	-	144	77	65	288	-
4:00 PM	118	80	127	84	-	348	198	116	249	-
4:15 PM	137	115	142	98	-	306	175	126	210	-
4:30 PM	125	109	134	96	-	315	207	106	252	-
4:45 PM	145	118	110	128	-	303	220	87	271	-
5:00 PM	150	154	137	87	-	272	143	113	282	-
5:15 PM	141	100	118	109	-	380	97	87	256	-
5:30 PM	138	106	129	72	-	318	154	96	291	-
5:45 PM	110	140	164	98	-	303	105	87	219	-
AM Peak	344	512	700	672		576	328	400	1436	
PM Peak	600	616	656	512		1520	880	504	1164	

Table 124. Data from David Memorial Intersection.

Time Begin	Northbound David Memorial			Southbound David Memorial			Eastbound Tamina			Westbound Tamina		
	L-turn	Thru	Right	L-turn	Thru	Right	Left	Thru	Right	Left	Thru	Right
7:00 AM	45	16	11	0	3	29	10	59	75	11	63	4
7:15 AM	44	6	14	3	3	32	10	55	42	10	80	2
7:30 AM	66	16	15	2	5	42	14	52	88	5	82	7
7:45 AM	43	8	4	5	4	43	20	51	80	2	107	10
8:00 AM	24	5	4	2	4	37	31	48	33	1	65	9
8:15 AM	14	10	3	5	5	34	31	36	17	3	74	8
8:30 AM	16	5	0	3	5	32	62	49	12	2	45	10
8:45 AM	19	5	0	0	3	33	44	47	36	4	58	5
4:00 PM	60	20	4	17	11	99	27	57	38	3	46	9
4:15 PM	28	25	8	11	17	90	40	77	48	4	42	3
4:30 PM	30	27	7	14	21	102	37	57	32	4	59	7
4:45 PM	11	22	7	13	23	88	29	75	41	4	49	7
5:00 PM	24	21	12	11	20	119	32	73	69	5	69	15
5:15 PM	21	31	9	12	21	111	35	77	56	6	65	15
5:30 PM	26	22	6	21	25	96	43	98	51	3	55	7
5:45 PM	20	13	1	13	25	66	21	72	81	1	52	6
AM Peak	264	64	60	20	20	172	248	236	320	24	428	40
PM Peak	240	108	48	84	100	476	172	392	324	24	276	60

APPENDIX F. DESCRIPTION OF VARIABLES USED IN SAFETY ANALYSIS

This appendix identifies the individual variables included in the project database in Task 5 of Project 0-6894.

Table 125. Variable Descriptions.

Variable Name	Description	Units/Options
Unique_ID	Unique ID given to each half site.	
Latitude	Geographic coordinates of the point.	Decimal Degrees
Longitude	Geographic coordinates of the point.	Decimal Degrees
District	Number of district based on TxDOT.	
County	Number of county according to TxDOT.	
Direction	Direction of travel at the first leg of the U-turn.	NB, SB, EB, WB
U-turn	If there is a U-turn in the interchange (1 = yes, 0 = no).	1 = yes 0 = no
Configuration	Is the intersection a regular diamond interchange or an X interchange?	
U-turn OverBridge?	Does U-turn take place over the bridge or under the bridge (1 = over, 0 = under).	1 = over 0 = under
Sample_Name	The sample to which the intersection belongs.	
Posted Speed	Posted speed on the frontage road.	mph
Interior Spacing	Distance from one stop line back to opposing-direction stop line across the freeway.	ft
UTurnLaneWidth	Width of the U-turn lane at the middle of the U-turn.	
AvailableUturnStorage	Available length of the U-turn that can be used to accommodate queue.	ft
IntersectionSkewed	Is the intersection skewed?	1 = yes 0 = no
SkewAngle	Interior skew angle in degrees. An intersection is not skewed if the interior angle is 90° ± 10°.	Degrees
DivergingOption	How did the first leg get divided from the roadway?	1 = shared lane 2 = exclusive lane 3 = deceleration lane
Leg1LaneWidth	Width of the approach leg of the U-turn.	ft
R1	Turning radius of the first leg in feet. This dimension shall be measured at the smallest radius.	ft
Leg1TotalLanes	Total number of lanes at the U-turn approach.	
Leg1Lane1	Choose the movement that describes the function of Lane 1 in the first approach in the best way.	
Leg1Lane2	Choose the movement that describes the function of Lane 2 in the first approach in the best way.	

Table 125. Variable Descriptions (Continued).

Variable Name	Description	Units/Options
Leg1Lane3	Choose the movement that describes the function of Lane 3 in the first approach in the best way.	
Leg1Lane4	Choose the movement that describes the function of Lane 4 in the first approach in the best way.	
Leg1Lane5	Choose the movement that describes the function of Lane 5 in the first approach in the best way.	
Leg1Lane6	Choose the movement that describes the function of Lane 6 in the first approach in the best way.	
Leg1Lane7	Choose the movement that describes the function of Lane 7 in the first approach in the best way.	
Leg1Lane8	Choose the movement that describes the function of Lane 8 in the first approach in the best way.	
Leg1DivergingLength	Length of the diverging leg in presence of a diverging lane.	ft
Leg1baylength	Distance from merging gore point to the point that the extension of the pavement marking cuts the road pavement marking.	ft
Merging Option	How did the first leg merge onto the roadway?	1 = shared lane 2 = exclusive lane 3 = acceleration lane
Leg2LaneWidth	Width of the merging (second) leg of the U-turn.	ft
R2	Turning radius of the second leg in feet. This dimension shall be measured at the smallest radius.	ft
Leg2TotalLanes	Total number of through lanes at the second leg of the U-turn.	
Leg2Lane1	Choose the movement that describes the function of Lane 1 in the second approach in the best way.	
Leg2Lane2	Choose the movement that describes the function of Lane 2 in the second approach in the best way.	
Leg2Lane3	Choose the movement that describes the function of Lane 3 in the second approach in the best way.	
Leg2Lane4	Choose the movement that describes the function of Lane 4 in the second approach in the best way.	
Leg2Lane5	Choose the movement that describes the function of Lane 5 in the second approach in the best way.	
Leg2Lane6	Choose the movement that describes the function of Lane 6 in the second approach in the best way.	
Leg2Lane7	Choose the movement that describes the function of Lane 7 in the second approach in the best way.	
Leg2Lane8	Choose the movement that describes the function of Lane 8 in the second approach in the best way.	

Table 125. Variable Descriptions (Continued).

Variable Name	Description	Units/Options
Leg2MergingLength	Length of the merging leg in presence of a merging lane.	ft
Leg2BayLength	Distance from merging gore point to the point that the extension of the pavement marking cuts the road pavement marking.	
IntersectionControl	If the intersection is signalized or not.	
NumConflictLT	Number of conflicting left-turn lanes.	
NumConflictT	Number of conflicting through lanes.	
NumConflictRT	Number of conflicting right-turn lanes.	
RTEntTreatment	Cross-street right-turn entrance treatment.	
RTExitTreatment	Cross-street right-turn exit treatment.	
RTwithExclusiveLane	Does arterial right-turn share a lane on the frontage road? (0) or does the right turn have an additional lane on the frontage road (1)?	
CSNoLanes	Total number of through lanes at the cross street.	
CSLeg1Lane1	Choose the movement that describes the function of Lane 1 in the cross street in the best way.	
CSLeg1Lane2	Choose the movement that describes the function of Lane 2 in the cross street in the best way.	
CSLeg1Lane3	Choose the movement that describes the function of Lane 3 in the cross street in the best way.	
CSLeg1Lane4	Choose the movement that describes the function of Lane 4 in the cross street in the best way.	
CSLeg1Lane5	Choose the movement that describes the function of Lane 5 in the cross street in the best way.	
CSLeg1Lane6	Choose the movement that describes the function of Lane 6 in the cross street in the best way.	
CSLeg1Lane7	Choose the movement that describes the function of Lane 7 in the cross street in the best way.	
CSLeg1Lane8	Choose the movement that describes the function of Lane 8 in the cross street in the best way.	
FirstDrivewayDistance	Distance from the gore point of the merging leg of the U-turn to the first driveway.	ft
Comments	Any extra comment on the intersection.	

APPENDIX G. SUMMARY OF CRASH DATA FOR OPERATIONAL STUDY SITES

This appendix contains the summaries of the site characteristics and crash data used to conduct the crash analysis in Task 5 of Project 0-6894.

SITE #1 INFORMATION (SITE ID: 6894_1)

Table 126. Site #1—Summary of Site Conditions.

Site Information	Value
District	Abilene
County	Taylor (221)
City	Abilene (2)
Road #1	I-20
Road #2	SH 351 (E Amber Avenue)
SB Frontage Road	E. Stamford
WB Frontage Road	E. Overland Trail
Direction (Road #1)	E/W
Latitude, Longitude	32.477309, -99.699588
U-turn Present	Yes (Visible on 11/8/2014 aerials, from 12/9/2012 and earlier the U-turns were not constructed)
Comments	Removed 2013–2014 crash data from analysis

Table 127. Site #1—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	1	3	10	14	2	29
2010	0	0	3	7	16	0	26
2011	0	0	1	6	21	0	28
2012	0	0	1	7	15	0	23
2013	U-Turn construction sometime between 12/9/2012 and 11/8/2014						
2014							
2015	0	0	2	5	26	1	34

Table 128. Site #1—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2012 (No U-Turn)	106	49	46.2%	57	53.8%
2015 (U-Turn Present)	34	10	29.4%	24	70.6%

Table 129. Site #1—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2012 (No U-Turn)	49	2	4.1%	47	95.9%
2015 (U-Turn Present)	10	0	0%	10	100%

SITE #2 INFORMATION (SITE ID: 6894_2)

Table 130. Site #2—Summary of Site Conditions.

Site Information	Value
District	Bryan
County	Brazos (21)
City	Bryan (55)
Road #1	SH 6
Road #2	Boonville Rd. (east side) E William J Bryan Pkwy (west side)
NB Frontage Road	Texas 6 Frontage Rd. (for N Earl Rudder Fwy)
SB Frontage Road	Texas 6 Frontage Rd. (for N Earl Rudder Fwy)
Direction (Road #1)	N/S
Latitude, Longitude	30.67207685, -96.33817638
U-Turn Present	No

Table 131. Site #2—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	0	1	4	12	0	17
2010	0	1	1	5	27	0	34
2011	0	0	1	5	30	0	36
2012	0	0	0	3	13	0	16
2013	0	0	2	4	21	0	27
2014	0	2	3	5	32	0	42
2015	0	0	5	8	32	0	45

Table 132. Site #2—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (No U-Turn)	217	47	21.7%	170	78.3%

Table 133. Site #2—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (No U-Turn)	47	20	42.6%	27	57.4%

SITE #3 INFORMATION (SITE ID: 6894_3)

Table 134. Site #3—Summary of Site Conditions.

Site Information	Value
District	Bryan
County	Brazos (21)
City	Bryan (55)
Road #1	SH 6
Road #2	Briarcrest
NB Frontage Road	Texas 6 Frontage Rd. (for N Earl Rudder Fwy)
SB Frontage Road	Texas 6 Frontage Rd. (for N Earl Rudder Fwy)
Direction (Road #1)	N/S
Latitude, Longitude	30.663284, -96.327422
U-Turn Present	No

Table 135. Site #3—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	0	6	4	40	0	50
2010	0	0	1	6	41	0	48
2011	0	0	2	9	24	0	35
2012	0	1	0	7	28	0	36
2013	0	0	6	9	45	0	60
2014	0	2	7	5	35	0	49
2015	0	0	2	7	49	0	58

Table 136. Site #3—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (No U-Turn)	336	66	19.6%	270	80.4%

Table 137. Site #3—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (No U-Turn)	66	40	60.6%	26	39.4%

SITE #4 INFORMATION (SITE ID: 6894_4)

Table 138. Site #4—Summary of Site Conditions.

Site Information	Value
District	Bryan
County	Brazos (21)
City	College Station (85)
Road #1	SH 6
Road #2	University Dr.
NB Frontage Road	Texas 6 Frontage Rd. (for N Earl Rudder Fwy)
SB Frontage Road	Texas 6 Frontage Rd. (for N Earl Rudder Fwy)
Direction (Road #1)	N/S
Latitude, Longitude	30.640582, -96.310203
U-Turn Present	No

Table 139. Site #4—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	0	7	5	27	0	39
2010	0	0	4	5	24	1	34
2011	0	0	2	9	15	0	26
2012	0	0	1	6	13	0	20
2013	0	0	7	7	14	0	28
2014	0	0	9	3	10	0	22
2015	0	0	5	4	17	0	26

Table 140. Site #4—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (No U-Turn)	195	22	11.3%	173	54.5%

Table 141. Site #4—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (No U-Turn)	22	10	45.5%	12	54.5%

SITE #5 INFORMATION (SITE ID: 6894_5)

Table 142. Site #5—Summary of Site Conditions.

Site Information	Value
District	Bryan
County	Brazos (21)
City	College Station (85)
Road #1	SH 6
Road #2	Rock Prairie Drive
NB Frontage Road	Texas 6 Frontage Rd. (for N Earl Rudder Fwy)
SB Frontage Road	Texas 6 Frontage Rd. (for N Earl Rudder Fwy)
Direction (Road #1)	N/S
Latitude, Longitude	30.584875, -96.284949
U-Turn Present	Yes (Visible on 11/2016 aerials, from 2/2013 and earlier the U-turns were not constructed)
Comments	Remove 2013-2015 crash data from analysis. For qualitative analysis, **treat this location as a "No" U-turn condition**.

Table 143. Site #5—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	0	0	2	3	0	5
2010	0	0	0	2	11	0	13
2011	0	0	0	2	6	0	8
2012	0	1	0	2	2	0	5
2013	U-turn under construction						
2014							
2015							

Table 144. Site #5—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2012 (No U-Turn)	31	13	41.9%	18	58.1%

Table 145. Site #5—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2012 (No U-Turn)	13	3	23.1%	10	76.9%

SITE #6 INFORMATION (SITE ID: 6894_6)

Table 146. Site #6—Summary of Site Conditions.

Site Information	Value
District	Bryan
County	Brazos (21)
City	College Station (85)
Road #1	SH 6
Road #2	William D Fitch (SH 40)
NB Frontage Road	Texas 6 Frontage Rd. (for N Earl Rudder Fwy)
SB Frontage Road	Texas 6 Frontage Rd. (for N Earl Rudder Fwy)
Direction (Road #1)	N/S
Latitude, Longitude	30.559353, -96.25784
U-Turn Present	Yes

Table 147. Site #6—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	0	0	3	9	0	12
2010	0	0	1	0	6	0	7
2011	0	0	2	1	5	0	8
2012	0	2	0	1	10	0	13
2013	0	0	6	1	9	1	17
2014	0	0	2	2	6	0	10
2015	0	0	1	2	7	1	11

Table 148. Site #6—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (U-Turn Present)	78	19	24.4%	59	75.6%

Table 149. Site #6—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (U-Turn Present)	19	13	68.4%	6	31.6%

SITE #7 INFORMATION (SITE ID: 6894_7)

Table 150. Site #7—Summary of Site Conditions.

Site Information	Value
District	Bryan
County	Washington (239)
City	Brenham (48)
Road #1	State Hwy 290 E
Road #2	SH 36
EB Frontage Road	Feeder Rd.
WB Frontage Road	Feeder Rd.
Direction (Road #1)	E/W
Latitude, Longitude	30.142583, -96.396075
U-Turn Present	Prior to 1/2011 there were not any U-turns. As of 4/2012, U-turns were open.
Comments	Remove 2011-2012 crash data for analysis. For qualitative analysis, consider as a before/after condition.

Table 151. Site #7—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	0	5	2	31	0	38
2010	0	0	5	8	40	0	53
2011	U-turn under construction						
2012							
2013	0	0	3	5	28	0	36
2014	0	0	6	2	46	0	54
2015	0	0	4	5	35	0	44

Table 152. Site #7—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2010 (No U-Turn)	91	25	27.5%	66	72.5%
2013–2015 (U-Turn Present)	134	24	17.9%	110	82.1%

Table 153. Site #7—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2010 (No U-Turn)	25	5	20%	20	80%
2013–2015 (U-Turn Present)	24	10	41.7%	14	58.3%

SITE #8 INFORMATION (SITE ID: 6894_8)

Table 154. Site #8—Summary of Site Conditions.

Site Information	Value
District	Corpus Christi
County	Nueces (178)
City	Corpus Christi (97)
Road #1	SH 358 (Padre Island Drive)
Road #2	Greenwood Dr. (SH 286)
EB Frontage Road	S Padre Island Dr.
WB Frontage Road	S Padre Island Dr.
Direction (Road #1)	E/W
Latitude, Longitude	27.742545, -97.441578
U-Turn Present	Yes, as of 10/2008, a U-turn is present on both sides.

Table 155. Site #8—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	0	0	6	23	0	29
2010	0	0	2	6	23	1	32
2011	0	1	4	4	29	0	38
2012	0	0	3	5	19	0	27
2013	0	0	1	2	12	0	15
2014	0	0	1	0	6	0	7
2015	0	0	0	1	10	1	12

Table 156. Site #8—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (U-Turn Present)	160	61	38.1%	99	61.9%

Table 157. Site #8—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (U-Turn Present)	61	4	6.6%	57	93.4%

SITE #9 INFORMATION (SITE ID: 6894_9)

Table 158. Site #9—Summary of Site Conditions.

Site Information	Value
District	El Paso
County	El Paso (71)
City	Socorro (403) / Rural El Paso County (1635)
Road #1	I-10
Road #2	FM 1281 (Horizon Blvd)
NB Frontage Road	Gateway Blvd. W
SB Frontage Road	Gateway Blvd. E
Direction (Road #1)	N/S
Latitude, Longitude	31.659729, -106.239883
U-Turn Present	Yes (Visible on 6/2010 aerials, from 10/2008 and earlier the U-turns were not constructed)
Comments	Remove crash data from 2010 or earlier from analysis. Treat remaining as a U-turn condition.

Table 159. Site #9—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	U-turn under construction						
2010							
2011	0	3	1	6	29	2	41
2012	0	0	0	8	23	0	31
2013	0	0	3	2	18	0	23
2014	0	0	1	8	27	1	37
2015	0	0	4	9	42	0	55

Table 160. Site #9—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2011–2015 (U-Turn Present)	187	55	29.4%	132	70.6%

Table 161. Site #9—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2011–2015 (U-Turn Present)	55	3	5.5%	52	94.5%

SITE #10 INFORMATION (SITE ID: 6894_10)

Table 162. Site #10—Summary of Site Conditions.

Site Information	Value
District	Fort Worth
County	Tarrant (220)
City	Burleson (59)
Road #1	I-35 W
Road #2	Alsbury Blvd. / E Alsbury Blvd. / NE Alsbury Blvd.
NB Frontage Road	South Fwy
SB Frontage Road	South Fwy
Direction (Road #1)	N/S
Latitude, Longitude	32.563121, -97.318871
U-Turn Present	No

Table 163. Site #10—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	0	0	1	3	0	4
2010	0	0	0	0	1	0	1
2011	0	0	1	0	0	0	1
2012	0	0	0	0	2	0	2
2013	0	0	1	0	2	0	3
2014	1	0	0	0	6	0	7
2015	0	1	1	1	5	0	8

Table 164. Site #10—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (No U-Turn)	26	4	15.4%	22	84.6%

Table 165. Site #10—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (No U-Turn)	4	2	50%	2	50%

SITE #11 INFORMATION (SITE ID: 6894_11)

Table 166. Site #11—Summary of Site Conditions.

Site Information	Value
District	Fort Worth
County	Tarrant (220)
City	Fort Worth (156)
Road #1	I-35W
Road #2	FM 1187 (E Rendon Crowley Rd. / W Rendon Crowley Rd.)
NB Frontage Road	South Fwy
SB Frontage Road	South Fwy
Direction (Road #1)	N/S
Latitude, Longitude	32.577726, -97.319589
U-Turn Present	Yes, on North side only (Visible on 12/2009 aerials, from 10/2008 and earlier, the U-turn was not constructed)
Comments	Remove 2009 crash data for analysis. Treat remaining as a one-side-only U-turn.

Table 167. Site #11—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	U-turn under construction						
2010	0	2	3	5	19	0	29
2011	0	1	2	8	24	0	35
2012	0	1	1	12	24	0	38
2013	0	0	3	11	30	1	45
2014	1	0	3	12	30	0	46
2015	0	2	4	14	32	0	52

Table 168. Site #11—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2010–2015 (U-Turn present on one side only)	245	46	18.8%	199	81.2%

Table 169. Site #11—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2010–2015 (U-Turn present on one side only)	46	16	34.8%	30	65.2%

SITE #12 INFORMATION (SITE ID: 6894_12)

Table 170. Site #12—Summary of Site Conditions.

Site Information	Value
District	Fort Worth
County	Tarrant (220)
City	Fort Worth (156)
Road #1	I-20
Road #2	McCart Ave.
EB Frontage Road	SW Loop 820
WB Frontage Road	SW Loop 820
U-Turn Name	SW Loop 820 Service Rd.
Direction (Road #1)	E/W
Latitude, Longitude	32.668105, -97.355975
U-Turn Present	Yes, on west side only

Table 171. Site #12—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	1	2	6	20	3	32
2010	0	1	2	8	18	0	29
2011	0	0	1	5	12	0	18
2012	0	1	2	8	15	0	26
2013	0	1	5	8	25	1	40
2014	0	2	6	21	30	3	62
2015	0	1	0	3	29	1	34

Table 172. Site #12—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (U-Turn present on one side only)	241	45	18.7%	196	81.3%

Table 173. Site #12—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (U-Turn present on one side only)	45	20	44.4%	25	55.6%

SITE #13 INFORMATION (SITE ID: 6894_13)

Table 174. Site #13—Summary of Site Conditions.

Site Information	Value
District	Fort Worth
County	Tarrant (220)
City	Fort Worth (156)
Road #1	I-20 / I-820
Road #2	S. Hulen St.
EB Frontage Road	SW Loop 820
WB Frontage Road	SW Loop 820
Direction (Road #1)	E/W
Latitude, Longitude	32.680751, -97.393145
U-Turn Present	Yes

Table 175. Site #13—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	1	7	14	40	0	62
2010	0	0	0	5	9	0	14
2011	0	0	2	6	12	0	20
2012	0	1	2	8	16	0	27
2013	0	0	5	11	23	0	39
2014	0	0	0	9	15	0	24
2015	0	1	4	7	12	0	24

Table 176. Site #13—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (U-Turn Present)	210	22	10.5%	188	89.5%

Table 177. Site #13—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (U-Turn Present)	22	9	40.9%	13	59.1%

SITE #14 INFORMATION (SITE ID: 6894_14)

Table 178. Site #14—Summary of Site Conditions.

Site Information	Value
District	Houston
County	Harris (101)
City	Houston (208)
Road #1	I-10
Road #2	Gessner Rd.
EB Frontage Road	Interstate 10 Frontage Rd.
WB Frontage Road	Old Katy Rd.
Direction (Road #1)	E/W
Latitude, Longitude	29.784472, -95.543989
U-Turn Present	Yes
Comments	Based on aerial photograph of 1/2009, construction was completed for the freeway widening, so use crash data for entire period in analysis.

Table 179. Site #14—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	1	5	6	19	3	34
2010	0	0	3	11	17	1	32
2011	0	0	5	9	28	0	42
2012	0	0	5	9	34	1	49
2013	0	1	7	13	37	3	61
2014	0	0	4	21	41	3	69
2015	1	1	5	16	48	2	73

Table 180. Site #14—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (U-Turn Present)	360	61	16.9%	299	83.1%

Table 181. Site #14—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (U-Turn Present)	61	45	73.8%	16	26.2%

SITE #15 INFORMATION (SITE ID: 6894_15)

Table 182. Site #15—Summary of Site Conditions.

Site Information	Value
District	Houston
County	Harris (101)
City	Houston (208)
Road #1	I-10
Road #2	Bunker Hill Rd.
EB Frontage Road	Interstate 10 Frontage Rd.
WB Frontage Road	Old Katy Rd.
Direction (Road #1)	E/W
Latitude, Longitude	29.78446, -95.531792
U-Turn Present	Yes
Comments	Based on aerial photograph of 1/2009, construction was completed for the freeway widening, so use crash data for entire period in analysis.

Table 183. Site #15—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	0	1	1	6	0	8
2010	0	0	0	5	9	0	14
2011	0	0	4	6	16	0	26
2012	0	1	2	5	13	0	21
2013	0	0	0	12	19	1	32
2014	0	0	1	6	20	0	27
2015	0	0	1	8	30	0	39

Table 184. Site #15—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (U-Turn Present)	167	32	19.2%	135	80.8%

Table 185. Site #15—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (U-Turn Present)	32	26	81.3%	6	18.8%

SITE #16 INFORMATION (SITE ID: 6894_16)

Table 186. Site #16—Summary of Site Conditions.

Site Information	Value
District	Houston
County	Montgomery (170)
City	Shenandoah (1323)
Road #1	I-45
Road #2	Research Forest Dr. /Tamina Rd.
NB Frontage Road	N Fwy Service Rd.
SB Frontage Road	N Fwy Service Rd.
Direction (Road #1)	N/S
Latitude, Longitude	30.178275, -95.451811
U-Turn Present	Yes

Table 187. Site #16—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	0	2	11	60	0	73
2010	0	0	6	1	42	0	49
2011	0	0	1	5	21	0	27
2012	0	1	4	1	24	0	30
2013	0	0	2	1	32	0	35
2014	0	1	0	4	47	0	52
2015	0	0	4	4	52	0	60

Table 188. Site #16—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (U-Turn Present)	326	76	23.3%	250	76.7%

Table 189. Site #16—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (U-Turn Present)	76	72	94.7%	4	5.3%

SITE #17 INFORMATION (SITE ID: 6894_17)

Table 190. Site #17—Summary of Site Conditions.

Site Information	Value
District	Houston
County	Montgomery (170)
City	Rural Montgomery County (1735)
Road #1	I-45
Road #2	Rayford Rd. / Sawdust Rd.
NB Frontage Road	N Fwy Service Rd.
SB Frontage Road	N Fwy Service Rd.
Direction (Road #1)	N/S
Latitude, Longitude	30.126738, -95.443177
U-Turn Present	Yes

Table 191. Site #17—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	1	5	9	57	0	72
2010	0	3	2	10	53	0	68
2011	1	1	6	11	46	0	65
2012	0	1	1	9	47	0	58
2013	0	1	3	10	58	0	72
2014	0	1	6	7	58	0	72
2015	0	0	4	12	73	3	92

Table 192. Site #17—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (U-Turn Present)	499	108	21.6%	391	78.4%

Table 193. Site #17—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (U-Turn Present)	108	84	77.8%	24	22.2%

SITE #18 INFORMATION (SITE ID: 6894_18)

Table 194. Site #18—Summary of Site Conditions.

Site Information	Value
District	Laredo
County	Webb (240)
City	Laredo (254)
Road #1	I-35
Road #2	W Mann Rd. / E Mann Rd.
NB Frontage Road	San Dario Ave.
SB Frontage Road	San Bernardo Ave.
Direction (Road #1)	N/S
Latitude, Longitude	27.556488, -99.503814
U-Turn Present	Yes

Table 195. Site #18—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	1	0	5	20	1	27
2010	0	0	0	0	23	0	23
2011	0	0	2	3	22	0	27
2012	0	0	0	5	24	0	29
2013	0	0	1	9	22	0	32
2014	0	0	0	5	25	0	30
2015	0	0	0	6	24	2	32

Table 196. Site #18—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (U-Turn Present)	200	53	26.5%	147	73.5%

Table 197. Site #18—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (U-Turn Present)	53	0	0%	53	100%

SITE #19 INFORMATION (SITE ID: 6894_19)

Table 198. Site #19—Summary of Site Conditions.

Site Information	Value
District	Pharr
County	Hidalgo (108)
City	McAllen (283)
Road #1	I-2
Road #2	FM 2220 (S Ware Rd.)
EB Frontage Road	W Expy 83 / E Frontage Rd.
WB Frontage Road	W Expy 83 / W Frontage Rd.
Direction (Road #1)	E/W
Latitude, Longitude	26.194732, -98.263662
U-Turn Present	Yes

Table 199. Site #19—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	0	4	21	16	0	41
2010	0	0	3	27	19	0	49
2011	0	0	1	18	16	1	36
2012	0	0	7	22	18	0	47
2013	0	0	10	19	9	0	38
2014	0	1	8	37	19	0	65
2015	0	0	3	23	16	1	43

Table 200. Site #19—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (U-Turn Present)	319	95	29.8%	224	70.2%

Table 201. Site #19—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (U-Turn Present)	95	46	48.4%	49	51.6%

SITE #20 INFORMATION (SITE ID: 6894_20)

Table 202. Site #20—Summary of Site Conditions.

Site Information	Value
District	Pharr
County	Hidalgo (108)
City	Mission (295)
Road #1	I-2
Road #2	FM 494 (S Shary Rd.)
EB Frontage Road	E Frontage Rd./E Expy 83
WB Frontage Road	W Frontage Rd./E Expy 83
Direction (Road #1)	E/W
Latitude, Longitude	26.195849, -98.288392
U-Turn Present	Yes

Table 203. Site #20—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	0	1	11	31	2	45
2010	0	0	3	8	78	1	90
2011	0	0	2	16	75	2	95
2012	0	0	0	20	95	1	116
2013	0	0	1	8	98	5	112
2014	0	0	1	12	139	11	163
2015	0	0	1	12	166	9	188

Table 204. Site #20—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (U-Turn Present)	809	136	16.8%	673	83.2%

Table 205. Site #20—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (U-Turn Present)	136	60	44.1%	76	55.9%

SITE #21 INFORMATION (SITE ID: 6894_21)—REMOVED FROM SAFETY ANALYSIS (ATYPICAL CONFIGURATION)

Table 206. Site #21—Summary of Site Conditions.

Site Information	Value
District	San Angelo
County	Tom Green (226)
City	San Angelo (378)
Road #1	SH 306 (W Houston Harte Expy.)
Road #2	US 67/Sherwood/TX 306 Loop
NB Frontage Road	N/A
SB Frontage Road	N/A
Direction (Road #1)	N/S
Latitude, Longitude	31.430981, -100.506442
U-Turn Present	Yes, but not the conventional configuration as at the other study sites
Comments	The road orientation is similar to a partial cloverleaf, and the U-turns are not traditional, so it is not practical to conduct an additional comparative analysis because it is not similar to the other sites.

Table 207. Site #21—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	Crash analysis not conducted due to atypical interchange configuration						
2010							
2011							
2012							
2013							
2014							
2015							

SITE #22 INFORMATION (SITE ID: 6894_22)

Table 208. Site #22—Summary of Site Conditions.

Site Information	Value
District	San Antonio
County	Bexar (15)
City	San Antonio (379)
Road #1	I-410 / Loop 410 (Connally Loop)
Road #2	Callaghan Rd.
EB Frontage Road	I-410 Access Rd./NW Loop 410
WB Frontage Road	I-410 Access Rd./NW Loop 410
Direction (Road #1)	E/W
Latitude, Longitude	29.489556, -98.5742
U-Turn Present	Yes

Table 209. Site #22—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	1	0	13	34	0	48
2010	0	1	3	13	25	0	42
2011	0	0	2	9	12	0	23
2012	0	0	6	7	27	0	40
2013	0	0	3	6	20	0	29
2014	0	0	3	11	29	0	43
2015	0	2	0	11	28	0	41

Table 210. Site #22—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (U-Turn Present)	266	38	14.3%	228	85.7%

Table 211. Site #22—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (U-Turn Present)	38	17	44.7%	21	55.3%

SITE #23 INFORMATION (SITE ID: 6894_23)

Table 212. Site #23—Summary of Site Conditions.

Site Information	Value
District	San Antonio
County	Bexar (15)
City	San Antonio (379)
Road #1	I-410 / Loop 410
Road #2	Ingram Rd.
NB Frontage Road	I-410 Access Rd./NW Loop 410
SB Frontage Road	I-410 Access Rd./NW Loop 410
Direction (Road #1)	N/S
Latitude, Longitude	29.466083, -98.618929
U-Turn Present	Yes

Table 213. Site #23—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	0	8	29	86	0	123
2010	0	2	1	14	51	0	68
2011	0	0	4	14	53	1	72
2012	0	2	6	19	62	1	90
2013	0	1	1	15	53	2	72
2014	1	1	5	20	68	1	96
2015	0	3	7	22	84	0	116

Table 214. Site #23—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (U-Turn Present)	637	121	19%	516	81%

Table 215. Site #23—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (U-Turn Present)	121	56	46.3%	65	53.7%

SITE #24 INFORMATION (SITE ID: 6894_24)

Table 216. Site #24—Summary of Site Conditions.

Site Information	Value
District	Waco
County	Hill (109)
City	Hillsboro (202)
Road #1	I-35
Road #2	FM 286 / Old Brandon Rd./Country Club Rd.
NB Frontage Road	S Interstate Hwy 35
SB Frontage Road	S Interstate Hwy 35
Direction (Road #1)	N/S
Latitude, Longitude	32.017068, -97.095662
U-Turn Present	Yes

Table 217. Site #24—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	0	0	0	1	0	1
2010	0	0	0	0	1	0	1
2011	0	0	0	0	5	0	5
2012	0	0	0	1	1	0	2
2013	0	0	0	1	3	0	4
2014	0	0	0	0	3	0	3
2015	0	0	0	0	6	0	6

Table 218. Site #24—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (U-Turn Present)	22	4	18.2%	18	81.8%

Table 219. Site #24—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (U-Turn Present)	4	4	100%	0	0%

SITE #25 INFORMATION (SITE ID: 6894_25)

Table 220. Site #25—Summary of Site Conditions.

Site Information	Value
District	Wichita Falls
County	Wichita (243)
City	Wichita Falls (459)
Road #1	US 82/US 277
Road #2	Kemp Blvd.
EB Frontage Road	Kell E Blvd.
WB Frontage Road	Kell W Blvd.
Direction (Road #1)	E/W
Latitude, Longitude	33.885571, -98.528158
U-Turn Present	Yes

Table 221. Site #25—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	0	1	5	16	0	22
2010	0	0	0	3	6	0	9
2011	0	0	0	3	16	0	19
2012	0	0	2	1	18	0	21
2013	0	0	1	4	18	0	23
2014	0	0	3	1	15	0	19
2015	0	0	1	5	25	0	31

Table 222. Site #25—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (U-Turn Present)	144	27	18.8%	117	81.3%

Table 223. Site #25—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (U-Turn Present)	27	6	22.2%	21	77.8%

SITE #26 INFORMATION (SITE ID: 6894_26)

Table 224. Site #26—Summary of Site Conditions.

Site Information	Value
District	Wichita Falls
County	Wichita (243)
City	Wichita Falls (459)
Road #1	US 82/US 277
Road #2	Lawrence Rd./Lebanon Rd.
EB Frontage Road	Kell E Blvd.
WB Frontage Road	Kell W Blvd.
Direction (Road #1)	E/W
Latitude, Longitude	33.880864, -98.540828
U-Turn Present	Yes

Table 225. Site #26—Summary of Crash Severity.

Year	Number of Crashes						
	K	A	B	C	O	Unknown	Total
2009	0	0	2	0	3	0	5
2010	0	0	0	0	6	0	6
2011	0	0	3	2	11	0	16
2012	0	0	0	6	18	0	24
2013	0	1	1	5	17	0	24
2014	0	0	1	2	24	0	27
2015	0	0	0	2	17	0	19

Table 226. Site #26—Summary of Left-Turn Crashes.

Time Period	Total Crashes	Crashes Involving Left Turns		All Other Crashes	
		Number	Percent	Number	Percent
2009–2015 (U-Turn Present)	121	22	18.2%	99	81.8%

Table 227. Site #26—Road Where Vehicle Maneuvers Involved in Left-Turn Crashes Initiated.

Time Period	Crashes Involving Left-Turns	Left-Turn Crashes from Frontage Rd.		Left-Turn Crashes from Cross Street	
		Number of Left-Turn Crashes	Percent of Left-Turn Crashes	Number of Other Left-Turn Crashes	Percent of Other Left-Turn Crashes
2009–2015 (U-Turn Present)	22	7	31.8%	15	68.2%

APPENDIX H. SUPPLEMENTAL STATISTICAL ANALYSIS

The statistical analysis included several iterations before the models converged on the optimal configuration. This appendix summarizes some of the milestone modeling steps considered during the statistical analysis process. Chapter 5 of this report contains the final non-freeway total crash model and the non-freeway KAB model

Because the prevention of injury crashes is a critical objective for safety assessments, researchers explored various configurations that could directly influence how the study locations and their respective configurations directly influenced injury crashes (defined for the purposes of this analysis as KAB crashes).

KAB PROPORTIONAL MODELS

This severity analysis included two general categories: (a) safety effects considering all sites (including locations with and without U-turns), and (b) safety effects considering only signalized intersections with U-turns. The resulting generalized linear mixed model considered 147 sites with a total of 981 site periods (see Table 228). Note that the highlighted cell represents a variable that is significant at the 5 percent level.

Table 228. Proportion of KAB Left-Turn Crashes of Frontage Road Left Turns (All Sites).

Variables	Estimate	Standard Error	Z Value	Pr(>|z|)
(Intercept)	-1.340	0.105	-12.784	< 2e-16
UTurns_per_siteOne	-0.007	0.259	-.025	0.9798
UTurns_per_siteTwo	-0.226	0.120	-1.891	0.0586
IntControlUnsignalized	0.576	0.287	2.006	0.0448
I(MaxOfPosted>50)	0.268	0.145	1.851	0.0641

Where:

UTurns_per_siteOne & UTurns_per_siteTwo = Number of U-turns at a site (ranging from 0 to 2).

IntControlUnsignalized = Intersection control (signalized, unsignalized, mixed).

MaxOfPosted = Maximum posted speed limit on the frontage road for both sides of the interchange.

Highlighted values of Pr represent significance of 5 percent or less.

By inspection of the variables included in the model (with a response variable that is the proportion of KAB crashes to the total FR left turns), the following general observations merit consideration:

- Traffic signal control is associated with a reduction in the proportion of KAB left-turn crashes originating on the FR (significant at 4.5 percent).
- Sites with two turnarounds experienced fewer severe crashes originating on the FR (significant at 5.9 percent).

- Sites with maximum posted speed limits below 50 mph had less severe KAB left-turn crashes that originate on the FR (significant at 6.4 percent).

Because the above model evaluated only the proportion of KAB left-turn crashes that originated on the FR contrasted to the total number of FR left-turn crashes, researchers next evaluated the effects of the proportion of KAB crashes to the total number of intersection crashes to determine the overall influence on the entire interchange configuration (see Table 229).

Table 229. Proportion of KAB Crashes among All Intersection Crashes (All Sites).

Variables	Estimate	Standard Error	Z Value	Pr(>|z|)
(Intercept)	-1.168	0.055	-21.411	< 2e-16
UTurns_per_siteOne	-0.032	0.123	0.495	0.6207
UTurns_per_siteTwo	-0.162	0.062	-2.615	0.0089
IntControlUnsignalized	0.279	0.140	1.966	0.0494
PostedDif	0.029	0.150	1.916	0.0553

Where:
UTurns_per_siteOne & UTurns_per_siteTwo = Number of U-turns at a site (0 to 2).
IntControlUnsignalized = Intersection control (signalized, unsignalized, mixed).
PostedDir = Difference in posted speed limits between the two frontage roads.
Highlighted values of Pr represent significance of 5 percent or less.

By inspection of the variables included in this alternative model (with a response variable of the proportion of intersection KAB crashes to the total number of crashes), the following general observations merit consideration:

- Sites with two U-turn lanes have fewer severe crashes (significant at 0.9 percent).
- Traffic signal control is associated with a reduction in the proportion of KAB crashes (significant at 4.9 percent).
- Sites with a smaller difference in posted speed limits between the two FRs have fewer KAB crashes (significant at 5.5 percent).

SAFETY EFFECTS FOR U-TURN SIGNALIZED SITES

Researchers next focused on the predominant site condition (i.e., locations with dedicated U-turns that operate using traffic signal control). The previous analysis noted that unsignalized intersections had a higher proportion of KAB crashes and represented a very small proportion of the study sites, so those sites are not included in this second analysis. The resulting generalized linear mixed model for FR left-turn crashes considered 76 sites with a total of 500 site-periods (see Table 230). The goal of this analysis was to determine if there were secondary influences that adversely impact safety performance at the signalized U-turn interchange locations.

Table 230. KAB Left-Turn Crashes of Frontage Road Left Turns (U-Turn, Signalized Sites).

Variables	Estimate	Standard Error	Z Value	Pr(>|z|)
(Intercept)	-1.139	0.100	-11.437	< 2e-16
$Merge_{RT}$	-0.172	0.057	-2.990	0.0028
RtD	-0.171	0.065	-2.637	0.00884
RTwithExclusiveLane_5	-0.289	0.099	-2.904	0.0037
Scale (I(AvgOfLeg1BayLength – AvgLeg1DivergingLength_mod)	-0.179	0.048	-3.699	0.0002
UnequallyPosted	0.603	0.186	3.243	0.0012
Scale(AvgInteriorSpacing_mod)	0.246	0.051	4.801	1.58e-06
Div_Shared_Lane	-0.330	0.168	-1.959	0.0501
Scale(DWY)	0.110	0.060	1.842	0.0655

Where:

$Merge_{RT}$ = Number of instances at the site where right-turn "zone" exit treatment merged into an existing lane.

RtD = Number of instances at the site where RTTreat had an exclusive right lane with a raised channelization island.

RTwithExclusiveLane_5 = Number of instances at the site where RTwithExclusiveLane required vehicles to merge with frontage road traffic (with no additional traffic control).

Scale (I(AvgOfLeg1BayLength – AvgLeg1DivergingLength_mod) = Difference between the average length of the turning bay and the average diverging length.

UnequallyPosted = Locations where the two frontage roads have different posted speed limits.

AvgInteriorSpacing_mod = Average distances between the stop bars at the intersections.

Div_Shared_Lane = Number of instances at a site where the shared lane was the diverging traffic option for U-turn traffic.

DWY = Minimum of distance to closest downstream driveway.

Highlighted values of Pr represent significance of 5 percent or less.

By inspection of the variables included in the model (with a response variable of the proportion of intersection KAB left-turning FRs to the total number of left-turning FR crashes for signalized locations with dedicated U-turn lanes), the following general observations merit consideration:

- Sites where the U-turn both merges and diverges from a shared lane tend to have fewer severe crashes (significant at < 0.001 percent). The diverging shared lane is associated with a reduction of 28.2 percent (calculated as exp(-0.330) = 0.722) (significant at 5 percent). The merging shared lane is associated with a reduction of 15.8 percent (calculated as exp(-0.172) = 0.842) (significant at 0.3 percent).
- Sites where the exclusive right-turn lane (raised island) from the cross street conflicts with the U-turn have 15.7 percent fewer KAB crashes (exp(-0.171) = 0.849) (significant at 0.9 percent). This trend, however, changes when total crashes are explicitly considered (rather than only left turns, as included in the Table 229 model).

- Sites where the right turn from the cross street must merge without additional traffic control have 25.1 percent fewer KAB crashes (calculated at 0.4 percent) than other exit treatments (significant at 0.4 percent).
- KAB crashes are smaller by 0.3 percent for each additional foot in the taper opening of the U-turn entry (significant at 0.02 percent).
- Sites with varying FR speed limits have 1.8 percent more KAB crashes.
- KAB crashes are 0.3 percent larger for each additional foot in interior spacing for the interchange (significant at 0.12 percent).
- KAB crashes are larger by 0.1 percent for each additional foot between the closest downstream driveway and the U-turn exit (significant at 6.6 percent).

SCALING VARIABLES

As part of the KAB crash model, researchers explored the application of adjusting select (widely dispersed) variables by a scale factor. Different scales of covariates may influence the efficiency of model-fitting algorithms, particularly for maximum-likelihood estimation of generalized-mixed-effects models whose feasible spaces are not necessarily concave. To control for this issue, researchers performed two-level scaling for some variables during the model selection process. As indicated by its name, two steps are taken to perform the procedure. In the case where the variable being scaled is called X, the scaling process would be performed as follows:

1. For a given dataset, subtract the mean of X from all X values.
2. Divide the differences obtained in Step 1 by the standard deviation of X.

The use of scaled variables for X has the following two impacts in the model coefficients:

- The intercept shifts so that the reference level is at mean(X).
- The regression coefficient $\beta_{scaled(X)}$ for the scaled variable is such that the effect of X on the link scale is:

$$\beta_X = \frac{\beta_{scale(X)}}{S.D.(X)}$$

Where:

$\boldsymbol{X}$	=	**Variable of analysis.**
$\boldsymbol{\beta_X}$	=	Log of the marginal effect of X on number of crashes.
$\boldsymbol{\beta_{scale(X)}}$	=	Log of the marginal effect of scaled X on the number of crashes.
$\boldsymbol{S.D.(X)}$	=	Standard deviation of X in the dataset used to estimate $\beta_{scale(X)}$.

Therefore, Table 231 depicts the set of standard deviations required to derive the effects of the scaled values from the final frequency model (as summarized in the following section).

Table 231. Standard Deviations Needed to Derive the Effects of Scaled Values.

Variable	Mean	Standard Deviation
CS_AADT	15,039.44	15,243.06
MaxFrontageAADT	11,298.16	9883.32
MinFrontageAADT	3789.48	8044.62
IntAngle	80.62	13.98
D_to_Closest_Driveway	229.95	174.45
MinNoLanesFrontage	2.24	0.51
Sum of AADTs	30,127.08	22,793.29
Sum of log of AADTs	24.21	3.45

FREQUENCY ANALYSIS

The severity analysis provided information related to the expected effect individual road characteristics may have on the total FR KAB left turns as well as the total interchange KAB crashes. This type of information is particularly useful if an agency is assessing an existing facility. There is also a need to estimate the predicted number of crashes that may occur based on the individual site characteristics so that agencies considering constructing these dedicated U-turns can determine how this construction may impact the overall facility's safety performance.

Descriptive Statistics

Because the database used for this analysis is the same as that used for the severity analysis, the descriptive statistics are the same; however, predictive models tend to include exposure variables (usually in the form of AADT values), so researchers graphically explored how the AADT on the cross street compared to the number of crashes per year. Figure 99 shows the crash data plotted against the cross-street AADT values and includes trend lines to help assess a preliminary model functional form. Three items are notable when inspecting this graphic. First, a large AADT number (up to almost 200,000 vehicles per day) is shown for only one site. The model development effort should then include a maximum AADT value to screen out these types of outliers. Second, the intercept of the trend line is substantially greater than zero. Finally, the shape of the trend line does not conform to traditional assumptions (i.e., as AADT increases, the number of crashes will always increase). Inspection of the FR crashes resulted in similar observations. Consequently, researchers focused on first identifying a model functional form that would be suitable for the proposed statistically derived model.

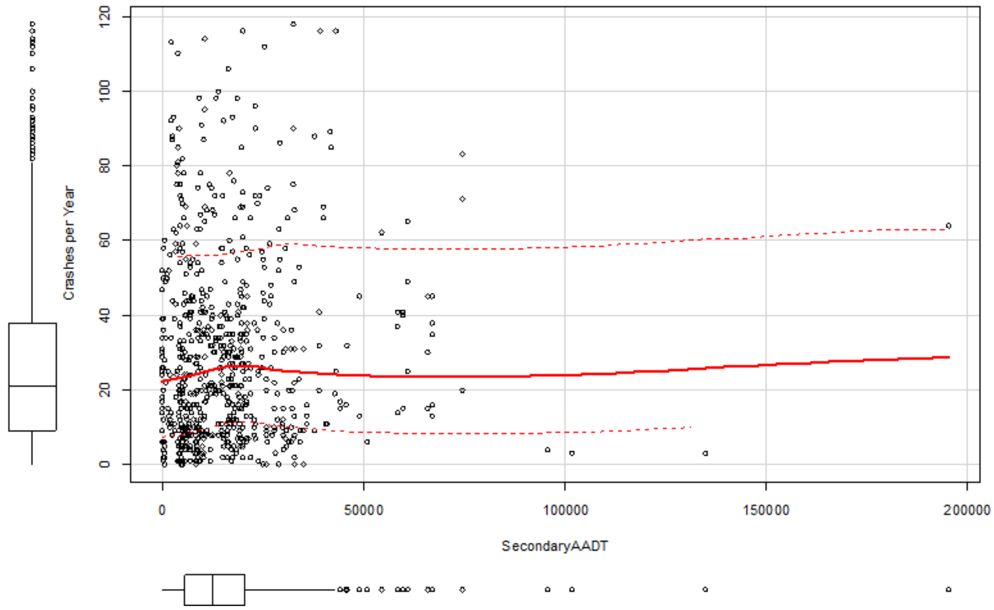

Figure 99. Cross-Street AADT Compared to the Number of Crashes per Year.

Statistical Analysis and Results

For the frequency analysis, researchers focused on the estimation of total crashes for each configuration. To do this, the first step required assessing the model functional form followed by deriving a final predictive model.

Assessing the Model Functional Form

The initial proposed predictive model explored three potential exposure variables—the AADT on the cross street and the AADT on each FR. For this analysis, and building on findings from the severity analysis, researchers used only the signalized intersection locations with speed limits greater than 30 mph for the frequency assessment. In addition, researchers only included sites where all three AADT values were available.

The identification of a functional form that captured the unusual data trends posed a unique challenge. Figure 100 depicts how the proposed model functional form (shown with a solid red line) appears when plotted against all three AADT conditions. This overall fit improves with the addition of significant variables to the model.

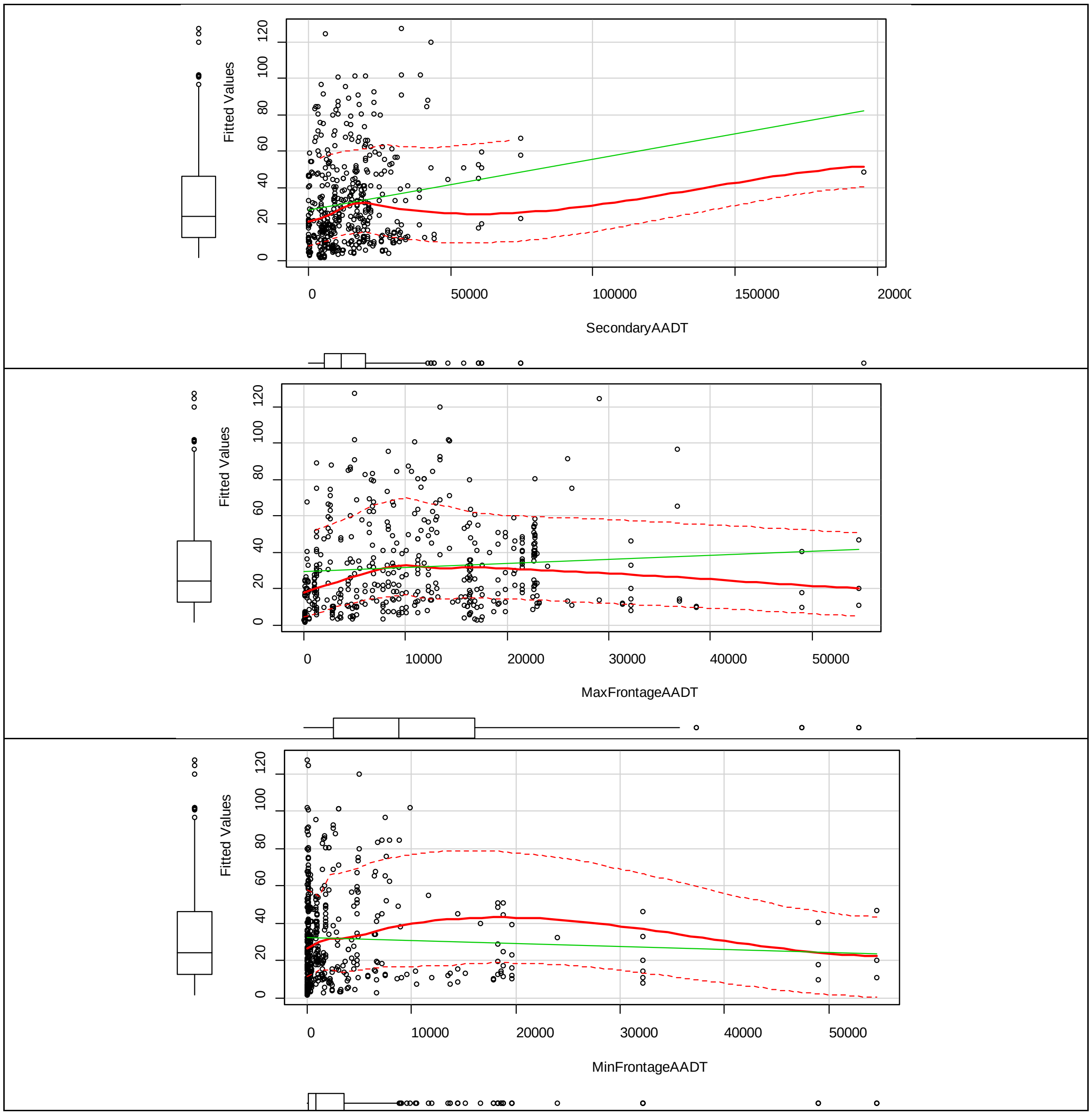

Figure 100. Raw Crash Data Plotted Against AADT Values.

Deriving the Total Crash Model (Initial Refinement)

For the development of the predictive model, researchers initially focused on signalized intersection locations with speed limits greater than 30 mph and AADT values available for all three roads. This data set resulted in 124 site locations with 440 site-periods. Through the use of stepwise regression procedures, researchers developed a candidate predictive model. This resulting model included the specific crash year as a key input into the model and is depicted in Table 232.

Table 232. Predictive Model for Total Crashes (Signalized Sites with Yearly Factor).

Variables	Estimate	Standard Error	Z Value	Pr(>|z|)
(Intercept)	5.701	0.168	34.00	< 2e-16
Scale(CS_AADT)	-0.072	0.028	-2.56	0.0105
RtA	-0.299	0.098	-3.07	0.0022
Scale(IntAngle)	-0.166	0.070	-2.36	0.0184
I(RTwithExclusiveLane_3 + RTwithExclusiveLane_1)	-0.311	0.177	-1.75	0.0793
Scale(DWY)	-0.137	0.069	-1.97	0.0494
Scale(MinNoLanesFrontage)	0.299	0.072	4.18	2.93e-05
Log(MinFrontageAADT)	-0.374	0.020	-18.98	< 2e-16

Where:

CS_AADT = Cross-street AADT value.

RtA = Number of instances at the site where the right-turn zone entrance treatment had a shared right-turn lane and no channelization island. Value of RtA ranges from zero (no shared lane option) up to two (shared lane option at both cross-street right-turn locations).

IntAngle = Average intersection angle (between both sides of interchange).

RTwithExclusiveLane_1 = Cross-street right-turn exit treatments with an additional lane but no additional traffic control.

RTwithExclusiveLane_3 = Cross-street right-turn exit lanes with a merge lane and stop sign traffic control.

DWY = Minimum of distance to closest downstream driveway.

MinNoLanesFrontage = Minimum total number of lanes at the approach of the U-turns at the site.

MinFrontageAADT = Lowest frontage road AADT value.

Highlighted values of Pr represent significance of 5 percent or less.

As noted in the model, there are seven significant road characteristics in the model. Because the model development depended on the specific crash year, the use of this type of a model is limited because it can only be applied to historic crash conditions. Consequently, researchers next evaluated the exact same model (with the same variables) but removed the requirement of incorporating the crash year. This change enables users to apply the model to other locations and for different years.

The resulting model, shown in Table 233, does not fit as well as the previous model, and the number of significant variables is much lower; however, upon inspection, it is notable that the variable estimates are basically the same as those for the yearly model. This finding means that the equation will be similar but less complex, and the application of the equation will allow expanded analysis. As a result, the final model that incorporates at least two of the exposure elements is the one shown in Table 233. Because the number of FR lanes is expected to be correlated to the FR AADT, researchers explored an additional total crash model (as presented in the body of this report).

One interesting observation about this intermediate model is that the presence of a U-turn does not appear as a critical variable in the model. Researchers included this variable in the stepwise analysis, and it was not significant. This finding reveals that constructing a dedicated U-turn lane does not reduce the overall number of crashes, but it does reduce the crash severity (as demonstrated by the severity analysis models).

Table 233. Predictive Model for Total Crashes (Signalized Sites but without a Yearly Factor).

Variables	Estimate	Standard Error	Z Value	Pr(>|z|)
(Intercept)	5.722	0.153	37.50	< 2e-16
Scale(CS_AADT)	-0.051	0.028	-1.83	0.0669
RtA	-0.292	0.098	-3.00	0.0027
Scale(IntAngle)	-0.162	0.070	-2.31	0.0212
I(RTwithExclusiveLane_3 + RTwithExclusiveLane_1)	-0.300	0.177	-1.69	0.0908
Scale(DWY)	-0.142	0.070	-2.04	0.0410
Scale(MinNoLanesFrontage)	0.298	0.072	4.17	3.07e-05
Log(MinFrontageAADT)	-0.377	0.019	-19.58	< 2e-16

Where:

CS_AADT = Cross-street AADT value.

RtA = Number of instances at the site where RTTreat had a shared right-turn lane and no channelization island.

IntAngle = Average intersection angle (between both sides of interchange).

RTwithExclusiveLane_1 = Cross-street right-turn exit treatments with an additional lane but no additional traffic control.

RTwithExclusiveLane_3 = Cross-street right-turn exit lanes with a merge lane and stop sign traffic control.

DWY = Minimum of distance to closest downstream driveway.

MinNoLanesFrontage = Minimum total number of lanes at the approach of the U-turns at the site.

MinFrontageAADT = Lowest frontage road AADT value.

Highlighted values of Pr represent significance of 5 percent or less.

The above-referenced model presents a complex functional form that incorporates scaling of some variables, multiple parameters for AADTs, and the use of a mixed-effect model specification. The equation for the final model can be written as follows:

$$N = \exp(0.7 \times \frac{\sum LogAADTs - 24.20913}{3.450924} + N(5.72150\,, 0.7253) - 0.05108$$

$$\times \frac{(SecondaryAADT - 15{,}039.44)}{15{,}243.06} - (0.29186 \times \mathrm{RtA}) - 0.1626$$

$$\times \frac{IntAngle - 80.6125}{13.97938} - 0.29965 \times (\mathrm{Num.RTLanes\_w\_Exit\_1}) - 0.29965$$

$$\times (\mathrm{Num.RTLanes\_w\_Exit\_3}) - 0.14210 \times \frac{\mathrm{DWY} - 229.9523}{174.4452} + 0.29840$$

$$\times \frac{\mathrm{MinNoLanesFrontage} - 2.236364}{0.512749} - 0.37698 \times \log(\mathrm{MinFrontageAADT})$$

It can be shown that after algebraic manipulations, the predictive equation is as follows:

$$N = \exp(0.7 \times \frac{\sum LogAADTs}{3.450924} - 0.7 \times \frac{-24.20913}{3.450924} + N(5.72150\,, 0.7253) - 0.05108$$

$$\times \frac{SecondaryAADT}{15{,}243.06} + 0.05108 \times \frac{15{,}039.44}{15{,}243.06} - 0.29186 \times \mathrm{RtA} - 0.1626 \times \frac{IntAngle}{13.97938}$$

$$+ 0.1626 \times \frac{80.6125}{13.97938} - 0.29965 \times \left(\mathrm{Num.RTLanes}_{\mathrm{w}_{\mathrm{Exit}_1}}\right) - 0.29965$$

$$\times \left(\mathrm{Num.RTLanes}_{\mathrm{w}_{\mathrm{Exit}_3}}\right) - 0.14210 \times \frac{\mathrm{DWY}}{174.4452} + 0.14210 \times \frac{229.9523}{174.4452} + 0.29840$$

$$\times \frac{\mathrm{MinNoLanesFrontage}}{0.512749} - 0.29840 \times \frac{2.236364}{0.512749} - 0.37698$$

$$\times \log(\mathrm{MinFrontageAADT})$$

Researchers then further refined this complex model to minimize correlations between variables and develop a simpler model with similar predictive powers.

Deriving the Total Crash Model (Second Refinement—Retaining Yearly Factor)

For the development of the final predictive model, researchers focused on signalized intersection locations with speed limits greater than 30 mph. Through the use of stepwise regression procedures, researchers developed a final predictive model for total crashes. The model selection was performed in several stages such that groups of variables jointly available for subsets of data were considered together at each stage. Once a stage had arrived at a parsimonious model, that model was fitted to the largest subset of data that had all variables in the model specification. The process was repeated multiple times until all variables had been considered twice for inclusion into the model. Last, the final model was estimated for the largest data set with all its variables available. This dataset represented 86 site locations with 459 site periods available for model estimation. This resulting model included the specific crash year as a key input into the model and is depicted in Table 234. This modified total crash model retains the yearly factor but also includes simplified variable formats.

The crash data spanned several years, so the first modeling attempt included a yearly factor. For predictive purposes, this type of variable can be limiting, so researchers developed a similar model without the yearly factor (this is the final non-freeway total crash model included in the body of this report).

A key difference from the initial refinement was the use of the cross-street AADT only combined with the average number of lanes on the FRs (a probable surrogate for exposure on these facilities).

Table 234. Predictive Model for Total Crashes (Signalized Sites with Yearly Factor).

Variables	Estimate	Standard Error	Z Value	Pr(>|z|)	Significance[b]
(Intercept)[a]	5.3041	1.0862	4.8834	1.0428E-06	***
RtA	-0.2708	0.1023	-2.6480	0.0081	**
scale(DWY)	-0.2684	0.0719	-3.7320	0.0002	***
log(Rmin)	-0.9512	0.2454	-3.8760	0.0001	***
scale(CS_AADT)	0.1131	0.0489	2.3120	0.0208	*
AvgLn	0.7027	0.1616	4.3490	0.0000	***

Where:

CS_AADT = Cross-street AADT value.

RtA = Number of instances at the site where RTTreat had a shared right-turn lane and no channelization island.

AvgInterionSpacing_mod = Average spacing between interior edges of the frontage roads (ft).

DWY = Minimum of distance to closest downstream driveway.

AvgLn = Average number of lanes per frontage approach at the site.

Notes:

[a] Includes adjustment due to random effects.

[b] Significance levels are as follows:

* Statistically different from 0.0 at the 5.0% significance level.

** Statistically different from 0.0 at the 1.0% significance level.

*** Statistically different from 0.0 at the 0.1% significance level.

As shown in Figure 101, the use of CURE plots shows minimal deviations beyond the expected boundaries for key variables in the model.

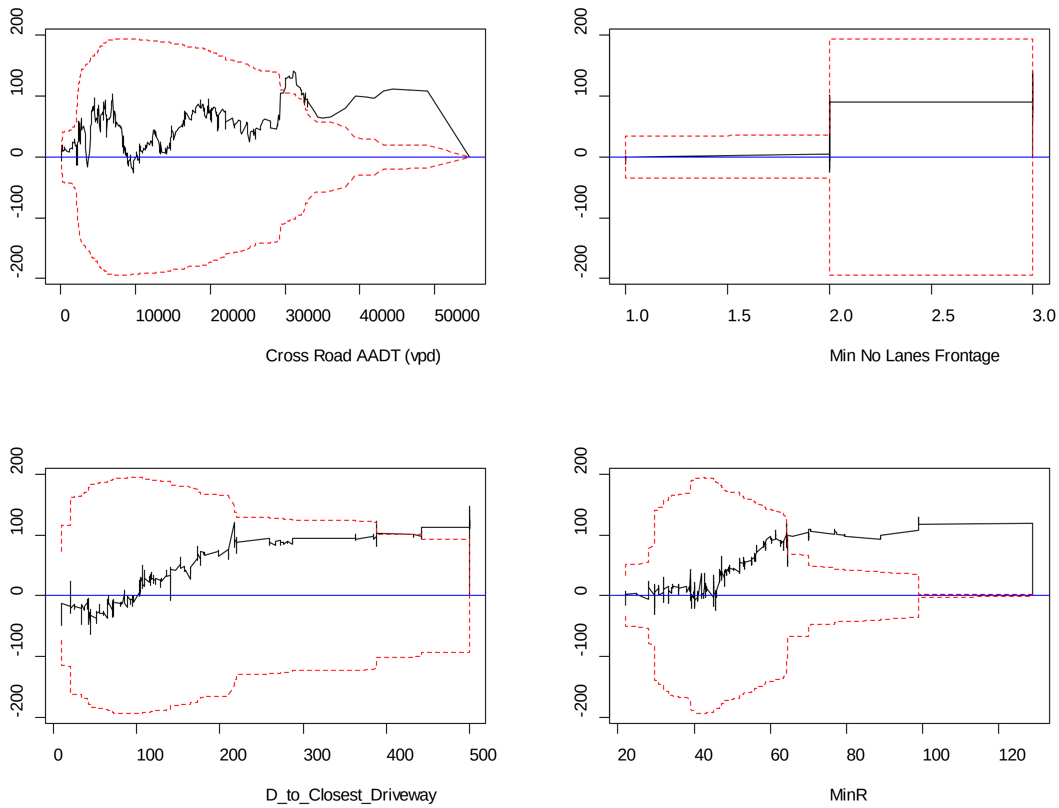

Figure 101. CURE Plots for Second Refinement of the Total Crash Model.

Figure 102 provides a graphic assessment of the prediction power of the model.

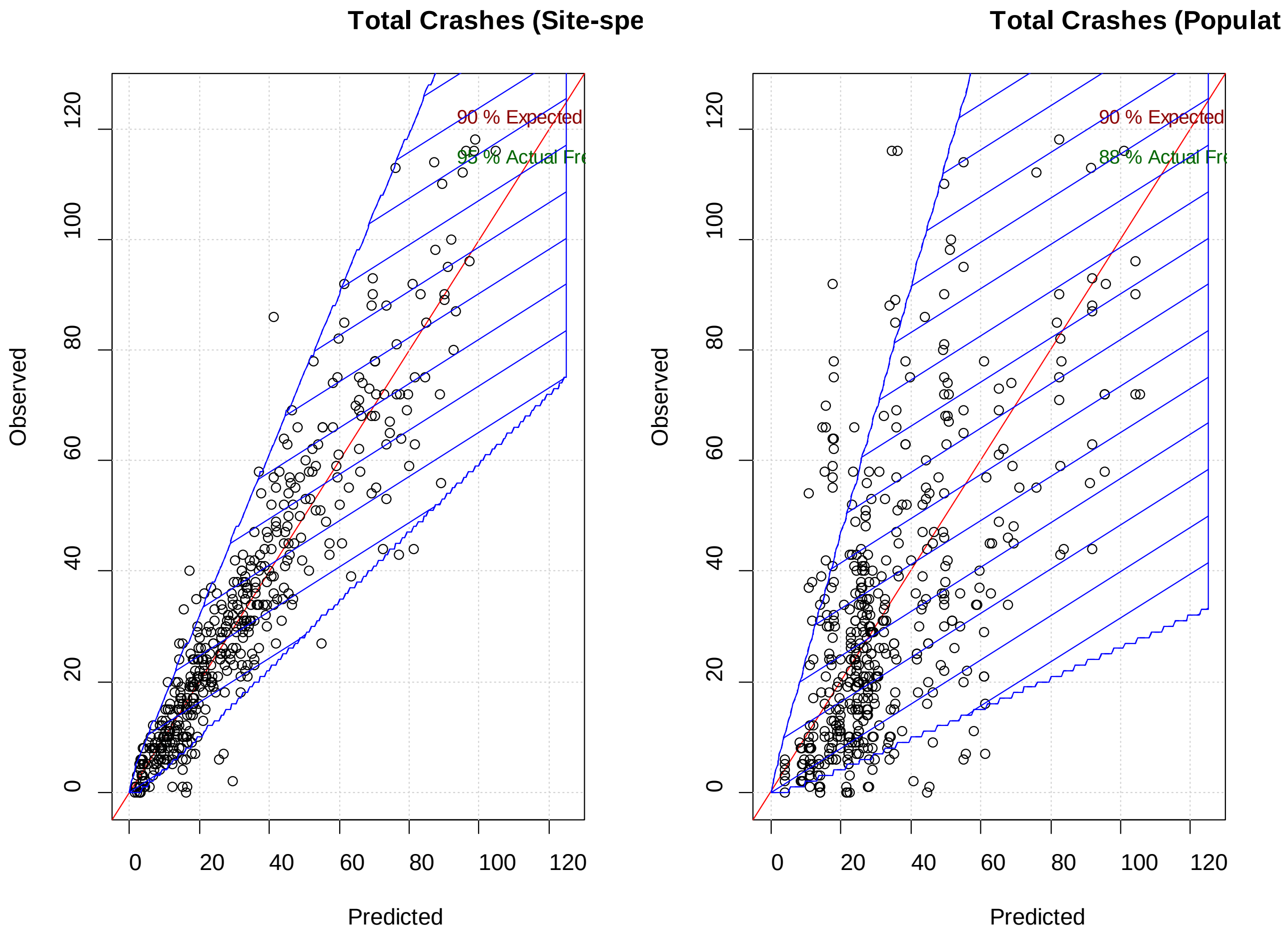

Figure 102. Model Fit for the Total Crashes Model (Second Refinement—Site-Specific versus Total Site Population).

As noted in the model, there are five significant road characteristics that relate to total crashes. Because the model development depended on the specific crash year, the use of this type of a model is limited since it can only be applied to historic crash conditions. Regardless, the equation above incorporates the variation due to the specific crash year as a small shift in the intercept as well as a small increase in the dispersion of the predictions. Therefore, users may apply the model to other locations and for different years. One interesting observation about this second refinement of the total crash model is that the presence of a U-turn does not appear as a critical variable in the model. Researchers included this variable in the stepwise analysis, and it was not significant. This finding demonstrates that constructing a dedicated U-turn lane does not affect the overall number of crashes, so the operational benefits do appear to come without an additional expectation of crashes.

By inspection of the variables included in the model (with a response variable of the number of intersection total crashes for signalized locations with dedicated U-turn lanes), the following general observations merit consideration:

- Sites' right turns from the cross street that must merge without additional traffic control have 23.7 percent fewer crashes (calculated as 1 − exp(-0.2708) = 0.237) (significant at 1 percent).
- The number of crashes is smaller by 1.7 percent for each additional 10 ft between the closest downstream driveway and the U-turn exit (calculated as 1 − exp(-0.2684/155.7521*10) = 0.0171) (significant at 0.1 percent).
- The number of crashes is smaller by 8.7 percent for each increase of 10 percent in the turning radius of the U-turn (calculated as 1 − exp(-0.9512*ln(1.1) = 0.0867) (significant at 0.1 percent).
- The number of crashes is larger by 1.1 percent for each additional 1000 vpd increase in Cross-road AADT (calculated as 1 − exp(0.1131/10,059.6*1000 = 0.011) (significant at 5 percent).
- The number of crashes increases by a factor of 2.01 (doubles) for each additional lane in the FR (calculated as exp(0.7027) = 2.014) (significant at 0.1 percent). (This metric is probably a surrogate of AADT on the FR).

The second refinement of the total crash model presents a functional form that incorporates scaling of some variables. This result can be written in an equation for the final model, as follows:

$$N = \exp[5.3041 - (0.2708 \times \mathrm{RtA}) - 0.2684 \times \frac{\mathrm{DWY} - 196.3186}{155.7521} + 0.1131 \times \frac{(CS\_AADT - 13{,}516.82)}{10{,}059.57} +$$

$$0.7027 \times \mathrm{AvgLn} - 0.9512 \times \ln(\mathrm{Rmin})]$$

After simplification, the predictive equation takes the following form:

$$N = \exp[5.4904 - (0.2708 \times \mathrm{RtA}) - (1.70 \times 10^{-3} \times \mathrm{DWY}) + \frac{0.2684 \times 196.3186}{155.7521} +$$

$$(1.124 \times 10^{-5} \times CS\_AADT) - \left(\frac{0.1131 \times 13{,}516.82}{10{,}059.57}\right) + 0.7027 \times \mathrm{AvgLn}] / \mathrm{Rmin}^{0.9512}$$

The final total crash refinement is included in the body of this report. This refined model does not include a yearly factor but does otherwise include variables consistent with those determined for the second refinement of the total crash model.