Implementation Strategies For Real-time Traffic Safety Improvements On Urban Freeways

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IMPLEMENTATION STRATEGIES FOR REAL-TIME TRAFFIC SAFETY IMPROVEMENTS ON URBAN FREEWAYS

by

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B.S.M.E. University of Central Florida, 2002
B.S. University of Central Florida, 2002

A thesis submitted in partial fulfillment of the requirements for the degree of Master of Science in the Department of Civil & Environmental Engineering in the College of Engineering and Computer Science at the University of Central Florida Orlando, Florida

Spring Term
2005

Major Professor: Dr. Mohamed Abdel-Aty
ABSTRACT

This research evaluates Intelligent Transportation System (ITS) implementation strategies to improve the safety of a freeway once a potential of a crash is detected. Among these strategies are Variable Speed Limit (VSL) and ramp metering. VSL are ITS devices that are commonly used to calm traffic in an attempt to relieve congestion and enhance throughput. With proper use, VSL can be more cost effective than adding more lanes. In addition to maximizing the capacity of a roadway, a different aspect of VSL can be realized by the potential of improving traffic safety. Through the use of multiple microscopic traffic simulations, best practices can be determined, and a final recommendation can be made. Ramp metering is a method to control the amount of traffic flow entering from on-ramps to achieve a better efficiency of the freeway. It can also have a potential benefit in improving the safety of the freeway.

This thesis pursues the goal of a best-case implementation of VSL. Two loading scenarios, a fully loaded case (90% of ramp maximums) and an off-peak loading case (60% of ramp maximums), at multiple stations with multiple implementation methods are strategically attempted until a best-case implementation is found.

The final recommendation for the off-peak loading is a 15 mph speed reduction for 2 miles upstream and a 15 mph increase in speed for the 2 miles downstream of the detector that shows a high crash potential. The speed change is to be implemented in 5 mph increments every 10 minutes. The recommended case is found to reduce relative crash potential from .065 to -.292, as measured by a high-speed crash prediction algorithm (Abdel-Aty et al. 2005). A possibility of crash migration to downstream and
upstream locations was observed, however, the safety and efficiency benefits far outweigh the crash migration potential.

No final recommendation is made for the use of VSL in the fully loaded case (low-speed case); however, ramp metering indicated a promising potential for safety improvement.
ACKNOWLEDGEMENTS
# TABLE OF CONTENTS

LIST OF FIGURES ................................................................................................................. xii

LIST OF TABLES .................................................................................................................... xviii

LIST OF ACRONYMS .............................................................................................................. xx

CHAPTER ONE: INTRODUCTION ..................................................................................... 1

1.1 Problem Definition ........................................................................................................ 1

1.2 Interstate 4 ..................................................................................................................... 2

1.3 Objective ......................................................................................................................... 3

1.4 Balancing Tradeoffs ...................................................................................................... 4

CHAPTER TWO: LITERATURE REVIEW ........................................................................... 5

2.1 Traffic Simulation .......................................................................................................... 5

  2.1.1 Simulation Choice .................................................................................................. 8

  2.1.2 Simulation Software Choice .................................................................................. 8

2.2 Verifying, Validating, and Calibrating PARAMICS ..................................................... 12

  2.2.1 Verification ........................................................................................................... 12

  2.2.2 Calibration ............................................................................................................ 15

  2.2.3 Validation .............................................................................................................. 15

2.3 Variable Speed Limits ................................................................................................. 16

  2.3.1 Who Uses VSL .................................................................................................... 16

  2.3.2 Why Use VSL ..................................................................................................... 16

  2.3.3 Method of Determining Displayed Speed ............................................................ 19

  2.3.4 Sign Spacing ....................................................................................................... 20

  2.3.5 Method of Investigation VSL Effects .................................................................... 20
2.3.6 Downfalls ................................................................................................................. 21

2.4 PARAMICS Simulation Software .................................................................................. 21

CHAPTER THREE: METHODOLOGY ................................................................................. 23

3.1 Scope of the Thesis ...................................................................................................... 23

3.2 Overlay Generation .................................................................................................... 23

3.2.1 AutoCAD .................................................................................................................. 23

3.2.2 Aerial Photographs .................................................................................................. 25

3.2.3 As-Built Drawings .................................................................................................... 25

3.3 Overlay Importation .................................................................................................... 26

3.3.1 Zero lay required ...................................................................................................... 27

3.4 Definitions .................................................................................................................. 28

3.5 Drawing Network ........................................................................................................ 29

3.5.1 Road Categories ...................................................................................................... 29

3.5.2 Zones ...................................................................................................................... 30

3.5.3 Two Roads versus One Road .................................................................................. 31

3.5.4 Left Ramps .............................................................................................................. 33

3.5.5 Loop Detectors ........................................................................................................ 34

3.5.5.1 Location Determination ....................................................................................... 34

3.5.5.2 Number of Lanes .................................................................................................. 34

3.6 Calibration of the OD Matrix ...................................................................................... 35

3.6.1 Data Sources .......................................................................................................... 35

3.6.2 Data Sources Compared ......................................................................................... 36

3.6.3 Finding Zeros .......................................................................................................... 36
3.10.1 Adjusted Detector Length ................................................................. 57
3.10.2 Adjusted Link Length .................................................................. 57
3.10.3 How accurate is the network ......................................................... 58
3.10.4 Network Accuracy ....................................................................... 58
3.11 Problems with PARAMICS ................................................................. 59
3.11.1 Merging Behavior on Onramps ..................................................... 59
3.11.2 Overestimation of Jams ................................................................. 60
3.11.3 Overreaction of Drivers ............................................................... 60
3.11.4 Difficult to Import Overlays ......................................................... 60
3.11.5 Inaccurate Examples of Code ...................................................... 61
3.11.6 Difficulty Collecting Specific Loop Information ......................... 62
3.11.7 Five Minute Interval Required On All Link Changes ................. 62
3.11.8 Unable to Specify Distributions ............................................... 63
3.11.9 Too Many U-Turns, Lost Drivers .............................................. 63
3.11.10 No Approaches; Too Much Lane Changing Behavior at Intersections ................................................................. 63
3.11.11 No Left Ramps ......................................................................... 64

CHAPTER FOUR: EXPERIMENTAL DESIGN ................................................. 65
4.1 Mathematical Model .......................................................................... 65
4.2 Network Loading ............................................................................... 69
4.3 Definitions ......................................................................................... 70
4.4 Variables in the Scenarios ............................................................... 72
4.4.1 Temporal Variation ...................................................................... 72
4.4.2 Spatial Variation .......................................................................... 73
<table>
<thead>
<tr>
<th>Section</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.4.2.2.1 Crash Potential at 47</td>
<td>110</td>
</tr>
<tr>
<td>5.4.2.2.2 Upstream Crash Potential</td>
<td>111</td>
</tr>
<tr>
<td>5.4.2.2.3 Downstream Crash Potential</td>
<td>117</td>
</tr>
<tr>
<td>5.4.2.3 Time Scenarios</td>
<td>123</td>
</tr>
<tr>
<td>5.4.2.4 Gap Scenarios</td>
<td>127</td>
</tr>
<tr>
<td>5.5 Ramp Metering</td>
<td>129</td>
</tr>
<tr>
<td>5.6 High Speed Best Case Scenario</td>
<td>138</td>
</tr>
<tr>
<td>5.6.1 Statistical Analysis</td>
<td>145</td>
</tr>
<tr>
<td>5.6.1.1 Station of Interest</td>
<td>147</td>
</tr>
<tr>
<td>5.6.1.2 Upstream of the Station of Interest</td>
<td>148</td>
</tr>
<tr>
<td>5.6.1.3 Downstream of the Station of Interest</td>
<td>148</td>
</tr>
<tr>
<td>5.6.2 Travel Time Analysis</td>
<td>149</td>
</tr>
<tr>
<td>5.6.3 Model Application</td>
<td>150</td>
</tr>
<tr>
<td>5.7 Best Low Speed Scenario</td>
<td>151</td>
</tr>
<tr>
<td>5.8 Ramp and Detector Locations</td>
<td>192</td>
</tr>
<tr>
<td>APPENDIX A: EIGHT SCENARIOS FOR THE LOW SPEED CASES</td>
<td>156</td>
</tr>
<tr>
<td>APPENDIX B: INPUT FROM PARAMICS FILES</td>
<td>159</td>
</tr>
<tr>
<td>APPENDIX C: OD MATRIX</td>
<td>163</td>
</tr>
<tr>
<td>APPENDIX D: VERIFICATION STEPS</td>
<td>166</td>
</tr>
<tr>
<td>APPENDIX E: VISUAL BASIC CODE FOR EXCEL EVALUATION</td>
<td>168</td>
</tr>
<tr>
<td>APPENDIX F: RAMP AND DETECTOR LOCATIONS</td>
<td>192</td>
</tr>
<tr>
<td>LIST OF REFERENCES</td>
<td>194</td>
</tr>
</tbody>
</table>
LIST OF FIGURES

Figure 1-1: A screenshot of the I-4 East-West Expressway Interchange with loops........ 3
Figure 3-1: I-4 with major road and station locations............................................... 24
Figure 3-2: View of original AutoCAD file with xref aerial Orange County .............. 26
Figure 3-3: View of original AutoCAD file with xref aerial of Orange and Seminole County...................................................................................................................... 27
Figure 3-4: Category window .................................................................................. 30
Figure 3-5: View of node location for the ramps....................................................... 32
Figure 3-6: Kerb properties of a northern South Street Ramps................................. 33
Figure 3-7: Zones view at the Orange Blossom Trail Interchange.............................. 36
Figure 3-8: Measurements window ........................................................................ 39
Figure 3-9: Ramp properties at the Fairbanks Interchange........................................ 42
Figure 3-10: Allowed turns as shown in the Junction window................................. 43
Figure 3-11: Nextlanes as shown in the Junction window ........................................ 44
Figure 3-12: Loading of vehicles versus simulated time (The simulation begins at 3:45pm)...................................................................................................................... 45
Figure 3-13: The core behavior variables as shown in the Configuration window ....... 46
Figure 3-14: The expected shape of a speed curve when congestion occurs............. 49
Figure 3-15: The shape of a speed curve with PARAMICS’ default queue speed and distance values ............................................................................................................. 50
Figure 3-16: Morning congestion locations.................................................................. 53
Figure 3-17: Grids at different spacing values over the I-4 network......................... 61
Figure 4-1 Crash Model Definitions......................................................................... 66
Figure 4-2: Relative log odds-ratio derivation ................................................................. 68
Figure 4-3: Sample westbound roadway with detector 47 as the detector of interest. .... 70
Figure 4-4: Map of Interstate-4 through Orlando. ............................................................ 71
Figure 4-5: Sample scenario for a westbound roadway with detector 47 as the detector of interest ........................................................................................................... 72
Figure 4-6: Cases 1 through 4 for the low-speed model ................................................. 76
Figure 4-7: Cases 5 through 8 for the low-speed model ................................................. 77
Figure 4-8: Tests that are to follow for the high-speed model ........................................ 80
Figure 4-9: Scenario with a one mile gap and 15 mph abrupt change in speed limit (time held constant) ........................................................................................................... 84
Figure 5-1: Station 33 speed profile through out the simulation period ......................... 88
Figure 5-2: Station 61 speed profile through out the simulation period ......................... 88
Figure 5-3: Station 61 speed profile through out the simulation period ......................... 89
Figure 5-4: Station 61 speed profile through out the simulation period ......................... 90
Figure 5-5: Station 62 speed profile through out the simulation period ......................... 91
Figure 5-6: Station 33 crash potential through out the simulation period ....................... 92
Figure 5-7: Station 61 crash potential through out the simulation period ....................... 93
Figure 5-8: Station 61 crash potential through out the simulation period ....................... 94
Figure 5-9: Station 47 crash potential through out the simulation period ....................... 95
Figure 5-10: Station 33 crash potential for the base case and VSL implementation cases ................................................................................................................................. 97
Figure 5-11: Station 61 crash potential for the base case and VSL implementation cases. ................................................................................................................................. 98
Figure 5-12: Station 62 crash potentials for the base case and upstream VSL implementation cases................................................................. 99

Figure 5-13: Cases 1 through 4 for the low-speed model.................................................. 100

Figure 5-14: Cases 5 through 8 for the low-speed model............................................... 101

Figure 5-15: Station 64 average speeds for the base case and upstream VSL implementation cases.................................................................................. 102

Figure 5-16: Station 61 crash potentials for the base case and downstream VSL implementation case........................................................................................................... 103

Figure 5-17: Station 61 crash potentials for the base case and downstream VSL implementation case via speed control................................................................. 104

Figure 5-18: Crash potentials at Station 47 for the 6 scenarios in Table 5-2.................. 106

Figure 5-19: Crash potentials at the upstream edge, Station 56, for the 6 scenarios in Table 5-2.......................................................................................................................... 107

Figure 5-20: Crash potentials at the downstream edge, Station 36, for the 6 scenarios in Table 5-2.......................................................................................................................... 108

Figure 5-21: Sample scenario for a westbound roadway.................................................... 109

Figure 5-22: Crash potentials at the detector of interest, Station 47, for the first 5 scenarios in Table 5-3.................................................................................................................. 110

Figure 5-23: Crash potentials at the detector of interest, Station 47, for the last 4 scenarios in Table 5-3.................................................................................................................. 111

Figure 5-24: Crash potentials upstream of the detector of interest, Station 60, for the first 5 scenarios in Table 5-3.................................................................................................................. 112
Figure 5-25: Crash potentials upstream of the detector of interest, Station 61, for the last 4 scenarios in Table 5-3. .................................................................................................................. 113

Figure 5-26: Crash potentials upstream of the detector of interest, Station 56, for the first 5 scenarios in Table 5-3. .................................................................................................................. 114

Figure 5-27: Crash potentials upstream of the detector of interest, Station 57, for the last 4 scenarios in Table 5-3. .................................................................................................................. 115

Figure 5-28: Crash potentials upstream of the detector of interest, Station 52, for the first 5 scenarios in Table 5-3. .................................................................................................................. 116

Figure 5-29: Crash potentials upstream of the detector of interest, Station 52, for the last 4 scenarios in Table 5-3. .................................................................................................................. 117

Figure 5-30: Crash potentials downstream of the detector of interest, Station 43, for the first 5 scenarios in Table 5-3.................................................................................................................. 118

Figure 5-31: Crash potentials downstream of the detector of interest, Station 43, for the last 4 scenarios in Table 5-3.................................................................................................................. 119

Figure 5-32: Crash potentials downstream of the detector of interest, Station 39, for the first 5 scenarios in Table 5-3.................................................................................................................. 120

Figure 5-33: Crash potentials downstream of the detector of interest, Station 39, for the last 4 scenarios in Table 5-3.................................................................................................................. 121

Figure 5-34: Crash potentials downstream of the detector of interest, Station 36, for the first 5 scenarios in Table 5-3.................................................................................................................. 122

Figure 5-35: Crash potentials downstream of the detector of interest, Station 36, for the last 4 scenarios in Table 5-3.................................................................................................................. 123
Figure 5-36: Crash potentials at the detector of interest, Station 47, for the scenarios in Table 5-4. .......................................................... 125
Figure 5-37: Crash potentials downstream of the detector of interest, Station 46, for the scenarios in Table 5-4. .......................................................... 126
Figure 5-38: Crash potentials downstream of the detector of interest, Station 45, for the scenarios in Table 5-4. .......................................................... 127
Figure 5-39: Sample scenario for a westbound roadway........................................ 128
Figure 5-40: Crash potentials at the detector of interest, Station 47, for the scenarios in Table 5-5. .......................................................... 129
Figure 5-41: Crash potentials at the detector of interest, Station 47, for the scenarios in Table 5-5. .......................................................... 130
Figure 5-42: Base scenario for the worst case (Detector 56, upstream of the station of interest) .................................................................................................................. 132
Figure 5-43: Scenarios from Table 5-7 evaluated using the low-speed algorithm at Detector 56 .......................................................... 134
Figure 5-44: Scenarios from Table 5-7 average speeds at Detector 58 (Upstream)..... 135
Figure 5-45: Scenarios from Table 5-7 evaluated using the high-speed algorithm at Detector 56 .......................................................... 136
Figure 5-46: Scenarios from Table 5-7 evaluated using the high-speed and low-speed algorithm at Detector 58. .......................................................... 137
Figure 5-47: Scenarios from Table 5-7 evaluated using the generic algorithm at Detector 56 .......................................................... 138
Figure 5-48: The crash potential at Station 51 (Upstream) for the best scenario under 60% loading. ............................................................ 140

Figure 5-49: The crash potential at Station 45 (Downstream) for the best scenario under 60% loading. ............................................................ 141

Figure 5-50: The crash potential at Station 48 (Upstream) for the best scenario under 60% loading. ............................................................ 142

Figure 5-51: The crash potential at Station 47, (Location of Interest) the detector of interest, for the best scenario under 60% loading. ............................................................ 143

Figure 5-52: The crash potential at Station 50 (Upstream) for the best scenario under 60% loading. ............................................................ 144

Figure 5-53: The crash potential at Station 49 (Upstream) for the best scenario under 60% loading. ............................................................ 145

Figure 5-55: Base and Best Case Scenarios’ average speed at Detector 47 (Detector of Interest). ............................................................ 151

Figure 5-56: Best scenario from Table 5-7 (Case 4) evaluated using the generic algorithm at Detector 56. ............................................................ 152
LIST OF TABLES

Table 2-1: Highway Capacity Manual Simulation Evaluation (HCM 2000) .................... 8
Table 2-2: Summary of Models Based on Initial Criteria (Boxill and Yu, 2000) ........... 10
Table 2-3: Summary of Models Based on In-Depth Criteria (Boxill and Yu, 2000) ...... 11
Table 2-4: Overview of Factors Used in Calibration as Noted by the Literature ............ 15
Table 3-1: Core variable specification and their resulting error percentages ............... 48
Table 3-2: Validation values inspected........................................................................ 51
Table 3-3: Six cases inspected with their factors......................................................... 51
Table 3-4: Case 1 and Case 2 inspection ..................................................................... 54
Table 4-1: VSL variable related assumptions ............................................................. 65
Table 4-2: Different scenarios based on changing speed limit patterns .................... 81
Table 4-3: Different scenarios based on changing distance for speed limit change....... 82
Table 4-4: Different scenarios based on changing speed limits over time ................. 82
Table 4-5: Different scenarios based on the changing gap after the detector of interest
    before raising the speed limit.................................................................................. 83
Table 5-1: The test scenario for the low speed model. ................................................ 96
Table 5-2: Different scenarios based on changing speed limit patterns .................... 105
Table 5-3: Different scenarios based on changing distance........................................ 109
Table 5-4: Different scenarios based on changing the temporal implementation. ....... 124
Table 5-5: Different scenarios based on changing the gap distance.......................... 128
Table 5-6: Different scenarios for the low speed model............................................. 130
Table 5-7: Cases for ramp metering and VSL implementation.................................... 131
Table 5-9: T-test for the network travel time (minutes) ............................................. 150
Table 6-1: The test scenario for the low speed model.
## LIST OF ACRONYMS

<table>
<thead>
<tr>
<th>Acronym</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>CATSS</td>
<td>Center for Advance Transportation Simulation Studies</td>
</tr>
<tr>
<td>FDOT</td>
<td>Florida Department of Transportation</td>
</tr>
<tr>
<td>FSUTMS</td>
<td>Florida Standard Urban Transportation Model Structure</td>
</tr>
<tr>
<td>FTI</td>
<td>Florida Traffic Information</td>
</tr>
<tr>
<td>GOAA</td>
<td>Greater Orlando Aviation Authority</td>
</tr>
<tr>
<td>HCM</td>
<td>Highway Capacity Manual</td>
</tr>
<tr>
<td>I-4</td>
<td>Interstate-4</td>
</tr>
<tr>
<td>ITS</td>
<td>Intelligent Transportation Systems</td>
</tr>
<tr>
<td>UCF</td>
<td>University of Central Florida</td>
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<tr>
<td>VSL</td>
<td>Variable Speed Limits</td>
</tr>
</tbody>
</table>
CHAPTER ONE: INTRODUCTION

1.1 Problem Definition

As long as there have been roads there have been crashes. As vehicle technology improved and speeds increased, these crashes became more and more destructive. While the implementation of crumple zones, driver cages, steel bar doors, and airbags have served to lower the cost in human loss to these crashes, a more comprehensive vision is needed, a view that looks to avoid the crash altogether. Antilock brake systems, four wheel steering, better roadway lighting, and strictly enforced geometric design are certainly a step in the right direction. Each only concentrates on prevention at the driver level attempting to affect how individuals drive. While this paradigm has yielded many advances, different approaches maybe equally effective. Imagine all vehicles functioning as a continuous flow and a crash being a particular development of the flow. The next logical step might be to identify what flow pattern that the traffic exhibited before the crash took place, like looking for the slow churning effects that might lead to a wave. Researchers at UCF developed a crash prediction algorithm based on this idea using Interstate 4 (I-4) loop detectors data. The algorithm has provided good results, outputting the likelihood of a crash based on given flow information. Using the studies results as a metric for crash probability, a known method for testing the traffic patterns effects on crash probability on I-4 is available.

The parameters of traffic flow that need to be changed are realized, with a crash prediction algorithm. If the algorithm shows that high occupancy half a mile upstream for a detector is strongly positively correlated with a crash, then a technique that minimizes occupancy needs to be carried out.
Imagine the expense of equipping a roadway with different technologies to see if the effect is a safer roadway, and then fine-tuning to get the optimal effect. Obviously, this is unrealistic. Simulations are used to carry out trials to determine what equipment, at which settings, yield the best results. Answers are reached relatively quickly, with little expense.

In this case the goal is to minimize the likelihood of a crash as evaluated by the crash algorithm, using two technologies, but focusing mainly on Variable Speed Limits (VSL). The work is similar to that preformed by Lee et al. (2004). However, the work is unique in that VSL are tested, where Lee et al. (2004) considered using Variable Message Signs (VMSs). Also the scope of the network is much larger. Lee et al. (2004) used a tenth of a mile section of interstate, while this research looks at a twenty mile stretch of I-4. In PARAMICS VSL command the same attention and relied on the same factor as static speed limits where VMSs make use of speedcontrol, a PARAMICS function whose effect is not specified.

1.2 Interstate 4

I-4 is the main arterial road for the City of Orlando and other nearby cities. It has spurred development along its path to both the north and south of downtown Orlando. It also serves to connect Orlando to Disney World. With Orlando’s booming population, the roadway has long been overburdened with traffic and can cripple the Orlando network if a crash blocks its lanes. I-4 is monitored 24 hours a day, seven days a week using loop detectors. The detectors are double loops to more accurately determine the speed of vehicles on the road. Each loop is six feet long with a ten-foot gap separating adjacent loops. The loop pairs are spaced half of a mile apart and run from North of Lake
Mary to South of Disney (FDOT later extended). See Figure 1-1 for an overview of loop locations. For this project not all loops on I-4 are considered.

![Figure 1-1: A screenshot of the I-4 East-West Expressway Interchange with loops](image)

### 1.3 Objective

The objective of this research is to explore different ITS strategies to improve safety on I-4. Implementation of variable speed limits on I-4, via simulation, in order to determine a best case for improving safety under different conditions is one of the most promising strategies that is investigated in this research. Multiple scenarios with characteristic trends will be used to generate general “rules” that should result in an optimal safe condition. The scope of the work is limited to looking at a single site for improvement. Multiple sites that simultaneously need to be improved are not considered. However, the relocation of crash risk will be considered when making recommendations.
1.4 Balancing Tradeoffs

The most successful way to prevent vehicle crashes from occurring is to remove all roads. The most efficient way to prevent vehicle crashes from resulting in deaths is to lower all speed limits to around 15mph and strictly enforce the limit. Obviously, neither is a good solution, because of the lessoned utility of roadways. Similarly, severe travel time losses are not acceptable during determination of the setup of VSL results in the safest roadway. Therefore the tradeoff of travel time is considered in all results.
CHAPTER TWO: LITERATURE REVIEW

2.1 Traffic Simulation

Simulation as defined by May (1990) is the “numeric technique for conducting experiments on a digital computer, which may include stochastic characteristics, be microscopic or macroscopic in nature, and involve mathematical models that describe the behavior of the transportation system over extended periods of real time.” The key statement there is that it is a mathematical model, not a life-like model, which describes the behavior of a transportation system, meaning that no more information needs to be entered than is needed, mathematically, to output an answer that is characteristic of the system.

To insure that the model is characteristic of the road network and that the mathematical model is acceptable, verification, calibration, and validation are necessary. The calibration and validation step in a network is dependent on the type of simulation that is being used. In general, traffic simulation is carried out on a driver simulator, with an immersed environment; a driver simulator on a personal computer; a situational experiment on a personal computer; or as simulation is defined by May (1990), on a system wide microscopic, mesoscopic, or macroscopic network simulation. The first three involve a view from a vehicle with individuals providing the input to a changing environment. May’s type of simulation is a network simulation. In which a user inputs the network and the loading conditions and traffic flow is generated.

Simulation has some desirable qualities that make it useful. First, it is cheaper than field-testing in most cases. The upfront cost of software and hardware does not compare
to the cost of outfitting a road network, or the loss in confidence of the driving public on an always-changing driving environment. Second, impossible scenarios maybe carried out. Technology advances rapidly and sometimes the anticipation of a technology may lead a researcher to want to carry out an experiment that makes use of software or hardware that has yet to be made. Third, time can be sped up to yield future results now. The effect of a change many months away can be determined in few hours. Forth, safety issues can be safely tested without the potential of harming any drivers.

When choosing a network or other type of simulator to use the choice comes down to a simple issue of whether the question to be answered involves network characteristics, such as flow, density, and velocity, or if it involves individual decisions, such as how closely one vehicle follows another or how closely a vehicle follows the radius of left hand turn in an intersection. Driver specific issues, such as whether a gap would be large enough to be accepted, are best evaluated using a driver simulator. Network characteristics are better handled using system wide network simulation.

Once that determination of which class of simulator is to be used the next step is determining what level of accuracy is needed, versus what tradeoffs are willing to be sacrificed to decrease the runtime of the simulation. For network wide information microscopic simulation provides the highest level of accuracy, while also requiring the longest run time to simulation. Macroscopic simulation is on the other end of the spectrum. It requires little time, but the results are not as accurate. Similarly, driver simulators with immerse environments provide the highest accuracy but require a large amount of hardware and software and are difficult, if not impossible, to transport. For driver simulators on personal computers, the hardware is transportable but the software is
complicated and expensive. Situation simulation requires no more than a questionnaire and a picture, making it the easiest to use, however, it is the least accurate method.

The accuracy of microscopic simulation is due to the driver-to-driver interactions being taken into account. The network is not loaded with a continuous flow, but is populated with individual vehicles that must stop at intersections based on green or red light conditions, which may turn based on available gaps, or that may merge based on available gaps. Unfortunately, the amount of information that must be processed is extremely high and the accuracy of the network must be higher than with macroscopic simulators since traffic is constantly interacting with the network geometry.

Macroscopic models look at traffic as a flow. Density, velocity and flow are considered together. The vehicles are lumped and their individual experience and interactions with one another are not taken into account, minimizing the needed number of computations. The result is a faster calculation but less accurate conclusion.

Mesoscopic models balance the two methods. In some models the traffic is looked at macroscopically, except at important points such as tollbooth, merge lanes, and intersections, where they are looked at microscopically. The resulting blend yields good accuracy and reasonable runtime. The accuracy of the software is in the blending of the two pieces of information and also in the identification of important points.

As with all techniques, simulation has some known strengths and weakness. Table 1 summarizes the strength and weakness of traffic simulation models.
Table 2-1: Highway Capacity Manual Simulation Evaluation (HCM 2000)

<table>
<thead>
<tr>
<th>Simulation Modeling Strengths</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Can vary demand over time and space</td>
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<td>• Can model unusual arrival and service patterns that do not follow more traditional mathematical distributions.</td>
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<tr>
<td>• Can experiment off-line without using on-line trial-and-error approach</td>
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<tr>
<td>• Other analytical approaches may not be appropriate</td>
</tr>
<tr>
<td>• Can experiment with new situations that do not exist today</td>
</tr>
<tr>
<td>• Can provide time and space sequence information as well as means and variances</td>
</tr>
<tr>
<td>• Can study system in real time, compressed time, or expanded time</td>
</tr>
<tr>
<td>• Can conduct potentially unsafe experiments without risk to system users</td>
</tr>
<tr>
<td>• Can replicate base conditions for equitable comparison of improvement alternatives</td>
</tr>
<tr>
<td>• Can study the effects of changes on the operation of a system</td>
</tr>
<tr>
<td>• Can handle interacting queuing processes</td>
</tr>
<tr>
<td>• Can transfer un-served queued traffic from one time period to the next</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Simulation Modeling Short comes</th>
</tr>
</thead>
<tbody>
<tr>
<td>• There may be easier ways to solve the problem</td>
</tr>
<tr>
<td>• Simulation models may require verification, calibration, and validation, which, if overlooked, make such models useless or not dependable</td>
</tr>
<tr>
<td>• Development of simulation models requires knowledge in a variety of disciplines, including traffic flow theory, computer programming and operation, probability, decision making, and statistical analysis</td>
</tr>
<tr>
<td>• The simulation model may be difficult for analysts to use because of lack of documentation or need for unique computer facilities</td>
</tr>
<tr>
<td>• Some users may apply simulation models and not understand what they represent</td>
</tr>
<tr>
<td>• Some users may apply simulation models and not know or appreciate model limitations and assumptions</td>
</tr>
<tr>
<td>• Simulation models require considerable input characteristics and data, which may be difficult or impossible to obtain</td>
</tr>
<tr>
<td>• Results may vary slightly each time a model is run</td>
</tr>
</tbody>
</table>

2.1.1 Simulation Choice

The objective of the simulation in this instance is to evaluate flow behavior on I-4, therefore a network simulator will be used. A microscopic simulator is used for the test because the effect to be modeled deals specifically with the interaction of vehicles on the interstate.

2.1.2 Simulation Software Choice

One of the biggest steps left before undertaking the simulation of the road network is the choice of simulation software. Past research in network modeling and in particular
VMS and other ITS applications is examined to determine the appropriate software to use.

Boxill and Yu (2000) evaluated 76 different network traffic simulators of all three types to determine their applicability to studying different ITS issues. The 76 were cut down to the top nine using a set of five criterions. The results are presented in Table 2-2.

1. Credible theories used in the model
2. The model has been tested for real world applications.
3. The ability to output measures of performance such as travel times and speeds.
4. Documentation has indicated incorporation of at least one ITS feature.
5. Model is obtainable to the public
Table 2-2: Summary of Models Based on Initial Criteria (Boxill and Yu, 2000)

<table>
<thead>
<tr>
<th>Model</th>
<th>Criteria</th>
<th>Model</th>
<th>Criteria</th>
<th>Model</th>
<th>Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>AIMSUN2</td>
<td>X X X X X</td>
<td>INTEGRATION</td>
<td>X X X X X</td>
<td>ROADSIM</td>
<td>X X</td>
</tr>
<tr>
<td>ANATOLL</td>
<td>X</td>
<td>INTRAS</td>
<td>X</td>
<td>SATURN</td>
<td>X X</td>
</tr>
<tr>
<td>ARCADY2</td>
<td>X</td>
<td>JAM</td>
<td>X</td>
<td>SHIVA</td>
<td>X X</td>
</tr>
<tr>
<td>AUTOBAHN</td>
<td>X X</td>
<td>KRONOS</td>
<td>X</td>
<td>SIGSIM</td>
<td>X X</td>
</tr>
<tr>
<td>AUTOS</td>
<td>X</td>
<td>MELROSE</td>
<td>X</td>
<td>SIMCO2</td>
<td></td>
</tr>
<tr>
<td>AVENUE</td>
<td>X</td>
<td>METACOR</td>
<td>X X X</td>
<td>SIMDAC</td>
<td>X</td>
</tr>
<tr>
<td>VEHICLESIM</td>
<td>X X</td>
<td>METANET</td>
<td>X X X</td>
<td>SIMNET</td>
<td>X</td>
</tr>
<tr>
<td>CASIMIR</td>
<td>X</td>
<td>METROPOL</td>
<td>X</td>
<td>SIMTRAFFIC</td>
<td></td>
</tr>
<tr>
<td>CONTRAM</td>
<td>X X X</td>
<td>MICROSIM</td>
<td>X X X</td>
<td>SITRA B+</td>
<td>X X</td>
</tr>
<tr>
<td>CORFLO</td>
<td>X X X X X</td>
<td>MICSTRAN</td>
<td>X</td>
<td>SITRAS</td>
<td>X X</td>
</tr>
<tr>
<td>CORSIM</td>
<td>X X X M</td>
<td>MIMIC</td>
<td>X</td>
<td>SMARTAHS</td>
<td>X X</td>
</tr>
<tr>
<td>DRACULA</td>
<td>X X</td>
<td>MITSIM</td>
<td>X X</td>
<td>SMARTPATH</td>
<td>X X</td>
</tr>
<tr>
<td>DYNAMIT</td>
<td>X X</td>
<td>MIXIC</td>
<td>X</td>
<td>SOUND</td>
<td>X X</td>
</tr>
<tr>
<td>DYNASMART</td>
<td>X X</td>
<td>NEMIS</td>
<td>X</td>
<td>SPEACS</td>
<td>X X X</td>
</tr>
<tr>
<td>DYNEMO</td>
<td>X</td>
<td>NETSIM</td>
<td>X</td>
<td>STEER</td>
<td></td>
</tr>
<tr>
<td>FLEXYT II</td>
<td>X X X X</td>
<td>NETFLO</td>
<td>X</td>
<td>STREETSIM</td>
<td>X X</td>
</tr>
<tr>
<td>FOSIM</td>
<td></td>
<td>NETVACI</td>
<td></td>
<td>TEXAS</td>
<td>X X</td>
</tr>
<tr>
<td>FRECON 2</td>
<td>X X X</td>
<td>OLSIM</td>
<td>X</td>
<td>THOREAU</td>
<td>X X</td>
</tr>
<tr>
<td>FREFLO</td>
<td>X</td>
<td>PADSIM</td>
<td>X X</td>
<td>TRAFFICQ</td>
<td>X</td>
</tr>
<tr>
<td>FREEVU</td>
<td>X</td>
<td>PARAMICS</td>
<td>X X X X</td>
<td>TRANSIMS</td>
<td>X X X</td>
</tr>
<tr>
<td>FRESIM</td>
<td>X X X</td>
<td>PASSERII</td>
<td>X X</td>
<td>TRANSYT7F</td>
<td>X X</td>
</tr>
<tr>
<td>FREQ</td>
<td>X</td>
<td>PASSERIV</td>
<td>X X X</td>
<td>TRANSYT10</td>
<td>X X</td>
</tr>
<tr>
<td>HIPERTRANS</td>
<td></td>
<td>PELOPS</td>
<td>X</td>
<td>VEDENS</td>
<td></td>
</tr>
<tr>
<td>HUTSIM</td>
<td>X X X</td>
<td>PHAROS</td>
<td>X</td>
<td>VISSIM</td>
<td>X X</td>
</tr>
<tr>
<td>IVEHICLEUS</td>
<td></td>
<td>PLANSIM-T</td>
<td>X</td>
<td>WATSIM</td>
<td>X X X</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>WEAVSIM</td>
<td>X X</td>
</tr>
</tbody>
</table>

The study then breaks down the remaining nine using more in depth information, as shown in Table 2-3. Strength and weaknesses are weighted, the information is summarized as to uses, and the overall best software is chosen. While the best overall software maybe concluded the best for modeling a specific intelligent transportation project is not.
Table 2-3: Summary of Models Based on In-Depth Criteria (Boxill and Yu, 2000)

<table>
<thead>
<tr>
<th>ITS Features Modeled</th>
<th>AIMSUN 2</th>
<th>CONTRAM</th>
<th>CORELO</th>
<th>CORSIM</th>
<th>FLEXYT II</th>
<th>HUTSIM</th>
<th>INTEGRATION</th>
<th>PARAMICS</th>
<th>VISSIM</th>
</tr>
</thead>
<tbody>
<tr>
<td>Traffic devices</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>Traffic device functions</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Traffic calming</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Driver behavior</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Vehicle interaction</td>
<td>X</td>
<td></td>
<td></td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Congestion pricing</td>
<td></td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Incident</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Queue spillback</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Ramp metering</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Coordinated traffic signals</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Adaptive traffic signals</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Interface w/other ITS algorithms</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
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<td></td>
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</tr>
<tr>
<td>Network conditions</td>
<td>X</td>
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<td></td>
<td></td>
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<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Network flow pattern predictions</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Route guidance</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Integrated simulation</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Other Properties</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Runs on a PC</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Graphical Network Builder</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td></td>
</tr>
<tr>
<td>Graphical Presentation of Results</td>
<td>x</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>Well Documented</td>
<td>X</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>X</td>
<td>X</td>
</tr>
</tbody>
</table>

For VMSs which are closely related to VSL, the simulated technology in this instance, only two software packages are compatible. The AIMSUN 2 software has VMS capability (Boxill and Yu, 2000). However, addition programming is needed to get the VMS to work properly. PARAMICS does not require additional programming according to the study (Boxill and Yu, 2000). AIMSUN’s need for additional programming is brought to light in Boxill and Yu’s study, but turning to PARAMICS user manual shows that PARAMICS, similarly, needs programming (Boxill and Yu, 2000, Quadstone...
Limited, 2003). While either would look to be a perfect choice PARAMICS is chosen due to its scalability and proven background on freeways and urban roads.

2.2 Verifying, Validating, and Calibrating PARAMICS

The accuracy of the results is dependent upon two factors: the quality of the model and the quality of the software in simulating the change. With quality software and a quality calibration and validation process the outcome of the simulation has validity and may be helpful.

The calibration and validation step for these network simulators is at the heart of quite a large amount of research. Many researchers have calibrated and validated programs under certain conditions. If these conditions are followed, then the simulator is assumed to be calibrated and validated. Otherwise, calibrating transportation networks is often done by comparing the traffic counts on major roadways in the simulator to counts taken in the real world then making adjustments to the calibration parameters to make the counts fit. Next, validation occurs when the uncalibrated roads are evaluated. If the differences between the simulated values and the real values are within acceptable limits then the network is validated and simulation may proceed.

2.2.1 Verification

When establishing a validated network the first step is checking to make sure the network that has been built is accurate. The SMART study informs modelers to check that all the allowed turns are being executed, priorities at intersections are being followed, queues are only building at the correct intersections, and signal timings and
phasing is correct in order to insure calibration (2003). Chu et al. (2002) and Gardes et al. (2002) agree that calibration should be carried out by making sure the geometry of the network is modeled correctly, specifying that the links are to be smooth, that signposting distances are long enough, and that the proportion of each type of vehicles is representative. Gardes et al. (2002) goes father, pointing out that if sharp points from poorly aligned nodes occur, kerb positions need to be adjusted and if backups at signpost locations occur, the signposting distance needs to be increased. Gardes et al. (2002) also states that link speeds should be representative of traffic flow not speed limits and that time steps and speed memory need to be increased if high densities are seen to allow for freer flow. While not called out specifically Gardes et al. (2002) is likely referencing ramp awareness distance as well as signposting distance.

The second step is setting up a sound test to compare real world data to modeled data. To do this there needs to be enough model runs with a long enough warm-up period. More than one model run needs to be carried out because of the stochastic nature of the software. Just like in real life each time the program is run different results can be generated. Mohammed and Abdulhai (2002) addressed the need for a statistically sound model run and concluded that thirty runs are needed to ensure the results. The conclusion was arrived at despite statically theory (t-tests), which shows fewer that thirty runs are adequate. No explanation is offered as to the importance of 30 runs. Mohammed and Abdulhai only stated that more tests needed to be run in order to get good numbers.

A warm-up period allows the network to be loaded with vehicles. Imagine a completely empty network and then suddenly rush hour traffic being released. It will take time for the system to come to an equilibrium, and thus to yield accurate numbers.
Abdulhai et al. (2002) used a warm-up period of thirty-minutes while Cheu et al. (2002) used a five-minute warm-up period, a five-minute incident free period, a 15-minute incident period, and finally 35-minute post-incident period. No consensus method appears to be found in the literature.

The third step is changing the driver and network characteristic in order to get the behaviors exhibited by the real life network. In order to get a good baseline literature is consulted. Chu et al. (2002), Abdulhai et al. (2002), Cheu et al. (2002), and Bertini et al. (2002) noted that in PARAMICS the mean target headway and mean driver’s reaction time need to be calibrated based on the area’s drivers for accurate flow characteristics, but did not reveal the values they used. Gardes et al. (2002), Abdulhai (2002), and Lee et al. (1999) stated their numbers as 1 second and .6 seconds, 1.65 seconds and .42 seconds, and .615 seconds and .415 seconds, respectively. The key difference in the studies was their locations, according to Gardes et al. (2002). No studies have been completed and published within Florida to determine the proper timings, however, the three sets shown will be tested and the most accurate will be used. Chu et al. (2002) also stated that vehicle following, lane-changing models, and acceleration and deceleration values are important to calibration. The acceleration and deceleration values between 2.8 m/s² and 5.0 m/s² had the best results for Chu et al. (2002). Other factors involved in verification are the time step, aggressiveness, and minimum gap values (Abdulhai et al., 2002, Cheu et al., 2002).
2.2.2 Calibration

The forth step involves changing the origin-destination matrix and some of the network factors in order to produce a simulation that numerically checks out with real world numbers. Two thirds of the counts in a network are, typically, used to calibrate. Table 2-4 summaries what factors four different studies used in order to calibrate their networks. A number of the variables that were changed in the verification step can be adjusted during the calibration step. The biggest difference between verification and calibration is that calibration looks at numbers while verification looks at behaviors.

Table 2-4: Overview of Factors Used in Calibration as Noted by the Literature

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Traffic Flows</td>
<td>Cycle Time</td>
<td>Travel Times</td>
<td>Observed Count Data</td>
</tr>
<tr>
<td>Queue Lengths</td>
<td>Average Transit Speed</td>
<td>Traffic Volumes</td>
<td>Journey Time Data</td>
</tr>
<tr>
<td>Delays</td>
<td>Average Headway</td>
<td>Local Speeds</td>
<td>Automatic Traffic Count Data</td>
</tr>
<tr>
<td>Traffic Speeds</td>
<td>Headway Standard Deviation</td>
<td>Floating Vehicle Data</td>
<td>Video Count Data</td>
</tr>
<tr>
<td>Traffic Densities</td>
<td>Average Service Frequency</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Travel Times</td>
<td>Person Throughput</td>
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</tr>
<tr>
<td></td>
<td>Transit Vehicle Bunching</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Fleet Size</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Overall Average Speed</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

2.2.3 Validation

The fifth step in developing a network is validation. In this case the one third of roadways not used to calibrate are compared with the simulated network counts to generate an error rate for the network. If the error rate is acceptable then the roadway is validated. Otherwise, everything starts over again. It must be stressed that roads that are used to calibrate cannot be used to validate. The two must be independent.
2.3 Variable Speed Limits

VMS can be used to display changing speed limits on roadways. While somewhat different than a typical VMS display, a VSL still requires the same reaction by a road user. The user must read, clearly understand, and react to the message. The primary differences between VMSs and VSL are that VSL display a limited amount of information and that VSL’s information is more likely to be enforceable.

2.3.1 Who Uses VSL


2.3.2 Why Use VSL

Like other advanced technologies VSL, or VSL, cost considerably more than the conventional alternatives. However, they also offer additional benefits. VSL biggest benefit is flexibility. When surrounding conditions change, worsening driving conditions, VSL can be adjust. The most common reason for reducing speed limits are weather related including wind speed, visibility, humidity, rain intensity and total rainfall. (Sisiopiku 2001). Current literature focuses on three weather conditions snow (Placer 2001, Senn 2004, Pilli-Sivola 2000, Pilli-Sivola 2004), fog or reduced visibility (Senn
VSL are also used in conjunction with work zones, alerting drivers of impending construction, protecting workers. Less common, but becoming more popular, is the use of VSL in order to induce changes in driver behavior. The two common goals when attempting to change driving behaviors is increasing average headways and reducing variances in speed (Borrough, 1997, Pilli-Sivola, 2004). Another potential goal of VSL is the controlling of roadways costs. One manner of controlling cost is using VSL to keep queues from building by slowing traffic before it reaches a queue, effectively reducing the intensity of traffic jams (Borrough, 1997). Borrough, (1997) found that through the use of VSL and strong enforcement (video cameras) that the number of crashes could be greatly reduced (28% over 18 months). The effect was attributed to not only a smoothing of traffic conditions through longer following distances, but also through reducing the number of lane changes during congestion (Borrough, 1997). Roadway cost can also be reduced by targeting heavy vehicles, which cause the majority of wear on roadways, to lower their speeds, thereby reducing road-wear (Panter, 2002).

VSL are commonly implemented in order to reduce variability in travel speeds. Less variability leads to fewer crashes, fewer short headways, and lower mean speeds according to Ha et al. (2003). The study consisted of speed and headway information being gathered 500m before, 500m after, and at the location of an Automatic Speed Enforcement System, or ASE, in Korea (Ha et al., 2003).

Smulder’s findings (1990) agree that fewer crashes occur, speeds are lowered, and fewer short headways are observed, but uses a simulation to show that all the results are due to the fewer short headways. Fewer crashes, lower speeds, and less variability result
from the fewer short headways in just the left lane. The simulation is a mathematical model of data collected on freeways in the Netherlands.

Whether fewer short headways cause fewer crashes or lower speeds lead to fewer crashes the result is the same. Hoogen and Smulder’s study which does not attempt to assign cause also determines that fewer crash result from the implementation of VSL. The study is based on field measurements along a single freeway in the Netherlands where the speed is restricted to being either 70 or 90 km/hr. Volume and speed measurement are taken across interval with less than 1 km spacing, increasing the scope beyond that observed by Ha et al. (2003).

A study by Rämä (1999) in Finland again shows that VSL leads to lower speeds and less variability, but the study is unique in that there was no difference in headway observed. This study makes use of two roadways with similar speed limits, geometries, and weather condition. One of the roadways has VSL and the other has static signs. Differences in driver’s behaviors on the two roadways lead to conclusions of lower speed and variability being caused by VSL.

Adding in-vehicle information or VMS to inform drivers of causes along with changes in speed limit would seem to improve adherence to the speed limit. The idea is supported by studies by Rutley (1972), Webb (1980), and Tenkinte (1990) which show that incorporating causes into VMS advisory information increased driver responses. However, Boyle and Mannering’s et al. study (2004) indicates otherwise. They showed that speed variability increases in two out of three cases when information was included with speed advisory information. The study makes use of four ways of delivering
information to participants in a driver simulator: in-vehicle, VMS, both, and neither. The driver’s response is analyzed to produce the study’s results.

The studies show varied method of evaluating the effectiveness of VSL’s. Each points to there being a decrease in crashes, even if the mechanism is subject to debate. Boyle and Mannering (2004) warn that causal information should not be included in VSL.

2.3.3 Method of Determining Displayed Speed

The idea behind VSL is simple. Conditions change and speed limits need to change with them in order to give accurate guidance. But different groups determine what threshold triggers a change in speed limits differently. Some methods that are used to decide whether to change speed limits include fuzzy logic (Placer, 2001, Sisiopiku, 2001, Harrill, 1999), 1 min data averages (Sisiopiku, 2001), 5 min averages (Rajagopal et al., 2002), one to one correspondence (Sisiopiku, 2001), logic tree (Sisiopiku, 2001), volumes (Sisiopiku, 2001), and weight-in-motion (Sisiopiku, 2001). Fuzzy logic is the application of percent yes, or percent no rather than yes or no to determine an outcome. Data averages give information of variability in speeds. Correspondences matrix are simple one to one correlations with measure inputs. Logic trees involve moving along predetermined paths to reach a conclusion. Weight-in-motion allows for weight measurement without restriction travel, with allows for weight based speed limits.
2.3.4 Sign Spacing

Once the display speed has been chosen using the appropriate algorithm the next problem is determining how far apart to space signs in order to get the proper number of vehicles to adhere to the speed limit. Studies from Finland show a spacing of approximately every .5 km (Sisiopiku, 2001). On the other end of the spectrum signs are spaced every 2 km in New Jersey, Washington State, Germany (Sisiopiku, 2001). Most lie between the two extremes at a distance of every 1 km (Sisiopiku, 2001). The logic for determining the spacing is not discussed, which is seems unusual. Placing signs too close together would waste money, while placing them too far apart could cause drivers not to follow their advisement. The subject should be studied, with consideration given for the surrounding clutter.

2.3.5 Method of Investigation VSL Effects

The studies looked at flow characteristic and not driver specific results, therefore driver simulators where not used in any of the studies. It should be remember that driver simulators would provide important information on VSL. One example of an application of driver simulators is using the simulator to generate acceptance percentages of drivers toward VSL to be characterized in microsimulators. In this way, a microsimulator would be able to provide meaningful data when used to analyze the usefulness of VSL. Some studies have already used a microsimulation (MITsim), without a fully validated VSL acceptance percentage, to study VSL (Rajagopal et al., 2002, Hegyi et al., 2003). No information was given on how acceptance percentage for VSL is determined, or if VSL were simulated using VMS or if the roadways speed limit are changed.

2.3.6 Downfalls

Like any option VSL are not without their problems. Sisiopiku mentions two problems that could be caused by providing speed limits that change with conditions. First, when signs are not operational it might leave drivers to conclude that nothing is wrong (no weather problems) (Sisiopiku, 2001). Their reliance on the system’s accuracy might cause speeds in excess of those present with static signs (because they is less unsure of impending conditions), raising their chance of a crash (Sisiopiku, 2001). Second, the sign takes away some responsibility in decision making from driver, and places it on the engineer (Sisiopiku, 2001). Currently when an incident happens, such as if it begins to rain excessively, a driver needs to react by reducing the speed appropriately. If drivers do not then they can be ticketed even if they are traveling under the speed limit. VSL are supposed to reduce their displayed speed, giving drivers the appropriate pace to drive during these conditions. If the display speed is too high during an incident and is followed by a driver that crashes then the system could absorb some of the blame.

2.4 PARAMICS Simulation Software

The PARAMICS’ website describes the software as:
“Paramics is a suite of microscopic simulation modules providing a powerful, integrated platform for modelling a complete range of real world traffic and transportation problems. Paramics modules work together to improve usability, integration, and productivity allowing users, and their clients, to get added value from the modelling process. Paramics is fully scaleable and designed to handle scenarios as wide-ranging as a single intersection, through to a congested freeway or the modelling of an entire city’s traffic system. (Paramics-Online, 2005)”

It goes on to say that:

“Paramics can be applied to any aspect of a modern transportation network with equal ease and clarity. Some of the most common application areas include:

- Priority intersections, signalized intersections and roundabouts
- Urban cities, corridor studies and congested freeways
- Public transport and light rail
- Actuated signal control
- Special user groups e.g. HOV, emergency vehicles, transit priority
- ITS elements e.g. ramp metering, VMS, route control, lane usage, freeway speed control
- Work zones, event management and pollution modelling
- Public presentation. (Paramics-Online, 2005)”

What the site does not say it that PARAMICS is similar to a black box application. While the manual states that the software makes use of multiple algorithms such as gap acceptance, car following, and vehicle release rate algorithms, it does not tell the user what the algorithms are (Quadstone Limited, 2004). This becomes an important consideration when validation steps are undertaken. In this thesis queue distance and queue speed are changed in order to validation the network, but the meaning of the two factors is not clearly spelled out in the User Guide, leaving the author to guess at what value the factors should take on (Quadstone Limited, 2004). In the end the author simply treated the software as a black box and tried a variety of different values until a combination looked to be best.
CHAPTER THREE: METHODOLOGY

3.1 Scope of the Thesis

The map in Figure 3-1 shows the area that is studied. A few major roads are labeled to help familiarize the reader with the study area.

3.2 Overlay Generation

The overlay that is used to draw the I-4 corridor is generated by a combination of aerial photography, obtained from the Orange County Property Appraiser’s Office and the Seminole County Property Appraiser’s Office, and AutoCAD drawings, obtained from the Orange County Transportation GIS department.

3.2.1 AutoCAD

Knowing that the I-4 corridor will be drawn in PARAMICS and that the program accepts .dxf files, an overlay is drawn in AutoCAD and then saved as a .dxf file. An AutoCAD drawing of the entire Orange County Network was obtained from the Greater Orlando Aviation Authority (GOAA) and a drawing of I-4 from Orange Blossom Trail to the Seminole County-Orange County border was obtained from the Center for Advanced Transportation System Simulation (CATSS) office at the University of Central Florida (UCF).

The drawings, after being overlaid, show good agreement. The Orange County Network shows only centerlines and does not give a clear indication as to the location of
Figure 3-1: I-4 with major road and station locations.
on and off ramps, accelerations and deceleration lane lengths, or exact geometries (turn radius, median locations). CATSS’s drawing does an adequate job of displaying geometries and on and off ramp locations, but does not show acceleration and deceleration lane lengths. Unfortunately, the drawing is smaller than the length of I-4 to be simulated. To regain the missing links of I-4 aerial photography is used.

3.2.2 Aerial Photographs

Orange County aerials must be purchased by section from the county office. Due to the cost, only the aerials that added to the I-4 drawing were selected (from Orange Blossom Trail to the Orange County-Osceola County border). Seminole County GIS provided aerials free of charge. The aerials are downloaded from Seminole County’s GIS website (http://www.seminolecountyfl.gov/it/programming/gis/pcl.asp) for the length of I-4. After obtaining all the different sections, all of the aerials are resized and combined in Photoshop. To see a screenshot of the aerial photography in AutoCAD see Figure 3-2 and Figure 3-3.

3.2.3 As-Built Drawings

After using the aerials, merge and diverge lengths remain the only geometric information missing. At first it may seem as though the lengths could be determined using aerials, but the accuracy of the aerials is too low. In order to determine these lengths as-built drawings are consulted. The drawings, available at the CATSS office at UCF, provide information for I-4 from Orange Blossom Trail to the Orange County-Seminole County border. Other as-builts could not be acquired, so the scope of the
The project has to be downgraded to Orange Blossom Trail in the south. The northern border remained Lake Mary Boulevard, because all of the interchanges up to Lake Mary have long acceleration and deceleration lanes and their lengths do not affect the performance of the vehicles moving through them.

![Figure 3-2: View of original AutoCAD file with xref aerial Orange County](image)

All long length ramps, greater than 750 ft, can be assumed to have a length of 750 ft, which is enough length in PARAMICS to allow for smooth behavior.

### 3.3 Overlay Importation

The size of the new combined aerial file proved to be too much for AutoCAD to handle. Therefore, the combined file needed to be split into thirds. Three AutoCAD files are generated to handle each of the overlays. The road edges are then redrawn within
AutoCAD. The aerial photos are then removed, and the resulting drawing files combined.

3.3.1 Zero lay required

The combined drawing files, unfortunately, could not be imported into PARAMICS. One of PARAMICS quirks is that it does not accept X-Ref layers into its overlays. As a result, each of the aerial photos has to be imported separately, generating five overlays (one for each aerial photos, making three in Orange County and one in Seminole County, and one for the AutoCAD file described above in the AutoCAD subsection).

Figure 3-3: View of original AutoCAD file with xref aerial of Orange and Seminole County
3.4 Definitions

Signposting distance, ramp awareness distance, kerb points, and speed memory are all terms associated with the PARAMICS software. Signposting distance is a link attribute that shows how far in advance of a particular geometry, such as an off ramp, that a driver becomes aware of that geometry. Ramp awareness distance is the distance before a ramp that vehicles begin to increase their headway to allow for merging behavior. Kerb points are similar to hot grips in AutoCAD. They are located at the four vertices of a link and control the location of the outer edge of the lanes. While lane width is specified in PARAMICS and almost always aligns links, correctly link modifiers such as curves or slip lanes (deceleration lanes) can alter otherwise smooth fit. Speed memory refers to how often the vehicles’ information (location, speed, headway, acceleration) is updated. PARAMICS is a continuous simulator, not a discrete event simulator and thus requires this parameter.

Some other key definitions are the factors, called core variables, PARAMICS uses to affect driver behavior. Mean reaction time, target headway, queue speed, queue distance make up the core variables for PARAMICS. Driver characteristics aggressiveness and awareness can also be used to a lesser extent to change driver behavior. Mean reaction time is the average amount of time it takes a driver to react to being too close or too far away from the vehicle in front of them. The target headway is the headway, in seconds, which a driver attempts to maintain in non-queuing conditions. Queue speed is the speed under which a queue is formed. In other words queuing behavior is exhibited once a vehicle goes beneath this speed. Queue distance is the distance between two vehicles that once obtained causes queue behavior to end. Each of the specified numbers is an average
of a distribution, not a value given to every vehicle. In other words specifying a mean reaction time of 1.3 seconds means that some drivers will have a reaction time above 1.3 seconds and some below 1.3 seconds, but the average works out close to 1.3 seconds. Driver aggressiveness helps to determine the allocation of reaction time and mean reaction time. Drivers are given an aggressiveness level between 1 and 10, with 10 being a high aggressiveness. High aggressiveness is more likely to correspond to lower headways and reaction times. Driver awareness has to do with the likeliness that a driver will miss an off ramp and have to correct there route. In this network there is only one path from one zone to another, so awareness does not come into play.

3.5 Drawing Network

Once the overlay is generated the next step is drawing the roads in PARAMICS. In order to draw, lane categories need to be identified, nextlane behaviors need to be specified, and priorities need to be assigned.

3.5.1 Road Categories

Finding the differences in speed limits and number of lanes proved to be a challenge, even though there is little variability along I-4. The Florida Department of Transportation (FDOT) was contacted first to obtain the information. They provided screen shots from their 2002 Florida Traffic Information (FTI) CD with speed limits written on them. The speed limit is shown to be 55 for the length of I-4. Field inspection shows this to be inaccurate. Eventually, field data is used to determine the number of lanes and speed limits north of the East-West Expressway. Speed limits and number of
lanes south of the East-West Expressway was determined through conversation with the FDOT’s Construction Project Manager, Seema Jagtap. Figure 3-4 shows a screenshot of PARAMICS’s Configuration Manager.

![Category window](image)

**Figure 3-4: Category window**

### 3.5.2 Zones

In other PARAMICS drawings of I-4, combined zones are used to minimize the complexity of OD Matrixes, and are preferred, if accuracy is not compromised, in the process. However, with the high sensitivity of the crash prediction algorithm, the chance of misrepresenting the traffic behavior, by oversimplifying the OD, is too high to risk. Instead, every on ramp starts with a zone and every off ramp ends with a zone. No zones are combined in order to simplify the problem.
3.5.3 Two Roads versus One Road

After the first round of drawing is complete, there are a few touchup issues. The first concerns the ramps along I-4. The ramps were originally drawn without regard to awareness distance and deceleration lane lengths and modeled with extra lanes on the freeway. Both decisions proved to be errors. Using extra lanes on the interstate instead of slip ramps leads to confusing driver behavior that does reflect real world scenarios. The awareness distance needs to be positive in order for vehicles traveling on the interstate to allow gaps to merging traffic from onramps. Without positive awareness distances large backups form on the mainline and the onramp. The deceleration lanes are referred to as slip lanes within PARAMICS and have their own designation within the links file.

Getting properly functioning on and off ramps involves more than specifying awareness distances and slip lane lengths, the two functions are related through node positioning and often come into conflict. Slip lanes stop at the end of a link and start a distance specified in the links’ characteristics, back from the end of the link. On ramps join in the middle of links with the awareness distance extending backward and acceleration lane extending forward. Neither the awareness distance nor the merge distance can extend beyond the link to which the on ramp connects. Conflict results when on and off ramps occur at, or near, the same location on opposite sides of the road. The off ramp needs a node at its location, while the on ramp requires that the node be placed at least a distance equal to the awareness distance upstream or the acceleration lane distance downstream. Figure 3-5 shows screenshots of links that have been specified correctly.
When the ramps are redrawn nextlane information has to be updated, because the default setting allows for left turns on the interstate. In other words, westbound traffic is allowed to use eastbound ramps, and vice versa, a huge modeling problem.

The node location problem and the need to control nextlane information can be avoided all together by using two separate one-way roads. In essence, that is how an interstate operates, so conceptually it is not a problem. However, setting up separate roads calls for extra nodes and links which all have to be tracked during runs. If the
network is sensitive enough to be affected by changing ramp locations, awareness distances, or acceleration or deceleration lane lengths then the extra memory and therefore runtime is justified. Because the network is an interstate, the model is redrawn with two roads: one running north (east) and one running south (west).

Figure 3-6: Kerb properties of a northern South Street Ramps

3.5.4 Left Ramps

South Street in downtown Orlando has three left ramps leading to and from it. PARAMICS does not allow for left hand ramps to be modeled. Contacting PARAMICS User Support, they suggest checking the wide entrance and wide exit characteristics on the link that has a left ramp. Checking the characteristics did not lead to changes in the drivers’ behavior, so the suggestion of PARAMICS Support is not used. Instead, the ramp is modeled as a continuation of the interstate with a lower speed limit. Luckily, the roads have low enough counts that the lane changing behavior (which is different for
ramps than lanes) does not have a dramatic impact on the overall system. If the Kirkman Road Exit had been modeled this assumption may not have been able to be made. Kirkman Road connects I-4 to Universal Studios, Islands of Adventure, Wet and Wild, and other tourist locations; therefore, it is a heavily traveled ramp. A screenshot of the South Street Interchange is shown in Figure 3-6.

3.5.5 Loop Detectors

3.5.5.1 Location Determination

A second drawing based on data obtained from the FDOT RCI database indicates loop locations that needed to be incorporated into the simulation. The drawing, which spans the entire length of the simulation project, is overlaid onto the network to find loop location. The location of loop detector is critical, because they are the eyes for the crash prediction algorithm.

3.5.5.2 Number of Lanes

The loop detectors on I-4 do not span all of the lanes. Instead, they only reach the three left most lanes, in some sections there are 4 loops, but only 3 are consistently archived. Therefore the crash prediction algorithm uses data from only 3 lanes (Abdel-Aty, 2005). Instead of making sure each loop detector in the model only spans three lanes, the post processing only calls for counts, occupancies, and densities from the inner most three lanes even though the model records all lanes. The loops are drawn to
encompass all the lanes, whether three or four, on the cross-section of I-4 where they are located. The process simplifies the drawing.

3.6 Calibration of the OD Matrix

3.6.1 Data Sources

To construct the Origin Destination Matrix of the I-4 corridor six sources are looked into: the 2002 Florida Traffic Information (FTI) CD, 2003 FTI CD, the 2002 AADT provided on the FDOT website, the 2003 AADT provided on the FDOT website, a 2002 study constructed by URS, and the 2003 Florida Standard Urban Transportation Model Structure (FSUTMS) model output. When comparing the sources it becomes apparent that the data on the FDOT website is the same data as the FTI CD, dropping the number of sources to four. The FTI CD data is not used because the FDOT website has a better format.

Because the number of lanes on I-4 has been increased to 4 on the most heavily traffic portions and only three lanes have loop detectors, the counts would have to be adjusted to avoid underestimation. It could be assumed that the forth lane could be interpreted using the first three lanes data. The estimated values of the forth lane detectors could be higher or lower than the real value and correcting it could introduce more error. In any case, the workload would be too large and error prone, making the method undesirable.
3.6.2 Data Sources Compared

A non-parametric analysis is used to compare the data sources to see if they are different and if a decision as to which one should be used needs to be made. The Friedman Test shows a significant difference between the four data sources. Using the ranks it shows that the URS data is significantly higher than the FDOT data, which is unexpected. As expected, the 2002 FDOT data is significantly lower than the 2003 FDOT data. The 2003 FDOT data is chosen. It is the most recent data and from a reliable source.

![Figure 3-7: Zones view at the Orange Blossom Trail Interchange.](image)

3.6.3 Finding Zeros

With all but two of the zones (the ones on the I-4 terminuses) in the modeling being one way, the OD matrix is mostly populated with zeros. In determining the location of all the zeros, all rows that are associated with off ramps are given zeros. Next, all columns associated with on ramps are given zeros. Then, all indices associated with on ramps and off ramps on different roads (different directions on the interstate) are given zeros. Finally, all off ramps behind a given on ramp are given zeros.
3.6.4 Gravity Model

Research Conducted at UCF used the gravity model in order to sort out an OD, but did not have the accuracy that the heuristically corrected OD showed (Kanike, 2003). As a result, for this corridor the gravity model will start the process and then the resulting OD will be corrected. Equation 3-1 below shows the gravity model.

\[
\text{Count} = \frac{P_i \cdot (A_j \cdot T_{ij})}{\sum A_i \cdot T_{ij}}
\]  

(3-1)

\(P_i\): Number of trips produced by the zone  
\(A_i\): Number of arrivals received by the zone \(i\)  
\(T_{ij}\): Time of the trip from zone \(i\) to zone \(j\)

Notice that the equation does not include a correction for socioeconomic factors. Instead the equation relies on the on and off ramp peak hour counts and the travel time between ramps. The peak hour is determined using the AADT and the K factor for each ramp. The CATSS office at UCF provides the travel time data.

3.6.5 Heuristically Corrected

The output of the gravity model shows several problems. First, there are large difference between the sum of the columns and the counts at that off ramp. To correct this all the counts are adjusted until the error is minimized. The adjustments are done in order to give values that minimize error. The steps are as follows:
1. When rows and columns both are either too low or too high then the count is adjusted appropriately.
2. When a column is too high then the count is adjusted downward.
3. If both a column and row for a cell is too low the count is adjusted upward.
4. Steps 2 and 3 are repeated.

It should be noted that the correction steps are preformed at all other ramps besides the East-West Expressway first and then the East-West Expressway counts are adjusted. It is assumed that in the calibration and validation step the values will be changed in order to get accurate value.

3.6.6 Remaining Error

The remaining error, the difference between the peak hour ramp counts and the OD matrix, after adjustment, is 3.90% and has been concentrated in the zones leading to the East-West Expressway. The error is concentrated here because it has the largest counts and thus will have the smallest effect on the outcome. The final OD for the 73 zones, in terms of vehicles per hour, may be seen in Appendix C.

3.6.7 Measurements

PARAMICS requires that the measurements, or outputs of each network, be specified. In each case only the minimum data is collected in order to maximize run speed and minimize runtime. Figure 3-8 shows the measurements that maybe collected using loop data.
3.6.7.1 Occupancy

The occupancy measurement in PARAMICS is part of the instantaneous measures generated by loops. The output is not grouped and averaged into neat thirty second intervals as are needed to predict crashes, instead every time a vehicle passes over a loop the time and the amount of time spend over the detector is recorded. This outputted information is organized into thirty-second packets afterwards, using a macro in Excel.

3.6.7.2 Flow

Flow can be measured as a thirty second average. The measurement does not take place at the detector, even though the document that the information is reported is labeled according to the detector. Instead, the measurement takes place at the end of the link that the detector is on. It would seem that the best way to account for the discrepancy would be to place the detector at the very end of the link, or to create a node just past the end of the loop detector. However, the PARAMICS modeler user guide warns that double...
counting may occur if this approach is used. It advises that a distance equal to twice the length of the largest vehicle in the network should be used in order to prevent this double counting. Twice the length of the largest truck in the network works out to be about 80ft. Also, when nodes are adjusted the loops must be redrawn, making the technique tedious and time consuming.

The instantaneous flow is calculated by PARAMICS by inverting the headways of two consecutive vehicles. The flows categorized by the vehicle type are extreme and difficult to work with therefore the instantaneous flow is not used.

By using the data generated by the occupancy, the number of vehicles that pass over a detector within a thirty-second interval are known. By dividing that count by the thirty seconds the hourly flow is obtained and can be compared to existing data. Using this method the amount of information collected by PARAMICS is minimized, speeding up the runtime.

3.6.7.3 Average speed

The average speed is collected in the same manner as the occupancy. The average speed measurement in PARAMICS is part of the instantaneous measures generated by loops. The output, similar to occupancy, is not grouped and averaged into neat thirty second intervals as needed to predict crashes. Instead, every time a vehicle passes over a loop the time the speed of the vehicle is recorded. This information is organized into thirty-second packets afterwards, using a macro in Excel.
3.7 Verification

Generally, verification takes place before calibration, but vehicles needed to be added to the network in order for the verification step to take place. As a result the steps are out of their traditional order. Often times the verification step is confused with the validation and calibration steps. Verification is meant simply to observe that the network’s behavior insure it looks similar to the behavior seen in the real world. Numbers do not need to be consulted. This level of exacting is reserved for the validation step. A summary of the verification steps can be viewed in Appendix D.

3.7.1 Merging Behavior

Awareness distance must be long enough in order to allow merging behavior to be smooth. Otherwise, large backups can clog the entire length of the simulation within a few minutes. Instead of checking each distance by hand, a simulation run is completed and problems with awareness distance become immediately obvious, due to the large backup that is formed, starting at the ramp with an error. Figure 3-9 shows typical ramp properties.
3.7.2 Absence of Prohibited Turns

Because the network was drawn using two roads (one for each direction) instead of one road, there are no prohibited turns. All turns also are required to be major priority, because there are no conflicting movements. Figure 3-10 shows how to specify allowable turns.
3.7.3 Smooth Behavior between Links

If lanes do not match up, or if a minor priority is given to through movements, vehicles do not travel through the roads smoothly. To correct this all priority settings are checked. Also, all kerb points are checked to make sure they are smooth. Specific attention is paid to off ramps. Because of the way they are drawn, it is common for kerb settings to be off.

3.7.4 Appropriate Nextlanes

PARAMICS tends to allow all lanes to turn off at an exit ramp. To correct this all nextlanes settings are changed so that only the outside lane, along with the sliplane, can use the off ramp. Figure 3-11 shows the nextlanes function in PARAMICS.
3.7.5 OD Adjustment

Having completed the heuristically corrected origin destination matrix further revisions are made when geometries cannot change improperly exhibited behavior. Making sure the changes are in areas that the heuristically corrected values were made, the variance from the gravity model can be minimized while a corridor that behaves correctly is obtained.

3.7.6 Determining Running Time

As part of the scope of the simulation the run length of the simulation was decided to be three peak hours in the afternoon. The time is chosen to be approximately 4:00pm to 7:00pm. However, because the network begins without any vehicles on the roadway there needs to be a warm-up period, unless it is assumed that the warm-up period is short enough that the values generated will be overwhelmed by the other data. To test this, the
number of vehicle within the network is tracked and plotted. Figure 3-12 shows that there is a significant warm-up period. Also that steady state range of approximately 1400 to 1600 vehicles per hour is reached. The time period that first reaches this range is approximately 15 minutes into the simulation. Therefore the start of the simulation is pushed back to 3:45pm, and the 3:45pm to 4:00pm data is disregarded.

![Loading of Vehicles on I-4 Network](image)

**Figure 3-12: Loading of vehicles versus simulated time (The simulation begins at 3:45pm)**

### 3.7.7 Ramp Modeling

The lights at the end of ramps and the number of lanes of ramps are not modeled. The reason they are not modeled is because the ramps are assume to have the capacity to handle all the loads present. As a result each ramp is one lane wide and ends with a zone that absorbs the vehicles when they reach the ramps. This is a simplification that should be addressed in future work.
3.8 Validation

The mainline counts provided by the 2003 FTI CD data allow for the network to be validated. The OD is not adjusted to account for any difference in the counts. Instead the core parameters within PARAMICS are changed.

3.8.1 Mean Target Headway & Mean Driver’s Reaction Time

The mean target headway and mean driver’s reaction time are changed to values found in the literature review section. The values along with the corresponding results are shown in Table 3-1. For a recap, Gardes et al. (2002), Addulhai (2002), and Lee et al. (1999) stated their numbers as 1 second and .6 seconds, 1.65 seconds and .42 seconds, and .615 seconds and .415 seconds, respectively.

![Figure 3-13: The core behavior variables as shown in the Configuration window](image-url)
3.8.2 Queuing Distance

The default queuing distance is 32.81 ft. To see the effect of the queuing distance an alternative distance is chosen. The value, 20 ft, does not have any significance, other than to test the effect of queuing distance. The results of the different queuing distances are shown in Table 3-1. The results show that queuing distance does not have a major effect on flows except for the cases with low mean target headways. It appears that when headways are low enough there is an interplay with the two factors.

3.8.3 Queuing Speed

The default queuing speed is 4.47 miles per hour. To see the effect of the queuing speed an alternative speed is chosen. The value, 24.47 miles per hour, does not have any significance, other than to test the effect of queuing speed. The results of the different queuing speeds are shown in Table 3-1. The results show that increasing the queue speed decreases the percent error. Most of the error in the model is due to the low flow numbers, so, taking into consideration what queue speed controls, it is plausible to assume that the overall effect of increasing the queue speed is to increase flows when congestion takes place. Queue speed is a significant variable and produces a large change in flow.
Table 3-1: Core variable specification and their resulting error percentages

<table>
<thead>
<tr>
<th>Run Number</th>
<th>Headway</th>
<th>Reaction Time</th>
<th>Queue distance</th>
<th>Queue speed</th>
<th>Error Rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.00 sec</td>
<td>0.60 sec</td>
<td>31.80 ft</td>
<td>4.470 mph</td>
<td>17.14</td>
</tr>
<tr>
<td>2</td>
<td>1.65 sec</td>
<td>0.42 sec</td>
<td>31.80 ft</td>
<td>4.470 mph</td>
<td>20.5</td>
</tr>
<tr>
<td>3</td>
<td>0.61 sec</td>
<td>0.42 sec</td>
<td>31.80 ft</td>
<td>4.470 mph</td>
<td>18.91</td>
</tr>
<tr>
<td>4</td>
<td>0.50 sec</td>
<td>0.50 sec</td>
<td>31.80 ft</td>
<td>4.470 mph</td>
<td>14.19</td>
</tr>
<tr>
<td>5</td>
<td>1.00 sec</td>
<td>0.60 sec</td>
<td>31.80 ft</td>
<td>24.470 mph</td>
<td>14.77</td>
</tr>
<tr>
<td>6</td>
<td>1.65 sec</td>
<td>0.42 sec</td>
<td>31.80 ft</td>
<td>24.470 mph</td>
<td>19.77</td>
</tr>
<tr>
<td>7</td>
<td>0.61 sec</td>
<td>0.42 sec</td>
<td>31.80 ft</td>
<td>24.470 mph</td>
<td>13.52</td>
</tr>
<tr>
<td>8</td>
<td>0.50 sec</td>
<td>0.50 sec</td>
<td>31.80 ft</td>
<td>24.470 mph</td>
<td>13.19</td>
</tr>
<tr>
<td>9</td>
<td>1.00 sec</td>
<td>0.60 sec</td>
<td>20 ft</td>
<td>4.470 mph</td>
<td>17.14</td>
</tr>
<tr>
<td>10</td>
<td>1.65 sec</td>
<td>0.42 sec</td>
<td>20 ft</td>
<td>4.470 mph</td>
<td>20.5</td>
</tr>
<tr>
<td>11</td>
<td>0.61 sec</td>
<td>0.42 sec</td>
<td>20 ft</td>
<td>4.470 mph</td>
<td>14.54</td>
</tr>
<tr>
<td>12</td>
<td>0.50 sec</td>
<td>0.50 sec</td>
<td>20 ft</td>
<td>4.470 mph</td>
<td>14.36</td>
</tr>
<tr>
<td>13</td>
<td>1.00 sec</td>
<td>0.60 sec</td>
<td>20 ft</td>
<td>24.470 mph</td>
<td>14.77</td>
</tr>
<tr>
<td>14</td>
<td>1.65 sec</td>
<td>0.42 sec</td>
<td>20 ft</td>
<td>24.470 mph</td>
<td>19.77</td>
</tr>
<tr>
<td>15</td>
<td>0.61 sec</td>
<td>0.42 sec</td>
<td>20 ft</td>
<td>24.470 mph</td>
<td>13.52</td>
</tr>
<tr>
<td>16</td>
<td>0.50 sec</td>
<td>0.50 sec</td>
<td>20 ft</td>
<td>24.470 mph</td>
<td>13.19</td>
</tr>
</tbody>
</table>

3.8.4 Inspecting Speeds

While the flows were verifying rather well, the speeds, the crucial output of the simulation, looked uncharacteristic. Generally, a high traffic roadway runs in one of two states, uncongested or congested. In the uncongested states the speeds are similar to speed seen on low traffic roadways of similar geometry; with limited access roads this means around 55 mph. In the congested state speed take a nosedive, dropping down to around half the uncongested speed if not less. Looking at the transition of a roadway from uncongested to congested over time. The change happens at a specific instance. At one time the average speed at a given point is 57 mph and then at the same point 30 seconds later it is 15 mph. Once the congestion takes hold and forms a queue. The queue grows until the number of vehicles entering the queue from the backside is below the number of vehicle leaving the queue at the front side. After the demand on the roadway drops below this threshold the queue begins to dissipate.
As a result of this queuing behavior, a graph similar to Figure 3-14 is expected. When running PARAMICS using its default values, the results looked like Figure 13-14. Notice the sharp peaks in Figure 13-14 in the congested area. The peaks are due to vehicles leaving the front side of the queue. Using PARAMICS’s default values, vehicles leaving the front side of the queue will not begin to accelerate until the vehicle in front of them is 35.41 feet (the default queue distance) away from their front bumper. The space causes vehicles to spread out into a free flow condition, preventing any queues from forming just upstream. Once the queue is clear, the flow and speed increase, causing a peak, until congestion conditions reappear, causing a sudden drop in speed.

Figure 3-14: The expected shape of a speed curve when congestion occurs
A strong backward shockwave at the head of the queue is expected when a crash is cleared, or some other type of bottleneck is removed from a system, but when the bottleneck does not move and persists over the entire simulation there is no reason to expect this type of shockwave. To correct the simulation, by removing the shockwave the driver characteristics are changed and their effect determined.

Twenty-five runs of PARAMICS are inspected with different values of core parameters: target headway, mean reaction time, queuing speed, and queuing distance. Behavior characteristics aggressiveness and awareness are also tested, but no effect is seen from changing their values. The values for each run tested are shown in Table 3-2. Each runs effect on vehicle behavior is qualitatively compared in the modeler to determine a ballpark values for each parameter.
Table 3-2: Validation values inspected

<table>
<thead>
<tr>
<th>Curve Speed Factor</th>
<th>Mean Headway</th>
<th>Mean Reaction Time</th>
<th>Queue Speed</th>
<th>Queue Distance</th>
<th>Weight Heavy</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.5</td>
<td>0.5</td>
<td>4.470 mph</td>
<td>32.810 ft</td>
<td>2.950 ton</td>
</tr>
<tr>
<td>0</td>
<td>0.5</td>
<td>0.5</td>
<td>4.470 mph</td>
<td>32.810 ft</td>
<td>2.950 ton</td>
</tr>
<tr>
<td>100</td>
<td>0.5</td>
<td>0.5</td>
<td>4.470 mph</td>
<td>32.810 ft</td>
<td>2.950 ton</td>
</tr>
<tr>
<td>100</td>
<td>0.5</td>
<td>0.5</td>
<td>4.470 mph</td>
<td>6.000 ft</td>
<td>2.950 ton</td>
</tr>
<tr>
<td>100</td>
<td>0.5</td>
<td>0.5</td>
<td>0.100 mph</td>
<td>1.000 ft</td>
<td>2.950 ton</td>
</tr>
<tr>
<td>100</td>
<td>1.65</td>
<td>0.5</td>
<td>0.100 mph</td>
<td>1.000 ft</td>
<td>2.950 ton</td>
</tr>
<tr>
<td>100</td>
<td>1.65</td>
<td>0.4</td>
<td>0.100 mph</td>
<td>1.000 ft</td>
<td>2.950 ton</td>
</tr>
<tr>
<td>100</td>
<td>1.65</td>
<td>0.42</td>
<td>0.100 mph</td>
<td>0.100 ft</td>
<td>1.000 ton</td>
</tr>
<tr>
<td>100</td>
<td>1.65</td>
<td>0.5</td>
<td>0.100 mph</td>
<td>1.000 ft</td>
<td>2.950 ton</td>
</tr>
<tr>
<td>100</td>
<td>1.65</td>
<td>0.5</td>
<td>1.000 mph</td>
<td>1.000 ft</td>
<td>2.950 ton</td>
</tr>
<tr>
<td>100</td>
<td>1</td>
<td>0.25</td>
<td>1.000 mph</td>
<td>1.000 ft</td>
<td>2.000 ton</td>
</tr>
<tr>
<td>100</td>
<td>1</td>
<td>0.42</td>
<td>4.000 mph</td>
<td>1.000 ft</td>
<td>2.000 ton</td>
</tr>
</tbody>
</table>

Once an idea about what the values is determined 6 cases with different values are run and their speed versus time graphs for the 36 detector locations are compared qualitatively. Table 3-3 shows a summary of the 6 cases tested. The result of the inspection of the graphs is that Case 1 and Case 2 show the best results. To determine which of these two cases will be used a final test comparing congestion location is run.

Table 3-3: Six cases inspected with their factors

<table>
<thead>
<tr>
<th>Case</th>
<th>Headway</th>
<th>Reaction Time</th>
<th>Queuing Speed</th>
<th>Queuing Distance</th>
<th>Evaluation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.0 s</td>
<td>0.42 s</td>
<td>8 mph</td>
<td>9 ft</td>
<td>Accurate regions for West, less for East</td>
</tr>
<tr>
<td>2</td>
<td>1.0 s</td>
<td>0.42 s</td>
<td>8 mph</td>
<td>10 ft</td>
<td>Accurate regions for West, less for East</td>
</tr>
<tr>
<td>3</td>
<td>1.0 s</td>
<td>0.42 s</td>
<td>8 mph</td>
<td>7 ft</td>
<td>Too much middle speeds (Lack of Congested uncongested regions)</td>
</tr>
<tr>
<td>4</td>
<td>1.2 s</td>
<td>0.42 s</td>
<td>8 mph</td>
<td>6 ft</td>
<td>Too much middle speeds (Lack of Congested uncongested regions), low speeds not low enough</td>
</tr>
<tr>
<td>5</td>
<td>1.2 s</td>
<td>0.42 s</td>
<td>10 mph</td>
<td>8 ft</td>
<td>Too much volatility in Speed</td>
</tr>
<tr>
<td>6</td>
<td>1.2 s</td>
<td>0.42 s</td>
<td>12 mph</td>
<td>10 ft</td>
<td>Too much middle speeds (Lack of Congested uncongested regions)</td>
</tr>
</tbody>
</table>

Conclusion:
Cases 1 and 2 need to be inspected further to determine which should be used.
By inspecting each of the detectors 5 minute average speeds over the three hour simulation and determining if the detector is experiencing no congestion, mild congestion, or heavy congestion and comparing the results to where congestion if expected the best core values is determined. Speeds of under 30 mph, but over 20 mph are considered to indicate mild congestion, speeds under 20 mph are considered heavy congestion and speeds over 30 mph are considered no congestion. Table 3-4 shows the qualitative analysis of the two scenarios. Case 1 slightly overestimates congestion, but is more realistic than Case 2, which underestimates congestion. Congestion locations are shown in Figure 3-16. The location are determined by listening to morning radio reports.
Figure 3-16: Morning congestion locations
### Table 3-4: Case 1 and Case 2 inspection

<table>
<thead>
<tr>
<th>Station</th>
<th>1</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>EB</td>
<td>WB</td>
</tr>
<tr>
<td>28</td>
<td></td>
<td></td>
</tr>
<tr>
<td>29</td>
<td></td>
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<td>31</td>
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<td></td>
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<tr>
<td>48</td>
<td>Congested</td>
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<td>67</td>
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<tr>
<td>68</td>
<td></td>
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</tbody>
</table>
3.8.5 Concluding: Which Variables are Appropriate

While the exact effect of any of the variable is not known for certain, as the author did not find any documentation of the effect in the PARAMICS User Guides, to evaluate which of the variables is plausible it is assumed that the author definitions given in the preceding paragraphs are correct. Disregarding the 24mph speeds, the lowest error rate is produced by a mean headway of .5 seconds, a mean reaction time of .5 seconds, and a queuing distance of 32.81ft. However, these values do not provide the appropriate queuing behavior. Because the end use of the simulation will be to evaluate intervehicle behavior, the values that produce the most appropriate behavior are used in the network. Those values are a mean headway of 1.0 second, a mean reaction time of .42 seconds, a queuing speed of 8 mph and a queuing distance of 9 ft. The values for mean reaction time and mean headway most closely resemble Lee’s et al. values (1999) of .42 and .62. The mean headway is shorter for Lee et al. (1999) because they used the default queue distance and queue speed. As pointed out in section 3.8.2 the queue distance and the mean headway appear to interact when there the headway is small enough. A headway of 1.0 is the default offered by PARAMICS and is supported by Gardes’ et al. work (2002). The literature did not comment on queue speed and queue distance values, as a result, it is assumed that they used the default values.

3.9 Implementing the Crash Prediction Algorithm

PARAMICS allows a user to write code in the incidence file to induce a crash. While this method could have been adopted in order to detect crashes the runs would have had to be long enough for crashes to take place. Instead the occupancy, speed, and flow data
collected are exported into an Excel spreadsheet, which is processed using the crash
detection algorithm developed by Abdel-Aty et al. (2005). After the analysis the
probability of a crash is determined.

3.10 Changing Speed Limits

In a similar attempt, Lee et al. (2004) chose VMS to change speed limits. The API
program for PARAMICS allowed them to write code for changing the VMS display
speed in real-simulated-time. The method has strong merit, but the PARAMICS modeler
user guide describes the criteria that effect drivers’ adherence to VMS as being different
that the criterion affects their adherence to speed limits.

In an attempt to keep the reaction of drivers’ to VSL the same as to a static speed
limit, a different tactic for changing speed limits is employed. PARAMICS was not
designed with VSL in mind, but it was designed to look at bus routes and other forms of
mass transit. As a result link properties are allowed to change over time. This feature
can be used to change speed limits on roadways by changing link characteristics.

The advantage of changing the link characteristics is that drivers will respond to the
new link speeds in the same way they responded to the old link speeds, keeping a
uniform adherence to speed. The disadvantage is that PARAMICS allows the changes to
be made only at 5 minute intervals, meaning a speed limit could not be changed 18
minutes into the simulation and then back 22 minutes into the simulation. Instead, both
of the times have to be changed so that their values are multiples of 5 minutes.
3.10.1 Adjusted Detector Length

In modeling loop detectors in PARAMICS double loop detectors must be shown as two separate detectors. Each has its own function and speed is not calculated more accurately than using a single loop detector. Twice the output is generated, both slowing down the runtime and further complicating the data processing work after the runs are completed. To avoid these pitfalls the double loop detectors on I-4 are modeled using a single loop detector. The only problem with this approach is trying to figure out what size loops are appropriate for the job. The network is run with three possible different loop sizes in an attempt to determine the loop size that does the best job of simulating the double loop detectors. Occupancy information is selected to compare the three situations. The three sizes chosen to compare are six ft, 10 ft, and 22 ft. Six feet corresponds to an individual loop detector size. Thirty-two feet corresponds to the total length of the double loops and the distance between the loops (6 ft + 6 ft + 10 ft = 22 ft). Ten feet represents a value somewhere between the six feet and 22 ft and happens to be the same value as the distance between the two loops. The analysis was inconclusive. Since detectors use only one six foot loop in order to determine occupancy a six foot loop length is used.

3.10.2 Adjusted Link Length

Following the example set by previous work the VSL values are changed at regular intervals upstream of the loop detector that indicates a high probability of a crash. In order to allow this change to take place links within the network must end at the regular intervals. In order to accommodate this, node locations are added just upstream of loop
detectors. The detector file is then checked to ensure that the detectors are not too close to the nodes. If the nodes are too close double counting could occur. In three cases the loops were too close and the detector file was changed in order to move the detector away for the node. In each case the distance moved will not affect the performance to the detector.

3.10.3 How accurate is the network

As mentioned before the error rate for the ramps in PARAMICS is approximately 3.9%. This error represents discrepancies with the FDOT’s 2003 AADTs peak hour counts. The error is partly because peak numbers are used in all cases. If one ramp peaks in the morning and another in the afternoon then the simulation is comparing two different times. In any case, the error does not represent the overall error of the corridor, because the system was calibrated using ramp numbers not mainline counts. The error is actually higher. After validation the percent error of the mainline is found to be 14.19%. This error represents the overall accuracy of the network.

3.10.4 Network Accuracy

An error rate of over fourteen percent is high, especially for a corridor. The error could be lowered by adjusting the OD, but this would effectively be using the mainline counts as calibration tools. Validation could not be accomplished, because there would not be any sources of information that had not already been used to calibrate the corridor. Furthermore, the interest in the corridor is with the occupancy, flows, and speeds found at the loop detectors within the thirty-second intervals, not with the AADT produced. In order to look at the thirty second intervals it is important that the reaction of the drivers in
the simulation is similar to that of drivers in the real world situation and that the overall behavior of the flow should be similar to what is seen in the real world. The verification step insures that real world behaviors are seen, and the way PARAMICS is programmed the drivers react reasonable to each others presence. Particular emphasis is placed on modeling congested conditions. Queue speed and queue distance, which have not been dealt with in the literature, are adjust fit these behaviors. Because vehicles has been taken to accurately reflect intervehicle behaviors the 14% error is assumed to be low enough to get an accurate model.

3.11 Problems with PARAMICS

While a literature review revealed that PARAMICS is a superior software in many ways, it is not without faults. In the process of working with PARAMICS some of its problems have surfaced. Most are minor, but are worth noting so that future users of the program, as well as those reading this thesis, may better understand the role the software played and the possible limitations caused by the software.

3.11.1 Merging Behavior on Onramps

In PARAMICS vehicles do not merge before the end of acceleration lanes. As a result, long acceleration lanes provide little additional benefit over short acceleration lanes. Congestion occurs even with long acceleration lanes. The lengthening of the lanes only provides additional storage space for vehicles, but does not relieve congestion.
3.11.2 Overestimation of Jams

PARAMICS tends to overestimate traffic jams in networks. As an example, in another network that the author developed certain seed values would cause a vehicle to pause just north of the 528 on I-4, reducing its speed to 30mph in a 65mph zone. The result was cataclysmic. A three mile backup then follows within 15 minutes of simulated time. In the next 45 minutes of simulated time, the full length of the simulation, I-4 does not recover a quarter of a mile of uncongested highway. With other seeds, no backup occurs because no vehicle pauses. While I-4 routinely hinges on congestions one, vehicle causing so much congestion is an overestimation.

3.11.3 Overreaction of Drivers

Part of the problem with the congestion is due to overreaction by drivers. When the vehicle mentioned before paused, the vehicle behind it slow down immediately as did the vehicle behind it and so on, quickly propagating a shockwave. The problem here is how quickly the shockwave propagates. Without measuring the wave, just by viewing the simulation, it is obvious it is moving far too quickly. Drivers in the adjacent lanes react by slowing down, looking as though they are rubber necking, causing a backup in their lanes. Drivers with low aggressiveness show themselves by slowing down more and creating more backup. Soon all lanes on I-4 are backed up.

3.11.4 Difficult to Import Overlays

PARAMICS has a grid that is unlimited in size and can be used to scale of distances, but attempting to use it alone to draw by requires knowing coordinates of nodes, which
can be difficult to determine. To overcome the conflict, overlays are imported to allow for drawing references. PARAMICS allows for bmp or dxf files to be imported, which seems to include all AutoCAD files, but experience shows differently. AutoCAD files must be saved as AutoCAD R12/LT2 dxf version. Even then only information on the zero layer is imported, xref files are not imported. Small links are often lost in the process. The best solution that the author has found is to try multiple imports, with data missing on the first import comprising the second file. This multiple overlays then have to be matched up, which can be time consuming. After using the overlays they are removed from the network to improve save and refresh times.

Figure 3-17: Grids at different spacing values over the I-4 network

3.11.5 Inaccurate Examples of Code

The modeler user guide and modeler reference guides are valuable resources, yet they are not without their problems. The format of files reference by PARAMICS differs slightly but significantly from the documented code. Only through multiple try and error
attempts can the correct code be determined. As a result, Appendix B of this thesis contains some example files with correct coding.

3.11.6 Difficulty Collecting Specific Loop Information

As explained in the measurements section of this thesis, the information collected by loops is either instantaneous data that needs refining or average data that is taken from a point other than the loop location, making it impossible to coordinate the two sets of data. Lee et al. (2004) overcomes some of these shortcomings by accessing loop data that PARAMICS displays in the simulation using the API interface. The data is a better grouping of the flow and speed measurements, but is not offered in the measurements toolbar. If the data is already being processed by PARAMICS, it seems reasonable that this data is available to be exported as a measurement. The way the author has post-processed information using a macro requires an addition hour of computer time for each simulation run.

3.11.7 Five Minute Interval Required On All Link Changes

PARAMICS is an exceedingly flexible software with measurements that can be taken every minute, OD matrixes that can be changed every five minutes, and network that can be, theoretically, infinitely big. However, more flexibility is desirable. For the purpose of the authors work, it would be desirable to change link speeds down to the thirty-second interval. Others working with the software might desire the ability to change OD down to every thirty seconds. Currently PARAMICS has a number of features that are
updateable only in 5 minute intervals. These features are the ones that reference the profile file.

3.11.8 Unable to Specify Distributions

PARAMICS uses distributions in order to create randomness. One distribution is used to create randomness in the release rate of vehicles from origins. Data collection could possible yield information on the type of distribution that would be desirable from a specific origin, but PARAMICS does not allow a user to specify distribution types, in its modeler program. This applies to all distributions, not just the release rate.

3.11.9 Too Many U-Turns, Lost Drivers

When modeling I-4 as part of a network and not as its own corridor, many drivers get “lost” or “miss an exit.” The behavior is meant to model a behavior that happens everyday. The problem is that certain exits are missed frequently, the I-4 and East-West Expressway interchange while heading west on I-4, and cause back up on the next exit as drivers attempt to U-turn and work their way back to the exit. Changing the awareness level of these drivers can improve the situation, but having to specify a separate OD for one exit is frustrating and time consuming.

3.11.10 No Approaches; Too Much Lane Changing Behavior at Intersections

PARAMICS does not have a function that allows for approach to intersections or toll booths to be simulated. Instead it is up to the modeler to add separate links with the appropriate number of lanes. Unfortunately, more lanes, or possibly the presence of an
intersection, induce lane changes by drivers as they approach intersections. As a result backups can occur, though they are relatively infrequent. It makes for poor presentation quality. When displaying the network to clients or co-workers the behaviors sticks out and may reduce their confidence in the model.

3.11.11 No Left Ramps

As mentioned before left ramps are not modeled in PARAMICS. PARAMICS Support suggests using a wide entrance and wide exit option on the preceding link to approximate a ramp, but the authors attempt to use these options produced no recognizable changes to vehicle behavior.
CHAPTER FOUR: EXPERIMENTAL DESIGN

In order to determine how to best reduce the probability of a crash, the variables that affect traffic flow are established and possible values of each variable are considered. The variables that are shown in the literature to affect VSL are where the signs are located, what speed limits are displayed on the signs, when the speed limits are displayed, and where among the sign locations the speed limits is displayed. To make the problem easier to simulate a few assumptions are made about the variables. To see a list of the assumptions see Table 4-1. Due to the assumptions, the total number of variables is reduced to three: temporal variation (when the speed limit changes are displayed), spatial variation (where the speed limits are displayed), and speed limit variation (what is magnitude of speed limit is being displayed).

<table>
<thead>
<tr>
<th>Assumptions</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. A suitable solution maybe found using only VSL.</td>
</tr>
<tr>
<td>2. The VSL attributes of PARAMICS are correct.</td>
</tr>
<tr>
<td>3. The low speed and high speed models are accurate (Abdel-Aty et al. 2005).</td>
</tr>
<tr>
<td>4. VSL are regulatory and will be followed by drivers in the same manner that speed limits are followed.</td>
</tr>
<tr>
<td>5. VSL values will be in intervals of 5mph.</td>
</tr>
<tr>
<td>6. VSL values are determined in five minute intervals.</td>
</tr>
<tr>
<td>7. VSL are distributed along I-4 every half a mile at the loop detector locations.</td>
</tr>
</tbody>
</table>

4.1 Mathematical Model

The mathematical model used (Abdel-Aty et al., 2005) is divided into two regimes: a high-speed and a low-speed. The cut off point for the two models occurs at 37.5 mph.
Above this speed, a high-speed model that takes into account occupancy and flow data are used. Below this speed, a low-speed model that takes into account volume, occupancy, and speed variation data are used. The high-speed model is shown in Equation 4-1. The equation variables are made up of three parts: a function, a location, and a time. The function tells what operation to perform and the location and time are taken relative to the location and time of the location being tested for crash potential.

**FUNCTIONS**

- **LogAO**: Log of the average occupancy
- **SV**: Standard Deviation of Volume
- **AV**: Average Volume
- **LogCVS**: Log of (the Standard Deviation of Speed Divided by the Average Speed)

**LOCATIONS**

<table>
<thead>
<tr>
<th></th>
<th>B</th>
<th>C</th>
<th>D</th>
<th>E</th>
<th>F</th>
<th>G</th>
<th>H</th>
<th>I</th>
<th>J</th>
</tr>
</thead>
</table>

**TIMES**

1: Zero to five minutes before time being tested
2: Five to ten minutes before time being tested
3: Ten to fifteen minutes before time being tested

Figure 4-1 Crash Model Definitions

The relative measures, times and location, are difficult to grasp. Station F is the station being tested. For instance imagine testing detector 33 in the westbound direction.
D is the detector two stations upstream from detector 33, or detector 35. Had the direction been eastbound D would stand for detector 31. Time works similarly. If the time of interest is 6:15 then time 1 corresponds to the time 6:10. Because the model is predictive all times are before the crash. By combing the three effects, in order, the variable of interest is determined.

\[
Risk = -0.93423 \log\text{AOF2} + 1.14584 \log\text{AOH3} - 0.22878 \text{SVH2} - 0.10055 \text{AVG2} + 05932 \text{AVE3}
\]  

(4-1)

For the low-speed model the same notation is used. However an earlier model than that specified by Pande et al. (2005) was used to first analyze the data. The reason for the change is that the low speed models were carried out before the publication of the more recent model. The model is not specific for the low speed case it incorporates the variables that show up only in the low speed case in the more recent paper. The generic model from Abdel-Aty et al. (2004) is shown in Equation 4-2. For consistency all the low speed model results that are shown in this thesis have been recalculated using the low speed model by Abdel-Aty et al. (2005), shown in Equation 4-3.

\[
Risk = 1.21405 \log\text{CVSF2} + 0.02466 \text{AOG2} - 0.19124 \text{SVG2}
\]  

(4-2)

\[
Risk = 2.64827 \log\text{CVSF2} + 0.88842 \log\text{CVSF3} + 1.33966 \log\text{AOE2} + 0.97766 \log\text{AOH3} - 0.43603 \text{SVF2}
\]  

(4-3)
The left side of Equations 4-1, 4-2, and 4-3 are the same measure, risk. Risk, for this thesis, is the log of the odd of a crash, ignoring the population adjustment factor. Odd of a crash is the probability of a crash divided by the probability of a non-crash. Log of the odds is assumed by Abdel-Aty et al. (2004 and 2005) to be a linear regression on X where X may be a vector. Abdel-Aty (2004 and 2005), however, does not solve for the log of the odds. Instead the log of the odds ratio is used. By making the log of the odds a ratio the interception coefficient (the population adjustment factor) is removed. In this way Abdel-Aty does not need to solve for the population parameter. Because this population parameter is unknown the individual value of risk has no meaning, but the difference between risks represents the log of the odds ratio. A positive difference means the likelihood of a crash is increasing, while negative difference means the likelihood of a crash is decreasing. The derivation of this result may be seen below in Figure 4-2.

The central point is that an individual station at an instant in time cannot be characterized, but comparisons being two or more scenarios, two or more time periods, or two or more stations can be accomplished.

\[
\text{odds} - \text{ratio} = \frac{P_c}{P_{nc}} = e^{(\beta_0 + \beta_1 X_1 + \beta_2 X_2 + \beta_3 X_3)}
\]

\[
\text{relative odds} - \text{ratio} = \frac{P'_c}{P'_{nc}} = \frac{e^{(\beta'_0 + \beta'_1 X'_1 + \beta'_2 X'_2 + \beta'_3 X'_3)}}{e^{(\beta_0 + \beta_1 X_1 + \beta_2 X_2 + \beta_3 X_3)}} = e^{(\beta'_0 - \beta_0)(X'_1 - X_1) + (\beta'_1 - \beta_1)X'_2 + (\beta'_2 - \beta_2)X'_3 + (\beta'_3 - \beta_3)X'_3}
\]

\[
\text{log} \_ \text{relative odds} - \text{ratio} = \beta'_1 X'_1 + \beta'_2 X'_2 + \beta'_3 X'_3 - (\beta_1 X_1 + \beta_2 X_2 + \beta_3 X_3)
\]

\[
\log \_ \text{relative odds} - \text{ratio} = \text{risk}' - \text{risk}''
\]

Figure 4-2: Relative log odds-ratio derivation
4.2 Network Loading

With the models being dependent on the speed of the vehicles traveling the corridor and the speed of the vehicles being dependent on the loading condition it is crucial that the loading be linked to the model type. Interstate-4 normally operates at an F Level of Service. It is common for average speeds to be below 30 mph for certain stations throughout the rush hour. With the way the OD was specified, using ramp peak hour counts, the loading errors are on the high side, even though the mainline counts are below the mainline peaks for the network. The lower mainline counts can be attributed to congestion lowering the flow. Because the counts error on the high side the peak hour scenario is assumed to have a loading of 90% of the OD matrix. This loading causes the low-speed crash risk model to be applicable since the average speed is below 37.5 mph. The assumption that the low-speed model holds is tested in Chapter 5. The 90% case is henceforth called the low speed case and will have its crash potential evaluated with the low-speed algorithm (Equation 4-3).

The off-peak loading is approximated by a 60% loading of the OD matrix. The loading should cause speeds to be above the 37.5 mph mark. Therefore the 60% loading causes the high speed model to be applicable. The assumption that the high-speed model holds is tested in the Results Chapter. The 60% case is henceforth called the high-speed case and will have its crash potential evaluated with the high-speed algorithm (Equation 4-1).
4.3 Definitions

In order to make it easier to understand the rest of the Experimental Design Chapter and the results in Chapter 5 a few definitions are laid out using Figure 4-3.

The first terms to consider are upstream and downstream. Downstream refers to detectors that a vehicle has yet to pass when at the detector of interest. Upstream refers to detectors that a vehicle has already passed when it arrives at the detector of interest. Notice how distances are measured from the detector of interest to the other detectors. Distances can be listed as either number of miles or number of detectors plus the direction. For example 1 mile upstream of detector 47 or 2 detectors upstream of detector 47 would be specifying the same location.

Notice that the default speed limit is 55 mph at all locations. This is true for the majority of the length of Interstate-4 except for the Fairbanks curve shown in Figure 4-4. The Fairbanks curve is a curve on I-4 just west of the Fairbanks exit where a restrictive geometry brings the speed limit down to 50 mph, although the geometry is a 45 mph design.
When looking at the scenarios that are to be compared a few terms, such as the downstream raising, the upstream lowering, and the gap, will be used. Figure 4-5 shows these terms as they will be used. Upstream lowering refers to the section of roadway upstream of the detector of interest where the speed limit will be lowered in order to keep vehicles from rushing onto slower moving traffic. Notice the distance for the upstream lowering in the diagram is 1.5 miles. Downstream raising refers to the section of roadway downstream of the detector of interest where the speed limit will be raised in order to speed up high density traffic thereby raising the capacity of the roadway. Notice the distance for the downstream raising in the diagram is 1.5 miles. Gap refers to the distance between the detector downstream of the detector of interest and the detector.
where the downstream raising begins. The half of a mile after the detector of interest is maintained at 55 mph in all scenarios therefore its distance is not considered. Gap is suppose allow the raised speed limits to effect the front of any queue or high density traffic instead of bumping up the speed limit where traffic is already contained by the traffic downstream of their location.

<table>
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<th>2.0 mi</th>
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<td>55 mph</td>
<td>55 mph</td>
<td>55 mph</td>
<td>40 mph</td>
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</tr>
</tbody>
</table>

Figure 4-5: Sample scenario for a westbound roadway with detector 47 as the detector of interest.

### 4.4 Variables in the Scenarios

The variables that are used in the different scenarios are divided into three groups: temporal, spatial, and speed limit variation. The three groups look into possible values for each type of variation.

#### 4.4.1 Temporal Variation

The temporal variation, or the rate that the speed limit is changed, is confined to 5 mph interval changes in speed limit every five-minute intervals because the algorithm makes use of the 30-second data in five-minute intervals and PARAMICS is only capable
of handling changes every five minutes. The changing speed limits before and after a potential crash will be experimented with to determine how to best prevent a crash and then reintegrate the speed limits without inducing a crash. After changing the speed limits, their values will need to return back to their default values.

4.4.2 Spatial Variation

Although the spatial variation is greatly reduced by confining the location of the VSL signs, the assumption is needed to limit the number of cases. Half mile spacing serves to line up the signs with the detectors, so that the VSL line up with the loop detectors. Because PARAMICS has a factor known as sign distance (the distance before a link that a driver is aware of the links characteristics, including speed limit) drivers react before the detector and allow that behavior to be captured in the measurements. Certainly, the literature has shown that a half-mile distribution is dense, but the solution is not meant to be cost effective. Assuming that this simulation produces positive results, future research could experiment with less dense distributions of signs.

As with temporal variation, the spatial variation is comprised of two parts, preventing a crash and reintegrating the speed limit. In this case, reintegrating the speed limit consists of determining how far after the potential crash location does the roadway return to its normal speed limit. According to Abdel-Aty et al. (2005), crashes are caused by vehicles at higher speeds (low occupancy) approaching lower speed vehicles (higher occupancy). In order to change this situation the higher speed vehicles can be given a lower speed limit, the low speed vehicles can be given a higher speed limit, or both. The allocation of these speed limits over the VSL is their spatial variation.
4.4.3 Speed Limit Variation

The vast majority of I-4, in the study area, operates with static speed limits of 55 mph. Two scenarios with greater reductions, reducing by 10 mph and by 15 mph, are tested. When raising speed limits, two scenarios are chosen: increasing the speed limit by 10 mph and increasing the speed limit by 15 mph. A 15 mph increase would result in a 70 mph speed limit in all areas except the Fairbanks curve where the speed limit would be 65 mph. As a result of safety and design considerations, any speeds higher than this cannot be supported. The speed limit change is also capped at 15 mph (70 mph top speed limit) due to current safety standards. On the low side a decrease of 15 mph results in a speed limit of 40 mph. While this speed is low it will be considered in the model, but will be the lower bound. PARAMICS will not allow for the safe speed of a curve to be exceeded, therefore any geometric constraints will be override the specified speed limits in the links file. For example, when a speed of 65 mph is specified for the Fairbanks curve that speed limit will be dropped to 50 mph as soon as the network is opened in the program.

4.5 Scenarios for the Low Speed Model

To test the effect of VSL on the low speed model three cases are considered for temporal variation: five minutes before a crash, 10 minutes before a crash, and 15 minutes before a potential crash. For reintegrating the speed limits three time periods are considered at the potential crash time, five minutes after the potential crash, and 10 minutes after the potential crash. For spatial variation, distances of half a mile upstream,
one mile upstream, and one and a half miles upstream are chosen. For reintegrating, the speed limit distances of zero miles, half a mile, and one mile are chosen. Spatial variation is limited to lowering the speed limit upstream, because of the extensive queuing downstream. It is assumed that raising the speed limit downstream would not affect flow because the vehicle will already be traveling below the speed limit. Both of the possible speed limits changes, 10 mph and 15 mph, are considered. Instead of testing all the possible combinations eight sample scenarios are picked to represent different effects. See Figures 4-6 and 4-7 for a list of the eight scenarios.

To explain the notation used in Figures 4-6 and 4-7 Case 4 is described. Looking at time t, the time where the scenario is based, the gradual implementation of speed limit changes across space can be seen. From 2 miles upstream to 1.5 miles upstream the speed is reduced by 5 mph. If the base speed were 55 mph then the speed would be 50 mph here. From 1.5 miles to 1.0 mile upstream the speed would be 45 mph, from 1.0 miles upstream to the half a mile downstream of the detector of interest the speed would be 40, and from half a mile downstream to 1.0 mile downstream the speed limit would be 50 mph. As for the implementation across time

Be varying time, and holding the location constant, the gradual implementation of speed limit changes across time can be seen. From 15 minutes before to 10 minutes before the time of interest the speed limit is reduced by 5 mph at the location of interest. Following a similar logic the other values across time can be determined. The cases in Appendix A are expressed in the same manner.
### Figure 4-6: Cases 1 through 4 for the low-speed model

<table>
<thead>
<tr>
<th>Case 1</th>
<th>t-15min</th>
<th>t-10min</th>
<th>t-5min</th>
<th>t</th>
<th>t+5min</th>
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<td>x+.5mi</td>
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<table>
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<td>0</td>
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Figure 4-7: Cases 5 through 8 for the low-speed model

By comparing the cases an idea about general trends in the use of VSL can be determined. For example comparing the effect of Cases 1-4 to Cases 5-8 the effect of changing speed limits gradually over time and space versus changing abruptly over time and space can be determined. Two other comparisons can be made using the eight cases.
One, the effect of a 10 mph change in speed limit (Cases 1, 2, 5, and 6) versus a 15 mph change in speed limit (Cases 3, 4, 7, and 8). Two, the effect of a longer upstream lowering (Cases 1, 3, 5, and 7) versus a shorter upstream lowering (Cases 2, 4, 6, and 8) on crash potential. The eight cases represent a full factorial design, so the interaction of the effects may also be determined.

4.6 Scenarios for the High Speed Model

In order to measure the interactions as well as the main effects a Full or Partial Fractional Design of Experiment is undertaken. To explore the effects of VSL on crash likelihood the following seven factors would need to be considered, leading to 128 test cases.

- Implementation in Space
- Maximum Speed Change
- Upstream Distance
- Downstream Distance
- Implementation Step Period
- Implementation Step Size
- Gap Distance

The result would be a response surface that would allow the best combination of factors to be determined as evaluated by a given metric. The problem is that metric that can be used to evaluate whether one case is better of worse than another cannot be determined. At first it would seem that average crash likelihood would provide the best measure, however, a short spike in crash potential would be insignificant in such a measure, but would be critical in determining the best case. Maximum crash likelihood
across a given time period might then be the best measure, but if the two values are close then the one that shows the greatest reduction in crash potential overall would be the most important factor, but that is captured by average crash potential.

Instead of attempt to construct a measure of effectiveness that is not representative of the desired final result, the different factors are broken down into groups that are assumed to act independently. As a result, the groups are tested on after another with the best case being used as the starting case of the next group of experiments. In the interest of expediting the process only one run of each scenario is run to hint at which case is best. A statistical test follows after a best case is recommended by the non-statistical tests.

In order to start testing the effectiveness of the high speed model a base case with a problem area must first be identified. The base case for this model is I-4 from Lake Mary to Orange Blossom Trail with a 60% of full loading situation. A three-hour simulation with a 15-minute warm-up period is being used. The westbound traffic at detector 47 is showing a high crash potential and is the detector whose crash potential should be improved. In order to see the effect of crash potential changing in time, thirty minutes into the simulation speed limits will start to change and will be maintained for only 30 minutes. After that time the speed limits will come back to their base value. In this way if the VSL cause a negative effect after their implementation then it will be observed in the model.

In order to test the various scenarios for the high-speed model four separate tests are conducted with the best scenario from the previous scenario being used as the input for the next test. To see the layout of the test see Figure 4-8. The tests that are referenced in the figure are shown in Tables 4-2, 4-3, 4-4, and 4-5, respectively.
The first variables that are considered are the spatial variation of various speed limits (whether upstream lowering the speed limits and downstream raising speed limits is...
better than just downstream raising speed limits, speed limit variation (whether a 10 mph maximum change is better than a 15 mph speed limit change), and the combination of the two variables (whether gradually changing the speed limit over space or abruptly changing the speed limit over space is better). A summary chart of the different scenarios is shown in Table 4-2.

Table 4-2: Different scenarios based on changing speed limit patterns

<table>
<thead>
<tr>
<th>Distance of Change</th>
<th>Amount of Spd. Change</th>
<th>Where Changed</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>Abrupt</td>
<td>10 mph</td>
</tr>
<tr>
<td>Case 2</td>
<td>Abrupt</td>
<td>15 mph</td>
</tr>
<tr>
<td>Case 3</td>
<td>Gradual</td>
<td>10 mph</td>
</tr>
<tr>
<td>Case 4</td>
<td>Gradual</td>
<td>15 mph</td>
</tr>
<tr>
<td>Case 5</td>
<td>Gradual</td>
<td>10 mph</td>
</tr>
<tr>
<td>Case 6</td>
<td>Gradual</td>
<td>15 mph</td>
</tr>
</tbody>
</table>

The second variable tested is the spatial location of changing speed limit signs. In this test the best case from the first test is used and the location of the change in speed limit is altered to see its effect. If the upstream case is found not to have an effect from the first test then the number of cases will be reduced to three by not have an upstream variable to change. See Table 4-3 for the scenarios.
Table 4-3: Different scenarios based on changing distance for speed limit change

<table>
<thead>
<tr>
<th>Case</th>
<th>Upstream Lowering Distance</th>
<th>Downstream Raising Distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>4 mi</td>
<td>5.5 mi</td>
</tr>
<tr>
<td>Case 2</td>
<td>6 mi</td>
<td>5.5 mi</td>
</tr>
<tr>
<td>Case 3</td>
<td>2 mi</td>
<td>5.5 mi</td>
</tr>
<tr>
<td>Case 4</td>
<td>4 mi</td>
<td>4 mi</td>
</tr>
<tr>
<td>Case 5</td>
<td>6 mi</td>
<td>4 mi</td>
</tr>
<tr>
<td>Case 6</td>
<td>2 mi</td>
<td>4 mi</td>
</tr>
<tr>
<td>Case 7</td>
<td>4 mi</td>
<td>2 mi</td>
</tr>
<tr>
<td>Case 8</td>
<td>6 mi</td>
<td>2 mi</td>
</tr>
<tr>
<td>Case 9</td>
<td>2 mi</td>
<td>2 mi</td>
</tr>
</tbody>
</table>

The third variable test is the temporal implementation of the VSL on the best case scenario from test 2. The first case that is considered is abruptly changing from the standard speed limits to the new speed limits. The other cases involve changing the speed limit stepwise from the normal speed limit to the crash prevention speed limit. The step size varies by the time period between changes and the value of the speed. The speed limits can be stepped either every five minutes or every 10 minutes and the speed can change by 5 mph or 10 mph. As with the second test, if the scenario from the earlier tests makes one or more of the cases irrelevant then the case will be disregarded. See Table 4-4 for the scenarios.

Table 4-4: Different scenarios based on changing speed limits over time

<table>
<thead>
<tr>
<th>Time Between Changes</th>
<th>Amount of Change</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>Abrupt</td>
</tr>
<tr>
<td>Case 2</td>
<td>5 min</td>
</tr>
<tr>
<td>Case 3</td>
<td>10 min</td>
</tr>
<tr>
<td>Case 4</td>
<td>5 min</td>
</tr>
<tr>
<td>Case 5</td>
<td>10 min</td>
</tr>
<tr>
<td>Case 6</td>
<td>5 min</td>
</tr>
</tbody>
</table>
The forth variable test is the gap distance, after the detector of high incidence, before raising the speed limit. In this test it is assumed that in order to affect the high density traffic the front of the high density needs run faster than the back, thereby increase the distance that the vehicles occupy, lowering their density. To test this 4 values of the gap are looked into. See Table 4-5 for the scenarios.

Table 4-5: Different scenarios based on the changing gap after the detector of interest before raising the speed limit.

<table>
<thead>
<tr>
<th>Case</th>
<th>Gap Distance</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>0 mi</td>
</tr>
<tr>
<td>Case 2</td>
<td>1 mi</td>
</tr>
<tr>
<td>Case 3</td>
<td>2 mi</td>
</tr>
<tr>
<td>Case 4</td>
<td>3 mi</td>
</tr>
</tbody>
</table>

To get an idea about what a scenarios might look like Figure 4-9 shows the one mile gap case in a graphical manner different manner.
Figure 4-9: Scenario with a one mile gap and 15 mph abrupt change in speed limit (time held constant)

4.7 Ramp Metering Application

If after the use of VSL on the high speed and low speed models no case shows positive results then ramp metering with VSL implementation is tested. Ramp metering is not to be applied using the API included with PARAMICS’ programmer, but is coded by a more brute force method. Because the effect of ramp metering is to calm traffic peaks the effect can be over exaggerated by reducing the ramp counts at specific locations. The high flow periods probability of occurring will be greatly reduced, causing a ramp-metering-like effect. Because this method overestimates, the effect of ramp metering, the API should be implemented upon a successful result.
4.8 Statistical Significance Testing

Once best scenarios for the high-speed and low-speed models are determined from the statistically significant effect of the implementation is determined. Twenty replications of the base case and the VSL implemented case are run. The average crash potential from 16:20 to 17:15 is then calculated for each of the forty runs. The 95% confidence interval for the two cases at each detector is then determined. Finally, the effect is concluded by determining if the confidence intervals overlap.

4.9 Travel Time Analysis

The last step before a scenario can be successfully recommended is a travel time analysis. Twenty replications of the base case and the VSL implemented case are run. The average travel time from 16:30 to 17:30 is then calculated for each of the forty runs. The 95% confidence interval for the two cases. Finally, the effect is concluded by determining if the confidence intervals overlap.
CHAPTER FIVE: RESULTS

The main goal of this thesis was to find VSL implementations that successfully lower crash risk on I-4 in Orlando, FL. I-4 is the major interstate in Orlando and runs North-South (East and West on the roadway) through the center of the city. It is equipped with dual loop detectors every half of a mile. These detectors output information for the simulated model that are analyzed in the same way that the real-world data is analyzed. A reminder of a few terms that are explained in the Methodology (4.3) are necessary so that this section can be fully understood. All speed and crash risk graphs were processed using instantaneous data that was outputted by PARAMICS through Excel macros that are included in Appendix E.

5.1 Model Applicability

Abdel-Aty et al. (2005) showed two different models that used traffic measurements to predict crashes. Their models output a factor called crash potential which when high, indicates that a crash is likely to occur and when low means that a crash is unlikely to occur. The low-speed model is based on speed below 37.5 mph and the high-speed model is based on speeds above 37.5 mph. To verify that the models are being properly utilized, the speed profiles for the detectors of interest are shown.

5.1.1 Low-Speed Model

Detector 33 in the eastbound direction, with 90% loading, shows that when congestion occurs, the low-speed model is the correct model to use. Keep in mind that
the base 100% case incorporates peak ramp counts, so both directions are peaking at the same time. By using a 90% case the loading is more similar to true peak hour conditions. Congestion sets in at 17:00. To see the outputted speeds, see Figure 5-1. Detector 61 in the westbound direction, with 90% loading, shows a speed below the 37.5 mph threshold through the entire simulation. To see the outputted speeds for detector 61, see Figure 5-2. These speeds validate the application of the low-speed model, Equation 5-1, because their five-minute moving average of speeds are below the 37.5 mph threshold. Note that Detector 61 is east of downtown, while Detector 33 is just west of downtown. The five-minute moving average of speeds is an average of ten consecutive 30-second values of average speed. Every thirty second a new average is computed, effectively sliding the 5-minute average along with time, or moving the average.

\[
Risk = 2.64827 \text{ LogCVSF2} + 0.88842 \text{ LogCVSF3} + 1.33966 \text{ LogAOE2} + 0.97766\text{Log AOH3} - 0.43603\text{SVF} \quad (5-1)
\]
Figure 5-1: Station 33 speed profile throughout the simulation period.

Figure 5-2: Station 61 speed profile throughout the simulation period.
5.1.2 High-Speed Model

Detectors 61 and 47 are used in order to perform the high-speed implementation of variable speed limits. With a 60% loading, both stations show a speed of well over 37.5 mph for the entire simulation period. See Figure 5-3 and Figure 5-4 for the speed profiles.

$$Risk = -0.93423 \log(\text{AOF2}) + 1.14584 \log(\text{AOH3}) - 0.22878 \text{SVH2} - 0.10055 \text{AVG2} + 05932 \text{AVE3} \quad (5-2)$$

Figure 5-3: Station 61 speed profile throughout the simulation period.
5.1.3 Speed Limits Changing

In implementing variable speed limits, in both the high speed and low speed models, a different technique than the one suggested by PARAMICS (Programmer) is used. Instead of implementing the change in speed limits using a C++ program, different links files are used. PARAMICS writes to multiple ascii files instead of one central file when saving. One of these files is the links file. The links file keeps all information about the links on the network, including speed limits. The model’s profile file, another ascii file, is then changed to specify which links files are referenced based on the simulated time. To insure that the speed limits are changing properly, vehicle speed, at a location where the speed limit is supposed to drop, is checked. Figure 5-5 shows the effect of changing the speed limit and proves that a change is taking place. Cases 1 and 2 are two different
versions of the networks where the speed is changed at 16:30 using VSL in the 2nd and third links file, respectively. The two cases are shown to be effective in Figure 5-5.

![Station 62 (WB) with 90% Loading](image)

Figure 5-5: Station 62 speed profile throughout the simulation period.

5.2 Model Application

5.2.1 Low-Speed Model

After showing that the low-speed model is applicable to detectors 33 and 61, when 90% loading is used, the next step is to apply the low-speed algorithm to see what kind of crash potentials are present. Figure 5-6 shows that as congestion begins to take hold of detector 33 (as shown in the speed profile, Figure 5-1) there is no impact on crash
potential. Variable speed limits (VSL) can be applied before and during congestion and their effect on crash potential can be assessed. VSL will be implemented at 16:30 and 17:00 to see the effect of both. Figure 5-7 shows a different scenario, at detector 61, in which congestion is persistent and the crash potential (calculated by Equation 4-3) is relatively high, compared to station 33, throughout the simulation. By applying VSL here, their behavior with an already congested section can be determined.

Figure 5-6: Station 33 crash potential throughout the simulation period.
5.2.2 High-Speed Model

The high-speed model detector 61, with a 60% loading, displays the effects of a low crash potential section, which is a relatively low crash risk. Detector 47, with a 60% loading, displays the effects of a high crash potential area, which is a relatively high crash risk. Detector 61 originally was chosen because of a sharp spike in crash potential at 16:45, but after working with the simulation it is determined that the spike is caused by improperly drawn geometry. Figure 5-8 shows the corrected crash potential diagram for detector 61. Figure 5-9 shows the crash potential calculated for detector 47 based on Equation 4-1.

Figure 5-7: Station 61 crash potential throughout the simulation period.
Figure 5-8: Station 61 crash potential throughout the simulation period.
Figure 5-9: Station 47 crash potential through out the simulation period.
5.3 Low-Speed Model after VSL Implementation

5.3.1 Station 33

VSL are implemented using an upstream lowering technique. One of the scenarios (from Appendix A) is tested on each detector to see if the system is affected. If no effect is seen then the other cases are not attempted. The scenario chosen is shown below in Table 5-1. The scenario shows a gradual implementation in both time and space.

Table 5-1: The test scenario for the low speed model.

<table>
<thead>
<tr>
<th></th>
<th>t-15min</th>
<th>t-10min</th>
<th>t-5min</th>
<th>t</th>
<th>t+5min</th>
</tr>
</thead>
<tbody>
<tr>
<td>x-2mi</td>
<td>0</td>
<td>0</td>
<td>-5</td>
<td>-5</td>
<td>0</td>
</tr>
<tr>
<td>x-1.5mi</td>
<td>0</td>
<td>-5</td>
<td>-10</td>
<td>-10</td>
<td>-5</td>
</tr>
<tr>
<td>x-1mi</td>
<td>-5</td>
<td>-10</td>
<td>-15</td>
<td>-15</td>
<td>-5</td>
</tr>
<tr>
<td>x</td>
<td>-5</td>
<td>-10</td>
<td>-15</td>
<td>-15</td>
<td>-5</td>
</tr>
<tr>
<td>x+.5mi</td>
<td>-5</td>
<td>-5</td>
<td>-5</td>
<td>-5</td>
<td>-5</td>
</tr>
</tbody>
</table>

Detector 33’s crash potential curve shows no significant difference after the implementation of VSL, as shown in Figure 5-10, where Case 1 is the scenario in Table 5-1. The difference between the two curves is attributed to randomness in the simulator.
5.3.2 Station 61

VSL are implemented using an upstream lowering technique. Again only one of the scenarios (from Appendix A) is tested on each detector to see if the system is affected. The same low speed scenario, shown in Table 5-1, is used.

Detector 61’s crash potential curve shows no significant difference after the implementation of VSL, as shown in Figure 5-11. As a result, the other cases are not implemented.
5.4 High-Speed Model after VSL Implementation

5.4.1 Station 61

Station 61 is rerun with the 60% loading using all 8 cases, making it the high-speed model. As shown in Figure 5-12, the application of VSL when using Case 4 shows a strong reduction in crash potential at detector 61, half a mile upstream from the detector where the changes were centered. The other cases do not show a significant difference from the base case, which is not expected. For example, Case 2 and Case 4 are similar
cases, so if Case 4 dramatically improves the situation, Case 2 is at least expected to have an effect. For a look at the case refer to Figures 5-13 and 5-14.

![Graph showing crash risk over time for different cases.]

Figure 5-12: Station 62 crash potentials for the base case and upstream VSL implementation cases.

To understand why Case 4 is the only case to improve safety, a speed plot is taken. Figure 5-15 shows the speed diagram at detector 64. Detector 64, which is 1.5 miles upstream of Detector 61, is used because it has the most dramatic difference in speeds and best illustrates what the problem is. When Case 4 is run, there is a different release rate. The release rate is such that congestion occurs before the detector of interest. The congestion limits the speeds and intervehicle interaction downstream, causing a complete different vehicle behavior. The case has been rerun several times and each time the exact
same result is obtained in Case 4, but is not seen in any other case. The cause of the problem is not determined.

Figure 5-13: Cases 1 through 4 for the low-speed model
### Case 5

<table>
<thead>
<tr>
<th></th>
<th>t-15min</th>
<th>t-10min</th>
<th>t-5min</th>
<th>t</th>
<th>t+5min</th>
</tr>
</thead>
<tbody>
<tr>
<td>x-2mi</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>x-1.5mi</td>
<td>0</td>
<td>-10</td>
<td>-10</td>
<td>-10</td>
<td>-10</td>
</tr>
<tr>
<td>x</td>
<td>0</td>
<td>-10</td>
<td>-10</td>
<td>-10</td>
<td>-10</td>
</tr>
</tbody>
</table>

### Case 6

<table>
<thead>
<tr>
<th></th>
<th>t-15min</th>
<th>t-10min</th>
<th>t-5min</th>
<th>t</th>
<th>t+5min</th>
</tr>
</thead>
<tbody>
<tr>
<td>x-2mi</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>x-1.5mi</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>x-1mi</td>
<td>0</td>
<td>-10</td>
<td>-10</td>
<td>-10</td>
<td>-10</td>
</tr>
<tr>
<td>x</td>
<td>0</td>
<td>-10</td>
<td>-10</td>
<td>-10</td>
<td>-10</td>
</tr>
<tr>
<td>x+.5mi</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

### Case 7

<table>
<thead>
<tr>
<th></th>
<th>t-15min</th>
<th>t-10min</th>
<th>t-5min</th>
<th>t</th>
<th>t+5min</th>
</tr>
</thead>
<tbody>
<tr>
<td>x-2mi</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>x-1.5mi</td>
<td>0</td>
<td>-15</td>
<td>-15</td>
<td>-15</td>
<td>-15</td>
</tr>
<tr>
<td>x-1mi</td>
<td>0</td>
<td>-15</td>
<td>-15</td>
<td>-15</td>
<td>-15</td>
</tr>
<tr>
<td>x+.5mi</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

### Case 8

<table>
<thead>
<tr>
<th></th>
<th>t-15min</th>
<th>t-10min</th>
<th>t-5min</th>
<th>t</th>
<th>t+5min</th>
</tr>
</thead>
<tbody>
<tr>
<td>x-2mi</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>x-1.5mi</td>
<td>0</td>
<td>-5</td>
<td>-5</td>
<td>-5</td>
<td>-5</td>
</tr>
<tr>
<td>x-1mi</td>
<td>0</td>
<td>-15</td>
<td>-15</td>
<td>-15</td>
<td>-15</td>
</tr>
<tr>
<td>x+.5mi</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

---

Figure 5-14: Cases 5 through 8 for the low-speed model

Case 4 therefore must be disregarded. After throwing out the case, none of the other cases produce a result that may say, convincingly, that they improve safety.
Figure 5-15: Station 64 average speeds for the base case and upstream VSL implementation cases.

Figure 5-16 and Figure 5-17 show a different story. By using both upstream and downstream VSL, a trend seems to have developed. While not significantly different, the VSL case looks to be lower in crash potential than the base case. It should be remembered that station 61 did not have a high crash potential to begin with. To see if this trend plays out, a higher crash potential location, station 47, is considered.

Figure 5-16 and Figure 5-17 differ, because Figure 5-16 uses the same speed limit function throughout the simulation, while Figure 5-17 uses speed control in order to lower the speed limit. Speed control is the technique PARAMICS uses in its programmer guide. Comparing these two techniques further validates the method of multiple links files.
Figure 5-16: Station 61 crash potentials for the base case and downstream VSL implementation case.
Figure 5-17: Station 61 crash potentials for the base case and downstream VSL implementation case via speed control.

5.4.2 Station 47

5.4.2.1 Speed Limit Patterns Scenarios

The first set of scenarios tested compare the effect of changing speed limits by different values across one time period. All speed limits change to their variable speed at 4:30pm (16:30) and change back to their base speed at 5:00pm (17:00). The difference between the scenarios is summarized in Table 5-2. The change in crash potential due to these different scenarios is shown in Figure 5-18. In Table 5-2 the variables in the different scenarios are spatial implementation rate, change amount, and location of the changes. Spatial implementation can either be gradual or abrupt. Gradual
implementation involve making the first half a mile either a 5 mph change or a ten mph change based on whether the total change is 10 mph or 15 mph. In other words for a 15 mph change 3.5 miles upstream the speed limit would be lowered by 15 mph, but from 4.0 miles to 3.5 miles the speed limit would only be lowered 10 mph. In the abrupt case all 4 miles would be lowered 15 mph. The amount of the change is limited to a 10 mph or 15 mph change. In either case both upstream and downstream changes will be by the same amount. The location of the change refers to whether the change is going to take place in the 5.5 miles downstream of the detector, or for the same stretch downstream plus an additional 4 miles upstream.

Table 5-2: Different scenarios based on changing speed limit patterns

<table>
<thead>
<tr>
<th>Case</th>
<th>Abrupt</th>
<th>Change</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>Abrupt</td>
<td>10 mph</td>
<td>Up and Downstream</td>
</tr>
<tr>
<td>Case 2</td>
<td>Abrupt</td>
<td>15 mph</td>
<td>Up and Downstream</td>
</tr>
<tr>
<td>Case 3</td>
<td>Gradual</td>
<td>10 mph</td>
<td>Up and Downstream</td>
</tr>
<tr>
<td>Case 4</td>
<td>Gradual</td>
<td>15 mph</td>
<td>Up and Downstream</td>
</tr>
<tr>
<td>Case 5</td>
<td>Gradual</td>
<td>10 mph</td>
<td>Downstream</td>
</tr>
<tr>
<td>Case 6</td>
<td>Gradual</td>
<td>15 mph</td>
<td>Downstream</td>
</tr>
</tbody>
</table>

*All upstream speeds changes are negative and all downstream changes are positive.

Figure 5-18 shows that the abrupt cases outperform the gradual cases and that 15 mph differences outperform 10 mph differences. Also, upstream decreases in speed increase the effect of the variable speed limits, especially at the beginning and ending periods of the change.
5.4.2.1.1 Crash Potential Relocation

Upstream of the location of interest, there are minor changes in crash potential with some stations showing a higher crash potential and some showing a lower crash potential. However, one mile downstream of the location of interest there is a spike that occurs at 16:30 as will be seen later this problem has been resolved. The reason for the increase at this station has not been determined. The speed limit changes here and half a mile upstream and downstream are exactly the same. As a result, the change in crash potential is expected to be the same.

The upstream and downstream edges where the change in speed limits takes place are where crash potentials are expected to relocate. Figures 5-19 and 5-20 clearly show that...
this is not the case. While there is a slight increase at the upstream edge, there is no difference at the downstream edge.

Figure 5-19: Crash potentials at the upstream edge, Station 56, for the 6 scenarios in Table 5-2.
5.4.2.2 Distance Scenarios

While the scenarios from Table 5-2 shows that an abrupt 15 mph change both upstream and downstream have the greatest effect, there are still lingering questions. It is important to ask how large of an area should be changed, how long of a time period will changing the speed limits be effective for, and if the abrupt change in distance should change slowly over time. Also of interest is the effect that changing the speed limits will have over time. In order to look into these questions another set of scenarios is devised.

To look at how much of an area should be changed, the scenarios in Table 5-3 will be run. Case 1 is the Case 2 from the earlier scenario trials. Upstream values both greater than and less than those in the base case are attempted because the upstream edge showed
an increase in crash potential (Figure 5-19). Only downstream values less than those in the base case are attempted because the downstream edge showed no change in crash potential, indicating that the distance was long enough (Figure 5-20).

In Table 5-3 there are only two variables, the upstream length and the downstream length, because gap distance is equal to zero miles. Figure 5-21 below shows what distance these distances translate into when comparing to the real world. Keep in mind that in the figure that detector 47 is the detector of interest.

Figure 5-21: Sample scenario for a westbound roadway.

Table 5-3: Different scenarios based on changing distance

<table>
<thead>
<tr>
<th>Case</th>
<th>Upstream</th>
<th>Downstream</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>4 mi</td>
<td>5.5 mi</td>
</tr>
<tr>
<td>Case 2</td>
<td>6 mi</td>
<td>5.5 mi</td>
</tr>
<tr>
<td>Case 3</td>
<td>2 mi</td>
<td>5.5 mi</td>
</tr>
<tr>
<td>Case 4</td>
<td>4 mi</td>
<td>4 mi</td>
</tr>
<tr>
<td>Case 5</td>
<td>6 mi</td>
<td>4 mi</td>
</tr>
<tr>
<td>Case 6</td>
<td>2 mi</td>
<td>4 mi</td>
</tr>
<tr>
<td>Case 7</td>
<td>4 mi</td>
<td>2 mi</td>
</tr>
<tr>
<td>Case 8</td>
<td>6 mi</td>
<td>2 mi</td>
</tr>
<tr>
<td>Case 9</td>
<td>2 mi</td>
<td>2 mi</td>
</tr>
</tbody>
</table>
5.4.2.2.1 Crash Potential at 47

Figures 5-22 and 5-23 show that all of the cases are effective in reducing the crash potential. No one case outperforms the others in reducing the crash potential at detector 47. Although Case 8 shows an increased crash potential at the onset of the change in speed limits (Figure 5-23). In order to determine which is best, the downstream and upstream effects are inspected.

Figure 5-22: Crash potentials at the detector of interest, Station 47, for the first 5 scenarios in Table 5-3.
Figure 5-23: Crash potentials at the detector of interest, Station 47, for the last 4 scenarios in Table 5-3.

5.4.2.2.2 Upstream Crash Potential

Figures 5-24 thru 5-29 characterize the upstream effects of implementing the VSL. Because the upstream distance is varied, the location of the change in crash potential due to the upstream edge also varies. Cases with the same upstream distance show similar effects on crash potential, which is an encouraging result. Unfortunately, all of the cases worsen the safety of the roadway approximately equally with the implementation of VSL. In order to determine which is best, the effect on travel time is considered. By reducing the speed limit for a shorter period of time, travel time will be increased the least; therefore, the shortest upstream distance is chosen (Cases 3, 6, and 9). To determine the downstream length the downstream effect is investigated.
Figure 5-24: Crash potentials upstream of the detector of interest, Station 60, for the first 5 scenarios in Table 5-3.
Figure 5-25: Crash potentials upstream of the detector of interest, Station 61, for the last 4 scenarios in Table 5-3.
Figure 5-26: Crash potentials upstream of the detector of interest, Station 56, for the first 5 scenarios in Table 5-3.
Figure 5-27: Crash potentials upstream of the detector of interest, Station 57, for the last 4 scenarios in Table 5-3.
Figure 5-28: Crash potentials upstream of the detector of interest, Station 52, for the first 5 scenarios in Table 5-3.
Figure 5-29: Crash potentials upstream of the detector of interest, Station 52, for the last 4 scenarios in Table 5-3.

5.4.2.2.3 Downstream Crash Potential

Figures 5-30 thru 5-35 characterize the downstream effects of implementing the VSL. Because the downstream distance is varied, the location of the change in crash potential varies. Cases with the same downstream distance show similar effects on crash potential. Case 9 shows the best results, particularly in Figure 5-31. Its lower crash potential and agreement with the earlier result make it the best case.
Figure 5-30: Crash potentials downstream of the detector of interest, Station 43, for the first 5 scenarios in Table 5-3.
Figure 5-31: Crash potentials downstream of the detector of interest, Station 43, for the last 4 scenarios in Table 5-3.
Figure 5-32: Crash potentials downstream of the detector of interest, Station 39, for the first 5 scenarios in Table 5-3.
Figure 5-33: Crash potentials downstream of the detector of interest, Station 39, for the last 4 scenarios in Table 5-3.
Figure 5-34: Crash potentials downstream of the detector of interest, Station 36, for the first 5 scenarios in Table 5-3.
Figure 5-35: Crash potentials downstream of the detector of interest, Station 36, for the last 4 scenarios in Table 5-3.

5.4.2.3 Time Scenarios

The next step is to consider the effect of the implementation of the speed limit patterns and distances over time. By fixing the pattern and the distance that will be affected, the only factor is how quickly to raise and lower the speed limits. Because the difference in speeds is so large, it is expected that a slow rate of implementing the change in speed will work out best. Table 5-4 shows the scenarios that are tested.

In Table 5-4 there are two variables the time between changes in speed limit and the amount of the change in speed limit. The time between changes can be either 5 minutes or 10 minutes and the amount of change can be either 5 mph or 10 mph. For example if the change is 5 mph every 5 minutes, then 10 minutes before the change for the abrupt
case the speed limit steps up 5 mph for the detectors downstream, five minutes later the speed downstream is 10 mph higher than the original speed limit, and then when the abrupt case would just be changing, the speed limit would be 15 mph higher than the original speed limit.

Table 5-4: Different scenarios based on changing the temporal implementation.

<table>
<thead>
<tr>
<th>Case</th>
<th>Time</th>
<th>Change</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>Abrupt</td>
<td></td>
</tr>
<tr>
<td>Case 2</td>
<td>5 min</td>
<td>5 &amp; 10 mph</td>
</tr>
<tr>
<td>Case 3</td>
<td>10 min</td>
<td>5 &amp; 10 mph</td>
</tr>
<tr>
<td>Case 4</td>
<td>5 min</td>
<td>10 mph</td>
</tr>
<tr>
<td>Case 5</td>
<td>10 min</td>
<td>10 mph</td>
</tr>
<tr>
<td>Case 6</td>
<td>5 min</td>
<td>5 mph</td>
</tr>
</tbody>
</table>

*Changes were made to both the upstream and downstream stations.

As shown in Figure 5-36, all of the cases reduce crash potential quickly and efficiently at the detector of interest. However, Case 3 has the most dramatic effect just downstream, as shown by Figure 5-37. Further downstream, as shown in Figure 5-37, Case 3 has an elevated crash potential for a longer time, but does not have a large value at anytime relative to the other cases. As a result Case 3 is the best scenario.
Figure 5-36: Crash potentials at the detector of interest, Station 47, for the scenarios in Table 5-4.
Figure 5-37: Crash potentials downstream of the detector of interest, Station 46, for the scenarios in Table 5-4.
Figure 5-38: Crash potentials downstream of the detector of interest, Station 45, for the scenarios in Table 5-4.

5.4.2.4 Gap Scenarios

The last step is to consider the effect of the gap distance on crash potential. Because raising the speed limits should help only in locations where there is enough freedom for the driver to increase their speed safely and the crash algorithm work by identifying location with high occupancy (low freedom) just upstream as dangerous, it is expected that a gap of 1 to 2 miles will be ideal. Table 5-5 shows the scenarios that are tested. In Table 5-5 there is only one variable, the gap length. Figure 5-39 below shows what location these distances translate into when comparing to the real world. Keep in mind that in the figure detector 47 is the detector of interest.
Table 5-5: Different scenarios based on changing the gap distance.

<table>
<thead>
<tr>
<th>Case 1</th>
<th>0 mi</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 2</td>
<td>1 mi</td>
</tr>
<tr>
<td>Case 3</td>
<td>2 mi</td>
</tr>
<tr>
<td>Case 4</td>
<td>3 mi</td>
</tr>
</tbody>
</table>

Figure 5-39: Sample scenario for a westbound roadway.

Figure 5-40 clearly shows that Case 1 is the best case. In fact Case 1 is the only case where the VSL have an effect. While this solution is not intuitive it is consistent with other findings (Lee et al., 2004).
Figure 5-40: Crash potentials at the detector of interest, Station 47, for the scenarios in Table 5-5.

5.5 Ramp Metering

The multiple tests conducted left a strong recommendation for the high speed model, but the low-speed model did not have a successful implementation of VSL. In order to try and find a satisfactory result ramp metering was attempted. Table 5-6 shows the scenarios that are tested. Ramp metering in this case means simply reducing the ramp counts by 40%. This will reduce the strength of the peak flows on the ramps and give a metering like effect. While the method is not the most elegant it is quick to implement. In the second case the scenario chosen for the high-speed model is used in conjunction with ramp metering to see it effect on the low-speed model.
Table 5-6: Different scenarios for the low speed model.

<table>
<thead>
<tr>
<th>Case 1</th>
<th>Ramp metering</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 2</td>
<td>Ramp Metering and High-Speed Result</td>
</tr>
</tbody>
</table>

The results in Figure 5-41 show that neither ramp metering nor the best high-speed scenario impacts the low-speed model.

![Graph showing crash potentials at Station 47 with 90% loading.](image)

Figure 5-41: Crash potentials at the detector of interest, Station 47, for the scenarios in Table 5-5.

To test the use of ramp metering in the low speed situation further a different form of representing ramp metering is used and it is used in conjunction with VSL implementation. Instead of metering all of the ramps equally the ramp that has the largest effect is blocked. This extreme case is used not to illustrate a step that should be taken in the field, but rather to see if ramp meter may hold a potential solution. The VSL
implementation is equally extreme. The detector that shows the largest crash potential will have all speed limits upstream lowered by 15 mph and all speed limits downstream raised by 15 mph. The implementation is abrupt in time and space with zero gap distance. The implementation start and finish time varies to see its effect.

Figure 5-46 shows the station with the worst crash potential, station 56. Station 56 is just after an on ramp, so that ramp (SR 436) is closed for some of the test cases. The figure also shows that the crash potential changes over time. By implementing the VSL at different times the use of VSL during high crash potentials, before high crash potentials, and during the transition in potential. Table 5-7 shows the cases that are implemented. Cases 0 and 4 do not involve the use of VSL. Cases 1 and 5 take place after crash potential has risen. Cases 2 and 6 take place before crash potential has risen. Cases 3 and 7 take place while crash potential is increasing.

Table 5-7: Cases for ramp metering and VSL implementation.

<table>
<thead>
<tr>
<th>Case</th>
<th>Implementation</th>
<th>Amount of Change</th>
<th>VSL</th>
<th>Ramp Metering</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Abrupt</td>
<td>15 mph</td>
<td>None</td>
<td>NO</td>
</tr>
<tr>
<td>1</td>
<td>Abrupt</td>
<td>15 mph</td>
<td>17:30-18:00</td>
<td>NO</td>
</tr>
<tr>
<td>2</td>
<td>Abrupt</td>
<td>15 mph</td>
<td>16:30-18:00</td>
<td>NO</td>
</tr>
<tr>
<td>3</td>
<td>Abrupt</td>
<td>15 mph</td>
<td>17:00-18:30</td>
<td>NO</td>
</tr>
<tr>
<td>4</td>
<td>Abrupt</td>
<td>15 mph</td>
<td>None</td>
<td>YES</td>
</tr>
<tr>
<td>5</td>
<td>Abrupt</td>
<td>15 mph</td>
<td>17:30-18:00</td>
<td>YES</td>
</tr>
<tr>
<td>6</td>
<td>Abrupt</td>
<td>15 mph</td>
<td>16:30-18:00</td>
<td>YES</td>
</tr>
<tr>
<td>7</td>
<td>Abrupt</td>
<td>15 mph</td>
<td>17:00-18:30</td>
<td>YES</td>
</tr>
</tbody>
</table>
Figure 5-42: Base scenario for the worst case (Detector 56, upstream of the station of interest)

Figure 5-43 shows the implementation of the low-speed algorithm at Detector 56. From this figure it appears that ramp metering has an effect, but the VSL do not. To make sure that the algorithms are being applied correctly the average speed for each scenario is measured and results are shown in Figure 5-44. The figure shows that the ramp metering moves the average speed into the high-speed regime. As a result, the cases are rerun with the high-speed algorithm. Figure 5-45 shows the output. Station 56 does not show a difference in crash potential for the different scenarios. However, Station 58 (in Figure 5-46) does show a difference in crash potential.

The application of just the high-speed algorithm to all cases is no more valid than the application of the low-speed algorithm therefore an alternative method is explored. Figure 5-47 shows the high-speed cases evaluated by the high-speed algorithm and the
low-speed cases evaluated by the low-speed algorithm. The algorithms do not appear to be on the same scale, because of the degree of difference in the scenarios and Figure 5-47 shows the opposite result as Figure 5-43 and Figure 5-46. The only option that appears to be open to compare high-speed and low-speed cases is Abdel-Aty’s et al. (2004) algorithm, which will be referred to as the generic algorithm (Equation 4-2).

Figure 5-47 shows the implementation of the generic algorithm. Ramp metering appears to have a strong positive effect, but just as important the VSL show a difference in response when no ramp metering is present. Cases 2 and 3 appear to outperform the no VSL case, while Case 4 seems to perform worse than the base. Cases 2 and 3 correspond to early implementation of the VSL, with the earliest implementation, Case 2, slightly outperforming all other cases. This behavior is also reflected in Figure 5-43, the low speed modeling. However, the effectiveness of VSL is dwarfed by that of ramp metering. In the ramp metering cases VSL have no significant impact. Therefore the conclusion from these tests is that ramp metering should be explored, without VSL, in depth in order to improve safety during high congestion periods.
Figure 5-43: Scenarios from Table 5-7 evaluated using the low-speed algorithm at Detector 56.
Figure 5-44: Scenarios from Table 5-7 average speeds at Detector 58 (Upstream).
Figure 5-45: Scenarios from Table 5-7 evaluated using the high-speed algorithm at Detector 56.
Figure 5-46: Scenarios from Table 5-7 evaluated using the high-speed and low-speed algorithm at Detector 58.
Figure 5-47: Scenarios from Table 5-7 evaluated using the generic algorithm at Detector 56.

### 5.6 High Speed Best Case Scenario

In the previous sections various test cases have determined the best scenario for implementing VSL. While no conclusive result about the 90% loaded case maybe determined it does provide evidence that ramp metering could lead to greater safety improvement than VSL. The 60% case did show a clear positive result.

In the first round of tests the pattern of variable speed limit was tested. The patterns consisted of scenarios with a rate of change in speed limit over space that is either abrupt or gradual and a maximum speed change of either 15 mph or 10 mph. The results showed that an abrupt change of 15 mph produced the best result.
In the second round of scenarios the length upstream and downstream of the detector of interest was tested. The different lengths for both the upstream and downstream changes attempted were 2 miles, 4 miles, and 5.5 miles. The results showed that a distance of 2 miles upstream and 2 miles downstream produced the best results.

In the third round of scenarios the rate of implementation of speed limits over time was tested. Changes in speed limit of 5 mph and 10 mph were tested along with time between changes of 5 minutes and 10 minutes. The results showed that a change of 5 minutes produced the best results.

In the forth and final test the effect of gap distance on crash potential was tested. Gap distance of 0 miles, 1 mile, 2 miles, and 3 miles were tested. The results showed that a gap of 0 miles produced the best results.

The best case is therefore an abrupt 15 mph change in speed limits both 2 miles upstream decrease and downstream increase, implemented in 5 mph increments every 10 minutes. Taking a closer look at the recommended case shows that the VSL have both positive and negative effect. Figures 5-48 and 5-49 shows that the condition at station 51 and station 45 are worse with the implementation of VSL, while Figures 5-50 and 5-51 shows that conditions at station 48 and station 47 are greatly improve. Other stations shown in Figures 5-52 through 5-53 show minor differences when using VSL. While the trend appears to be that the leading and the station one mile downstream of the station of interest edge have problems, while the stations where the speed limit is being raised are improved. To make sure that these effects are coming as a result of the VSL and are not due to the randomness in the system twenty matched runs of the base case and the recommended case are undertaken.
Figure 5-48: The crash potential at Station 51 (Upstream) for the best scenario under 60% loading.
Figure 5-49: The crash potential at Station 45 (Downstream) for the best scenario under 60% loading.
Figure 5-50: The crash potential at Station 48 (Upstream) for the best scenario under 60% loading.
Figure 5-51: The crash potential at Station 47, (Location of Interest) the detector of interest, for the best scenario under 60% loading.
Station 50 (WB) with 60% Loading

Figure 5-52: The crash potential at Station 50 (Upstream) for the best scenario under 60% loading.
Figure 5-53: The crash potential at Station 49 (Upstream) for the best scenario under 60% loading.

5.6.1 Statistical Analysis

After running nineteen matched cases of a base scenario and a best-case scenario the results are analyzed using a t-test. The half width at 95% for each station is shown in Table 5-8. The resulting ranges are plotted in Figure 5-54. The half width is determined by multiplying the t-value by the standard deviation divided by the square root of the number of observations, 19. For the differences that are significant the percent change of each are calculated. The percent change is the difference between the best case and the base case (less their half width) normalized by the base case value. The result is shown in Table 5-6. Note that the dark gray boxes are used to show an improvement, while the
lighter shows a worsened condition. All values are risk values that were calculated using Dr. Abdel-Aty et al. (2005) high-speed model.

Figure 5-54: Crash risk with confidence intervals for all stations.
5.6.1.1 Station of Interest

The station of interest, Station 47, shows the largest change. With a positive change of 122% conditions at Station 47 move from being the most at risk station to being the least at risk station. Figure 5-51 also shows that the situation just upstream of the station of interest shows great improvement.

<table>
<thead>
<tr>
<th>Station</th>
<th>Best Case Average</th>
<th>Half Width</th>
<th>Base Case Average</th>
<th>Half Width</th>
<th>Percent Change</th>
</tr>
</thead>
<tbody>
<tr>
<td>64</td>
<td>-0.267</td>
<td>0.001</td>
<td>-0.266</td>
<td>0.001</td>
<td></td>
</tr>
<tr>
<td>63</td>
<td>-0.238</td>
<td>0.001</td>
<td>-0.235</td>
<td>0.001</td>
<td>1%</td>
</tr>
<tr>
<td>62</td>
<td>-0.268</td>
<td>0.001</td>
<td>-0.267</td>
<td>0.001</td>
<td></td>
</tr>
<tr>
<td>61</td>
<td>-0.349</td>
<td>0.001</td>
<td>-0.348</td>
<td>0.001</td>
<td></td>
</tr>
<tr>
<td>60</td>
<td>-0.197</td>
<td>0.002</td>
<td>-0.194</td>
<td>0.002</td>
<td></td>
</tr>
<tr>
<td>59</td>
<td>-0.181</td>
<td>0.001</td>
<td>-0.179</td>
<td>0.001</td>
<td></td>
</tr>
<tr>
<td>58</td>
<td>-0.364</td>
<td>0.001</td>
<td>-0.361</td>
<td>0.001</td>
<td>1%</td>
</tr>
<tr>
<td>57</td>
<td>-0.361</td>
<td>0.002</td>
<td>-0.359</td>
<td>0.001</td>
<td></td>
</tr>
<tr>
<td>56</td>
<td>-0.292</td>
<td>0.002</td>
<td>-0.292</td>
<td>0.001</td>
<td></td>
</tr>
<tr>
<td>55</td>
<td>-0.254</td>
<td>0.001</td>
<td>-0.250</td>
<td>0.001</td>
<td>1%</td>
</tr>
<tr>
<td>54</td>
<td>0.065</td>
<td>0.002</td>
<td>-0.292</td>
<td>0.002</td>
<td>-122%</td>
</tr>
<tr>
<td>53</td>
<td>-0.365</td>
<td>0.002</td>
<td>-0.365</td>
<td>0.002</td>
<td></td>
</tr>
<tr>
<td>52</td>
<td>-0.368</td>
<td>0.001</td>
<td>-0.280</td>
<td>0.001</td>
<td>31%</td>
</tr>
<tr>
<td>51</td>
<td>-0.221</td>
<td>0.001</td>
<td>-0.143</td>
<td>0.001</td>
<td>55%</td>
</tr>
<tr>
<td>50</td>
<td>-0.317</td>
<td>0.002</td>
<td>-0.327</td>
<td>0.001</td>
<td>-3%</td>
</tr>
<tr>
<td>49</td>
<td>-0.292</td>
<td>0.002</td>
<td>-0.346</td>
<td>0.002</td>
<td>-16%</td>
</tr>
<tr>
<td>48</td>
<td>-0.321</td>
<td>0.002</td>
<td>-0.465</td>
<td>0.002</td>
<td>-31%</td>
</tr>
<tr>
<td>47</td>
<td>0.065</td>
<td>0.002</td>
<td>-0.292</td>
<td>0.002</td>
<td></td>
</tr>
<tr>
<td>46</td>
<td>-0.249</td>
<td>0.002</td>
<td>-0.246</td>
<td>0.002</td>
<td>1%</td>
</tr>
<tr>
<td>45</td>
<td>-0.486</td>
<td>0.002</td>
<td>-0.324</td>
<td>0.002</td>
<td>50%</td>
</tr>
<tr>
<td>44</td>
<td>-0.157</td>
<td>0.002</td>
<td>-0.212</td>
<td>0.002</td>
<td>-26%</td>
</tr>
<tr>
<td>43</td>
<td>-0.201</td>
<td>0.002</td>
<td>-0.139</td>
<td>0.002</td>
<td>45%</td>
</tr>
<tr>
<td>42</td>
<td>0.162</td>
<td>0.002</td>
<td>-0.104</td>
<td>0.003</td>
<td>57%</td>
</tr>
<tr>
<td>41</td>
<td>-0.419</td>
<td>0.003</td>
<td>-0.415</td>
<td>0.003</td>
<td></td>
</tr>
<tr>
<td>40</td>
<td>0.395</td>
<td>0.003</td>
<td>-0.383</td>
<td>0.003</td>
<td>3%</td>
</tr>
<tr>
<td>39</td>
<td>0.184</td>
<td>0.002</td>
<td>-0.160</td>
<td>0.002</td>
<td>15%</td>
</tr>
<tr>
<td>38</td>
<td>0.539</td>
<td>0.002</td>
<td>-0.512</td>
<td>0.003</td>
<td>5%</td>
</tr>
<tr>
<td>37</td>
<td>0.484</td>
<td>0.002</td>
<td>-0.447</td>
<td>0.002</td>
<td>8%</td>
</tr>
<tr>
<td>36</td>
<td>0.337</td>
<td>0.003</td>
<td>-0.319</td>
<td>0.002</td>
<td>5%</td>
</tr>
<tr>
<td>35</td>
<td>-0.275</td>
<td>0.002</td>
<td>-0.276</td>
<td>0.002</td>
<td></td>
</tr>
<tr>
<td>34</td>
<td>-0.251</td>
<td>0.002</td>
<td>-0.253</td>
<td>0.002</td>
<td></td>
</tr>
</tbody>
</table>
5.6.1.2 Upstream of the Station of Interest

As mentioned before the possibility of crash relocation is the biggest concern. Intuitively, crash potential would be more likely to relocate upstream of the detector of interest in the high-speed case. Abdel-Aty’s et al. (2005) algorithm can be interpreted to say that platoons of faster moving vehicle coming into contact with slower traffic cause high crash potential. If this is the case after the speed limits have been changed there is a region of high speed that suddenly comes on at a lower speed limit. The lower speed limit increases the density of the vehicles raising the loop detector’s occupancy, effectively creating the situation that the VSL are attempting to solve.

However, looking at the simulation results shows that the upstream the situation does not change much, except at the detectors where the sudden change in speed takes place. Here at station 51 and station 52 the crash potential increases, but never near the level of what station 47 was prior to the implementation of VSL.

5.6.1.3 Downstream of the Station of Interest

Making a case for crash potential relocating to a downstream station is a little more difficult. Increasing the speed limit could clear large packs, but geometries downstream (a lane drop or curve) could induce a bottleneck that causes packs to reform. However, VSL themselves would not be thought of as the direct cause of these increases in crash potential.

The results here are surprising. The crash potential appears to relocate to the detectors downstream of the detector of interest. The relocation occurs both one and two
miles downstream of detector 47. However, the station 1.5 miles downstream sees an improvement in crash potential. Geometry could be the culprit as there are curves at these locations as there are curves present near these detectors.

5.6.2 Travel Time Analysis

With a best case scenario for safety in hand the effect of the scenario on travel time is evaluated. Because base scenario is an off-peak case no improvement in travel time is expected to be seen, instead a success implementation is expect to have minimal effect on travel time.

To analyze the effect on travel time the same seventy runs of the base case and best case from section 5.6 are used and the travel time on each link across the entire three-hour simulation is summed. Because the scenarios have a common random number seed a paired t-test can be used. The paired t-test and a 95% confidence interval for both scenarios are shown in Table 5-9.

Table 5-9 shows there is a significant reduction in travel time due to the implementation of VSL. This result means that the best case can be concluded to effectively reduce crash potential and decrease travel times, meaning that crash migration is the only foreseeable drawback. In this case the overall maximum reduction in crash potential from 0.065 to -.104 even when considering crash migration, which is a significant improvement in safety for the roadway.
Table 5-9: T-test for the network travel time (minutes)

<table>
<thead>
<tr>
<th>Base Scenario</th>
<th>Best Scenario</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>36776.3</td>
<td>36807.6</td>
<td>-278.0</td>
</tr>
<tr>
<td>36868.9</td>
<td>36822.2</td>
<td>-121.3</td>
</tr>
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Average: -189.9
95% C.I.: 25.6

5.6.3 Model Application

For completeness the average speed of the base and best case scenarios are determined and shown to be within the regime of the applied model, as shown in Figure 5-55.
Figure 5-55: Base and Best Case Scenarios’ average speed at Detector 47 (Detector of Interest).

5.7 Best Low Speed Scenario

The best low-speed case involves the implementation of drastic ramp metering just upstream of the detector of interest without VSL. VSL has effect when ramp metering is not used, but fails to have effect when ramp metering is used. Figure 5-55 shows a plot of the best case versus the base case for detector 56, the detector of interest.
Figure 5-56: Best scenario from Table 5-7 (Case 4) evaluated using the generic algorithm at Detector 56.
CHAPTER SIX: CONCLUSION

The objective of this research was to explore different ITS strategies to improve safety on Interstate 4 (I-4), specifically variable speed limits (VSL) and ramp metering applications. Through the implementation of variable speed limits on I-4, via simulation, a best case for improving safety was determined. Multiple scenarios with characteristic trends in them were to be used to generate general “rules” or best practices that result in an optimal safe condition. The work centered on a single site for improvement. However, the relocation of crash risk was considered when making recommendations and the single site location was limited only by the length of roadway simulated. Two loading conditions, a high speed (low loading, 60%) and a low speed (high loading 90%), were considered.

The objective is successfully achieved for the high speed case. Through 4 rounds of testing a final best model is obtained. The first round considered is the spatial variation of various speed limits (whether upstream lowering the speed limits and downstream raising speed limits is better than just downstream raising speed limits), speed limit variation (whether a 10 mph maximum change is better than a 15 mph speed limit change), and the combination of the two variables (whether gradually changing the speed limit over space or abruptly changing the speed limit over space is better). The abrupt 15 mph change both upstream and downstream are concluded to produce the best case. The second round tested the spatial location of changing speed limit signs. In this test the upstream and downstream distances are set as 2 miles, 4 miles, or 5.5 miles and the impact on crash potential evaluated. The shortest downstream distance, 2 miles,
improved safety, while the shortest upstream distance, 2 miles, was used to have the least impact on travel time. The third round tested temporal implementation of the VSL on the best case scenario from test 2. Step size for VSL implementation was either 5 mph or 10 mph and step either every five minutes or every 10 minutes. The most gradual implementation, 5 mph steps every 10 minutes, had the best result. The forth round tested the gap distance used, after the detector of high incidence, before raising the speed limit. A gap of 0 miles greatly outperformed all other cases.

As a result of this best practice determination existing VSL can be thought of in a new light. Instead of just using them before or during periods of high congestion, VSL can be thought of as applicable during off-peak periods as well. Instead of aiming to reduce congestion time the VSL can be used to effectively reduce the hazard of certain locations. This study also concluded that by using VSL in off-peak conditions travel time is not effected, so there are no negative effects of having implemented their use with regard to delay. When implementing VSL practitioners should follow the recommendations of this study:

- Gradually introduce speed changes in time (5 mph every 10 minutes)
- Abruptly introduce speed changes in space (No gap distance)
- Use upstream reductions in speed and downstream increases in speed
- Changes speed limit by large values (15 mph)
- Short upstream and downstream distance are sufficient (2 miles each)

While the low-speed case analysis did not show clear results, it did indicate future areas that should be studied. VSL were introduced via the scenario shown in Table 6-1 and through the best case determined in the high-speed case. The scenario has a large
change in speed limit and is introduced gradually in space and time. Downstream raising is not performed, because the driver’s speed is being limited by factors outside of the speed limit and therefore any change is assumed to not have an effect. VSL implementation showed no significant change in crash potential during the simulation. As a result more drastic measures were taken and ramp metering was approximated using 60% loading from all ramps in one case and through shutting off a problem ramp in another case. While the first implementation of ramp metering did not have an effect the ramp specific approach appears to effectively reduce crash potential.

Table 6-1: The test scenario for the low speed model.

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The reduction in crash potential was not analyzed and the method of ramp metering is crude, but is suggestive that if ramp metering via a ramp just downstream of the detector of interest is used to effectively reduce congestion that there might be a safety benefit as well. Future research with a more realistic implementation of ramp metering should be looked into because of this result. Also Borough’s (1997) work suggests that lane changing behavior is an important factor in high congestion situation, therefore the impact of VSL implementation on lane change behavior could be an important factor that is missing for PARAMICS and therefore this thesis. It is the author’s belief that this factor is the most significant factor in seeing reductions in crash due to VSL implementation, and should be studied in detail.
APPENDIX A: EIGHT SCENARIOS FOR THE LOW SPEED CASES
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88  87  95  83  81  80
78  76  75  73  71  70
period 3 start 18:30:00
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simulation time  03:15:00
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seed  1234
split seed
demand matrix tuning level  0
generator  0
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closest destination vehiclepark  enabled
file time  ":-"
curve speed factor  1.000
amber time  3.000
speed drift  5
maximum diversion  300
right hand drive
units us
timestep detail  2
mean headway  1.00
mean reaction time  .42
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weight heavy  2.000 ton
feedback  00:00:00
feedback smoothing factor  0.500
feedback decay factor  0.995
perturbation disabled
plan count 4

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loops 1
parameters 0
if (init)
{
    variable;
}
if (count [1] type [17] >= 1)
{
    green1 = 0; ##
    green2 [2] = 40;
}

plan 2 definition
loops 1
parameters 0
if (time = 1)
{
    green2 [2] = 0;
    clear [1];
}

plan 3 definition
loops 1
parameters 0
if (init)
{
    variable;
}
if (count [1] type [17] >= 1)
{
    green1 = 0; ##
    green2 [3] = 40;
}

plan 4 definition
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<td>from 67</td>
<td>2</td>
<td>0</td>
</tr>
<tr>
<td>from 69</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>from 73</td>
<td>391</td>
<td>0</td>
</tr>
</tbody>
</table>
APPENDIX D: VERIFICATION STEPS
<table>
<thead>
<tr>
<th>Problem</th>
<th>Action Taken to Correct</th>
</tr>
</thead>
<tbody>
<tr>
<td>Awareness Distances (All)</td>
<td>Corrected to 500ft</td>
</tr>
<tr>
<td>Princeton W congestion</td>
<td>Lowered Speed Limit on Colonial W exit, causing backup</td>
</tr>
<tr>
<td>Princeton E congestion</td>
<td>Corrected Geometry</td>
</tr>
<tr>
<td>Michigan E Congestion</td>
<td>Adjusted Nextlanes to force space</td>
</tr>
<tr>
<td>EW Congestion, no merging</td>
<td>Reconstructed Interchange increase awareness distance, added</td>
</tr>
<tr>
<td></td>
<td>lanes</td>
</tr>
<tr>
<td>Lake Mary W congestion</td>
<td>Corrected Kerb Points</td>
</tr>
</tbody>
</table>
Common Code

The following sets of subroutines are code common to all of the methods used. Notice that the occupancy and flow subroutine is not called. By substituting the call for speed with the call for the occupancy and flow subroutine occupancy and flow data can be returned instead of speed data.
Subroutine ReseedI

Sub ReseedI()

Dim seed As Integer
Dim casenum As String
Dim runnum As String
Dim loading As String
Dim seedstr As String

runnum = (InputBox("What is the run number?
loading = (InputBox("What is the loading? (100, 075, 060)\n"))
casenum = InputBox("What is the case time? (1645, 1720, and 1800)\n")
Application.ScreenUpdating = False
For seed = 0 To 2
    seedstr = seed
    Call SpeedCal(seed, casenum, runnum)
    Call DataPrep(seed, runnum)
    ActiveWorkbook.SaveAs Filename:= "C:\Jeremy\I-4 Project\Data generation\Raw Data\" + loading + "Case " + casenum + "Run "+ seedstr + ":.xls", FileFormat:= xlNormal, Password:=", WriteResPassword:=", ReadOnlyRecommended:=False 
, CreateBackup:=False
    Call Remaking(sheetname, seed)
Next seed
Application.ScreenUpdating = True
End Sub
Subroutine SpeedCal

Sub SpeedCal(seed As Integer, casenum As String, runnum As String)

Dim x As String
Dim y As String
Dim z As String
Dim q As String
Dim p As String
Dim accumstr As String
Dim county As String
Dim strloop As String
Dim lane As String
Dim namefile As String
Dim DatType As String
Dim holder As String
Dim j As Integer
Dim k As Integer
Dim l As Integer
Dim accum As Integer
Dim numloop As Integer
Dim numlane As Integer
Dim runint As Integer
Dim numcase As Integer
Dim jstr As String
Dim kstr As String
Dim sheetnumstr As String
Dim headed As String
Dim rangestr As String
Dim seedstr As String
Dim xstr As String
Dim xstr1 As String
Dim xstr2 As String
Dim seedstr2 As String
Dim seedevalstrt As Integer
Dim seedevalend As Integer
Dim M As Integer

If seed = 0 Then
    For j = 41 To 1 Step -1  'Add sheets and name them
        seedstr = seed
        xstr = j + 27
        xstr = "Detect" + xstr + "_seed_" + seedstr
        Sheets.Add
        ActiveSheet.Name = xstr
        Sheets("Sheet37").Select
        Columns("A:A").Select
        Selection.Copy
        Sheets(xstr).Select
        Columns("A:A").Select
        ActiveSheet.Paste
    Next j
End If
For k = 1 To numloop Step 2
    If k < 10 Then  'Detector number
        strloop = "000" & k
    Else
        strloop = "00" & k
    End If
    'Count number
    kstr = k
    If k Mod 2 = 1 Then
        sheetnumstr = (81 - k) / 2 + 28
        headed = "West"
    Else
        sheetnumstr = (82 - k) / 2 + 28
        headed = "East"
    End If
    numlane = 3  'Number of Lanes
    For M = 1 To numlane 'Current Lane
        If (k <= 72 And k >= 71) Then
            lane = M + 1
        Else
            lane = M
        End If
        sheetname = "speed " + strloop + " " + seedstr + " "
        ActiveSheet.Name = sheetname

        'text file name
        If k < 10 Then
            p = DatType + "0" + kstr + "-Detector" + strloop + "-L" + lane
        Else
            p = DatType + kstr + "-Detector" + strloop + "-L" + lane
        End If

        'file location
        If seed < 10 Then
            If runint < 10 Then
                q = "$TEXT;C:\Jeremy\I-4 Project\I-4\Log\RUN-00" + runnum + "$\process-00" + seedstr + "$\" + p
            Else
                q = "$TEXT;C:\Jeremy\I-4 Project\I-4\Log\RUN-0" + runnum + "$\process-00" + seedstr + "$\" + p
            End If
        Else
            If runint < 10 Then 
                q = "$TEXT;C:\Jeremy\I-4 Project\I-4\Log\RUN-00" + runnum + "$\process-00" + seedstr + "$\" + p
            Else 
                q = "$TEXT;C:\Jeremy\I-4 Project\I-4\Log\RUN-0" + runnum + "$\process-00" + seedstr + "$\" + p
            End If
        End If
    Next M
Next k
q = "TEXT;C:\Jeremy\I-4 Project\I-4\Log\RUN-00" + runnum + "\process-0" + seedstr + ";" + p
Else
q = "TEXT;C:\Jeremy\I-4 Project\I-4\Log\RUN-00" + runnum + "\process-0" + seedstr + ";" + p
End If
End If

With ActiveSheet.QueryTables.Add(Connection:=q, Destination:=Range("A1"))
.Name = "p"
.FieldNames = True
.RowNumbers = False
.FillAdjacentFormulas = False
.PreserveFormatting = True
.RefreshOnFileOpen = False
.RefreshStyle = xlInsertDeleteCells
.SavePassword = False
.SaveData = True
.AdjustColumnWidth = True
.RefreshPeriod = 0
.TextFilePromptOnRefresh = False
.TextFilePlatform = 437
.TextFileStartRow = 1
.TextFileParseType = xlDelimited
.TextFileTextQualifier = xlTextQualifierDoubleQuote
.TextFileConsecutiveDelimiter = True
.TextFileTabDelimiter = False
.TextFileSemicolonDelimiter = False
.TextFileCommaDelimiter = False
.TextFileSpaceDelimiter = True
.TextFileColumnDataTypes = Array(1, 1, 1, 1, 1)
.TextFileTrailingMinusNumbers = True
.Refresh BackgroundQuery:=False
End With

sheetname2 = "Detect" + sheetnumstr + "_seed_" + seedstr

Call MinAndMax(M) 'it calls speed

Range("C31:C390").Select
Selection.Copy
Sheets(sheetname2).Select
Cells.Select
Selection.ClearContents
Range("A1").Select

Next M
Next k

End Sub
Subroutine MinAndMax

Sub MinAndMax(M As Integer)
'
' Macro2 Macro
' Macro recorded 8/14/2004 by dilmorej
'

Dim counter As Integer
Dim Min(1 To 360) As String
Dim Max(1 To 360) As String
Dim y As Integer
Dim ystr1 As String
Dim ystr2 As String
Dim ymax As Integer

For counter = 1 To 360
Min(counter) = "0"
Max(counter) = "0"
Next counter

For y = 1 To 10000
ystr1 = y
If Range("A" + ystr1).Value = "" Then
    y = 10000
Else
    ystr2 = y + 2
    Rows(ystr1 + ":" + ystr2).Select
    Selection.Delete Shift:=xlUp
    ymax = y
    y = y + 20
End If
Next y

Columns("A:B").Select
Selection.NumberFormat = "0.0000000000"
Selection.Sort Key1:=Range("A1"), Order1:=xlAscending, Header:=xlGuess, _
    OrderCustom:=1, MatchCase:=False, Orientation:=xlTopToBottom, _
    DataOption1:=xlSortNormal
For y = 1 To ymax + 22
ystr1 = y
If Range("A" + ystr1).Value = "" Then
    ElseIf Range("A" + ystr1).Value <= (1920 / 2880) Then
        Min(1) = y + 1
    Else
        For counter = 1 To 359
            If Range("A" + ystr1).Value <= ((1920 + counter) / 2880) Then
                Min(counter + 1) = y + 1
                Max(counter) = y
                counter = 359
            End If
        Next counter
        Min(360) = y
    End Else
    For counter = 1 To 359
        If Range("A" + ystr1).Value <= ((1920 + counter) / 2880) Then
            Min(counter + 1) = y + 1
            Max(counter) = y
        End If
    Next counter
End If
Next y

If Range("A" + ystr1).Value <= (2280 / 2880) Then
    Max(360) = y
End If
End If
End If
Next y

Call error_catch(Min(), Max(), M)

End Sub
Subroutine error_catch

Sub error_catch(Min() As String, Max() As String, M As Integer)

Dim counter As Integer

For counter = 2 To 360
    If Min(counter) = 0 Then Min(counter) = Min(counter - 1)
    If Max(counter) = 0 Then Max(counter) = Min(counter)
Next counter

Sheets(sheetname2).Select
Call OutputMinMax(Min(), Max(), M)
Sheets(sheetname).Select
Call Speed(Min(), Max())

End Sub
Subroutine OutputMinMax

Sub OutputMinMax(Min() As String, Max() As String, M As Integer)

Dim rangestr As String
Dim i As Integer
Dim istr As String

If M = 1 Then rangestr = "E"
If M = 2 Then rangestr = "F"
If M = 3 Then rangestr = "G"

For i = 1 To 360
    'MsgBox (rangestr + istr)
    istr = i
    Range(rangestr + istr).Value = Max(i) - Min(i)
Next i

End Sub
Subroutine Speed

Sub Speed(Min() As String, Max() As String)
'
'Macro5 Macro
'Macro recorded 8/21/2004 by dilmore
'
'
'Cells.Select
'Selection.ClearContents
Dim counter As Integer
Dim counterstr As String
Dim counter2 As Integer
Dim counter2str As String

For counter = 1 To 360
    If Max(counter) = 0 Or Min(counter) = 0 Then
        counterstr = counter
        Range("F" + counterstr).Value = 0
    Else
        counterstr = counter
        accum = 0
        For counter2 = Min(counter) To Max(counter)
            counter2str = counter2
            accum = Range("C" + counter2str).Value + accum
        Next counter2
        If (Max(counter) - Min(counter) + 1) < 1 Then
            Range("F" + counterstr).Value = 0
        Else
            Range("F" + counterstr).Value = accum / (Max(counter) - Min(counter) + 1)
        End If
    End If
Next counter

Columns("F:F").Select
Selection.NumberFormat = "0.0"
Selection.Copy
Columns("A:C").Select
Selection.Delete Shift:=xlToLeft
Rows("1:30").Select
Selection.Insert Shift:=xlDown

End Sub
Subroutine OccAndCount

Sub OccAndCount(Min() As String, Max() As String)

' Macro3 Macro
' Macro recorded 7/19/2004 by dilmorej

'Cells.Select
'Selection.ClearContents
Dim counter As Integer
Dim counterstr As String

For counter = 1 To 360
    counterstr = counter
    If Max(counter) = 0 Or Min(counter) = 0 Then
        counterstr = counter
        Range("F" + counterstr + ":G" + counterstr).Value = 0
    Else
        counterstr = counter
        Range("F" + counterstr).Value = ">=sum(B" + Min(counter) + ":B" + Max(counter) + ")/30"
        Range("G" + counterstr).Value = (Int(Max(counter)) - Int(Min(counter))) + 1
    End If
Next counter

Columns("F:F").Select
Selection.NumberFormat = "0.00000"
Columns("F:G").Select
Selection.Copy
Columns("A:C").Select
Selection.Delete Shift:=xlToLeft
Rows("1:30").Select
Selection.Insert Shift:=xlDown

End Sub
Subroutine DataPrep

Sub DataPrep(seed As Integer, runnum As String)
'
' Macro2 Macro
' Macro recorded 9/11/2004 by dilmorej
'
'
Dim seedstr As String
Dim detectornum As Integer
Dim detectorstr As String

    seedstr = seed
    For detectornum = 68 To 28 Step -1
        detectorstr = detectornum
        Sheets("Detect" + detectorstr + ".seed_." + seedstr).Select
        Range("A1").Select
        Call SimpleFormula
    Next detectornum
    Call AdvancedFormula(seedstr)

End Sub
Subroutine Remaking

Sub Remaking(sheetname As String, seed As Integer)

    Dim seedstr As String
    Dim seedstr2 As String
    Dim xstr As String
    Dim xstr1 As String
    Dim xstr2 As String

    Sheets(sheetname).Select
    ActiveSheet.Name = "Sheet37"
    Range("A1").Select
    ActiveCell.FormulaR1C1 = "4:00:00 PM"
    Range("A2").Select
    ActiveCell.FormulaR1C1 = "4:00:30 PM"
    Range("A1:A2").Select
    Selection.AutoFill Destination:=Range("A1:A360"), Type:=xlFillDefault
    Range("A1:A360").Select

    For j = 41 To 1 Step -1
        seedstr = seed
        seedstr2 = seed + 1
        xstr = j + 27
        xstr1 = "Detect" + xstr + ".seed_" + seedstr
        xstr2 = "Detect" + xstr + ".seed_" + seedstr2
        'MsgBox (xstr)
        Sheets(xstr1).Name = xstr2
        Sheets(xstr2).Select
        Cells.Select
        Selection.ClearContents
        Range("A1").Select
    Next j

End Sub
Different Models Code

The following sets of subroutines are for the different models. First is Pande’s (et al. 2005) model. The following two are Abdel-Aty’s (et al. 2005) low speed and high speed models, respectively. Notice that each is made up of two separate subroutines: SimpleFormula and AdvancedFormula.
COMBINED MODEL
Subroutine SimpleFormula

Sub SimpleFormula()

' Macro1 Macro
' Macro recorded 9/10/2001 by APande
'

Range("H10").Select
ActiveCell.FormulaR1C1 = ":=AVERAGE(R[-9]C[-6]:RC[-4])"
Range("I10").Select
ActiveCell.FormulaR1C1 = ":=STDEV(R[-9]C[-7]:R[1]C[-5])"
Range("J10").Select
ActiveCell.FormulaR1C1 = ":=LOG((RC[-1]/RC[-2]),10)+2"
Range("K10").Select
ActiveCell.FormulaR1C1 = ":=AVERAGE(R[-9]C[-6]:RC[-4])"
Range("L10").Select
ActiveCell.FormulaR1C1 = ":=STDEV(R[-9]C[-7]:RC[-5])"
Range("M10").Select
ActiveCell.FormulaR1C1 = ":=LOG((RC[-1]/RC[-2]),10)+2"
Range("H10:M10").Select
Selection.AutoFill Destination:=Range("H10:M360"), Type:=xlFillDefault

End Sub
Subroutine AdvanceFormula

Sub AdvancedFormula(seedstr As String)
' Macro1 Macro

    Dim xstr As String
    Dim x As Integer
    Dim xstrminus1 As String
    Dim xstrminus2 As String
    Dim xstrminus3 As String
    Dim xstrminus4 As String
    Dim xstrplus1 As String
    Dim xstrplus2 As String
    Dim xstrplus3 As String
    Dim xstrplus4 As String
    Dim y As Integer
    Dim ystr As String

    For x = 28 To 68 Step 1
        xstr = x
        Sheets("Detect" + xstr + "_seed_" + seedstr).Select
        Columns("K:K").Select
        Selection.Insert Shift:=xlToRight
    Next x

    For x = 33 To 64 Step 1
        xstr = x
        xstrminus4 = x - 4
        xstrminus3 = x - 3
        xstrminus2 = x - 2
        xstrminus1 = x - 1
        xstrplus1 = x + 1
        xstrplus2 = x + 2
        xstrplus3 = x + 3
        xstrplus4 = x + 4
        Sheets("Detect" + xstr + "_seed_" + seedstr).Select
        For y = 10 To 360
            ystr = y
            Range("K" + ystr).Select
            ActiveCell.FormulaR1C1 = "=Detect" + xstrplus4 + "_seed_" + seedstr + "}RC[-1]*1.776+2.301*Detect" + xstrplus3 + "_seed_" + seedstr + "}RC[-1]+2.92*Detect" + xstrplus2 + "_seed_" + seedstr + "}RC[-1]+4.096*Detect" + xstrplus1 + "_seed_" + seedstr + "}RC[-1]+6.216*Detect" + xstr + "_seed_" + seedstr + "}RC[-1]+4.3*Detect" + xstrminus1 + "_seed_" + seedstr + "}RC[-1]+3.122*Detect" + xstrminus2 + "_seed_" + seedstr + "}RC[-1]+4.3*Detect" + xstrplus1 + "_seed_" + seedstr + "}RC[-1]+3.122*Detect" + xstrplus2 + "_seed_" + seedstr + "}RC[-1]"
            Range("O" + ystr).Select
            ActiveCell.FormulaR1C1 = "=Detect" + xstrminus4 + "_seed_" + seedstr + "}RC[-1]*1.776+2.301*Detect" + xstrminus3 + "_seed_" + seedstr + "}RC[-1]+2.92*Detect" + xstrminus2 + "_seed_" + seedstr + "}RC[-1]+4.096*Detect" + xstrminus1 + "_seed_" + seedstr + "}RC[-1]+6.216*Detect" + xstr + "_seed_" + seedstr + "}RC[-1]+4.3*Detect" + xstrplus1 + "_seed_" + seedstr + "}RC[-1]+3.122*Detect" + xstrplus2 + "_seed_" + seedstr + "}RC[-1]"
    Next y
    Next x

End Sub
LOW SPEED MODEL
Subroutine SimpleFormula

Sub SimpleFormula()

'Average Occupancy
    Range("H10").Select

'StDev of Volume
    Range("I10").Select
    ActiveCell.FormulaR1C1 = "=(STDEV(R[-9]C[-4]:R[1]C[-2]))/30"

'LogCVS
    Range("J10").Select
    Range("H10:M10").Select
    Selection.AutoFill Destination:=Range("H10:M360"), Type:=xlFillDefault

End Sub
Subroutine AdvanceFormula

Sub AdvancedFormula(seedstr As String)
' Macro1 Macro
' Macro recorded 9/16/2004 by jdilmore
',

Dim xstr As String
Dim x As Integer
Dim xstrminus1 As String
Dim xstrminus2 As String
' Dim xstrminus3 As String
' Dim xstrminus4 As String
Dim xstrplus1 As String
Dim xstrplus2 As String
Dim xstrplus3 As String
Dim xstrplus4 As String
Dim y As Integer
Dim ystr As String

For x = 33 To 64 Step 1
    xstr = x
    xstrminus2 = x - 2
    xstrminus1 = x - 1
    xstrplus1 = x - 1
    Sheets("Detect" + xstr + "_seed_" + seedstr).Select
    For y = 30 To 360
        ystr = y
        ystrminfive = y - 10
        ystrminten = y - 20
        Range("K" + ystr).Select
        Next y
    Next x
End Sub
HIGH SPEED MODEL
Subroutine SimpleFormula

Sub SimpleFormula()

' Macro1 Macro
' Macro recorded 9/10/2001 by APande

'Average Occupancy
Range("H10").Select

'StDev of Volume
Range("I10").Select
ActiveCell.FormulaR1C1 = "=(STDEV(R[-9]C[-4]:R[1]C[-2]))/30"

'Ave of Volume
Range("J10").Select
ActiveCell.FormulaR1C1 = "=(AVERAGE(R[-9]C[-5]:R[1]C[-3]))/30"

Range("H10:M10").Select
Selection.AutoFill Destination:=Range("H10:M360"), Type:=xlFillDefault

End Sub
Subroutine AdvanceFormula

Sub AdvancedFormula(seedstr As String)
' Macro1 Macro
' Macro recorded 9/16/2004 by jdilmore
',

Dim xstr As String
Dim x As Integer
Dim xstrminus1 As String
Dim xstrminus2 As String
Dim xstrplus1 As String
Dim xstrplus2 As String
Dim xstrplus3 As String
Dim xstrplus4 As String
Dim y As Integer
Dim ystr As String

For x = 33 To 64 Step 1
xstr = x
xstrminus2 = x - 2
xstrminus1 = x - 1
xstrplus1 = x - 1
Sheets("Detect" + xstr + "_seed_" + seedstr).Select
For y = 30 To 360
ystr = y
ystrminfive = y - 10
ystrminten = y - 20
Range("K" + ystr).Select
Next y
Next x

End Sub
APPENDIX F: RAMP AND DETECTOR LOCATIONS
<table>
<thead>
<tr>
<th>Ramp / Station</th>
<th>Mile Marker*</th>
<th>Ramp / Station</th>
<th>Mile Marker*</th>
<th>Ramp / Station</th>
<th>Mile Marker*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Orange Blossom Trl</td>
<td>19.5</td>
<td>Colonial Dr</td>
<td>23.0</td>
<td>Maitland Blvd</td>
<td>28.7</td>
</tr>
<tr>
<td>Orange Blossom Trl</td>
<td>19.5</td>
<td>Station 41</td>
<td>23.1</td>
<td>Station 53</td>
<td>28.9</td>
</tr>
<tr>
<td>Orange Blossom Trl</td>
<td>19.7</td>
<td>Ivanhoe Rd</td>
<td>23.3</td>
<td>Maitland Blvd</td>
<td>29.0</td>
</tr>
<tr>
<td>Station 33</td>
<td>19.7</td>
<td>Ivanhoe Rd</td>
<td>23.4</td>
<td>Maitland Blvd</td>
<td>29.1</td>
</tr>
<tr>
<td>Orange Blossom Trl</td>
<td>19.7</td>
<td>Ivanhoe Rd</td>
<td>23.6</td>
<td>Station 54</td>
<td>29.4</td>
</tr>
<tr>
<td>Orange Blossom Trl</td>
<td>19.9</td>
<td>Station 42</td>
<td>23.6</td>
<td>Station 55</td>
<td>29.9</td>
</tr>
<tr>
<td>Station 34</td>
<td>20.2</td>
<td>Ivanhoe Rd</td>
<td>23.7</td>
<td>Station 56</td>
<td>30.5</td>
</tr>
<tr>
<td>Michigan Str</td>
<td>20.4</td>
<td>Station 43</td>
<td>24.1</td>
<td>SR 436</td>
<td>30.7</td>
</tr>
<tr>
<td>Michigan Str</td>
<td>20.4</td>
<td>Princeton Str</td>
<td>24.2</td>
<td>SR 436</td>
<td>30.7</td>
</tr>
<tr>
<td>Kaley Ave</td>
<td>20.5</td>
<td>Princeton Str</td>
<td>24.2</td>
<td>Station 57</td>
<td>30.9</td>
</tr>
<tr>
<td>Kaley Ave</td>
<td>20.5</td>
<td>Princeton Str</td>
<td>24.5</td>
<td>SR 436</td>
<td>31.1</td>
</tr>
<tr>
<td>Kaley Ave</td>
<td>20.6</td>
<td>Princeton Str</td>
<td>24.5</td>
<td>SR 436</td>
<td>31.1</td>
</tr>
<tr>
<td>Station 35</td>
<td>20.7</td>
<td>Station 44</td>
<td>24.6</td>
<td>Station 58</td>
<td>31.7</td>
</tr>
<tr>
<td>Kaley Ave</td>
<td>20.8</td>
<td>Par Ave</td>
<td>25.0</td>
<td>Station 59</td>
<td>32.2</td>
</tr>
<tr>
<td>Kaley Ave</td>
<td>20.9</td>
<td>Par Ave</td>
<td>25.0</td>
<td>SR 434</td>
<td>32.6</td>
</tr>
<tr>
<td>E-W Expressway</td>
<td>21.1</td>
<td>Station 45</td>
<td>25.1</td>
<td>SR 434</td>
<td>32.6</td>
</tr>
<tr>
<td>E-W Expressway</td>
<td>21.2</td>
<td>Station 46</td>
<td>25.7</td>
<td>Station 60</td>
<td>32.8</td>
</tr>
<tr>
<td>Station 36</td>
<td>21.2</td>
<td>Fairbanks Ave</td>
<td>25.8</td>
<td>SR 434</td>
<td>33.1</td>
</tr>
<tr>
<td>E-W Expressway</td>
<td>21.3</td>
<td>Fairbanks Ave</td>
<td>25.9</td>
<td>SR 434</td>
<td>33.1</td>
</tr>
<tr>
<td>E-W Expressway</td>
<td>21.4</td>
<td>Station 47</td>
<td>26.1</td>
<td>Station 61</td>
<td>33.3</td>
</tr>
<tr>
<td>Gore Str</td>
<td>21.5</td>
<td>Fairbanks Ave</td>
<td>26.1</td>
<td>Station 62</td>
<td>33.9</td>
</tr>
<tr>
<td>Anderson Str</td>
<td>21.7</td>
<td>Fairbanks Ave</td>
<td>26.2</td>
<td>Station 63</td>
<td>34.4</td>
</tr>
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*Mile Marker indicates the distance from the Orange County Osceola County border, which is Orange Counties southern limit. This point corresponds to the western terminus of I-4 in Orange County.
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