Evaluation of the Operational and Safety Effects

of the I-66 Active Traffic Management System

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ABSTRACT

A project to install an Active Traffic Management (ATM) system on Virginia Interstate 66 (I-66) from US 29 in Centreville to the Capital Beltway (I-495) began early 2013 and was completed in September 2015. This project installed smart infrastructure and employed dynamic operations techniques that mobilized ATM, which were intended to improve safety and operations on I-66 without physically expanding the roadway. The main components of ATM that were installed on I-66 include advisory Variable Speed Limits (VSL), Queue Warning Systems (QWS), Lane Use Control signs (LUCS) and Hard Shoulder Running (HSR). ATM has been successful in producing safety and operational improvements in many European countries, but there are limited ATM applications in the U.S. (Mirshahi et al., 2007; Fontaine and Miller, 2012). Since ATM is still a relatively new approach in the U.S., there was a need to analyze the safety and operational effects of ATM on I-66.

In this thesis, appropriate operational and safety measures of effectiveness (MOE) were defined, examined, and analyzed in order to conduct a before-and-after study to quantify the effectiveness of the ATM system on I-66. The operational MOEs included ATM utilization rate, average travel time, travel time reliability, and total travel time delay levels. The safety MOEs included crash rates by type and severity and a safety surrogate (speed drop events) for crashes. These MOEs were analyzed by using INRIX travel time data, limited traffic volume point sensor data and Virginia Police Crash Reports.

The results indicate that the ATM produced positive operational and safety benefits in several MOEs analyzed in this report. The analyzed operational and safety benefits from implementing ATM on I-66 were similar to the reported operational and safety benefits of ATM implementations in Europe and other states in the United States. The research found that ATM

generally had a limited operational impacts during the weekday peak periods on I-66, but some benefits were observed during the off peak weekday periods. Average weekday travel times during the middle of the day and in the off-peak direction typically improved by 2 to 6 percent. Large benefits were observed on the weekends, with average travel times improving by about 10 percent during the day. All of these differences were statistically significant. Travel time reliability improved by similar margins. Weekday peak period travel times and reliability continued to degrade following ATM installation, however. This was not surprising given that HSR was already in use during the peak weekday periods and there has been a historic trend towards increasing travel times on the corridor. Estimates of safety impact based on limited empirical data and safety surrogate analysis showed a 10 percent improvement in crashes during the weekdays and up to a 50 percent improvement on weekends. Those safety findings are preliminary, however.

High-level segment analysis was performed to determine the segments that benefitted the most from the implementation of the ATM. From this analysis, it was found that the HSR was the component of the ATM that produced the most operations and safety improvements on I-66. In terms of HSR utilization rate, HSR was being opened for a longer period of time after the implementation of ATM. On weekdays, the shoulders were open for an extra 2.5 hours/day and on weekends, the shoulders were open for an extra 4.5 hours/day for both directions combined. Before HSR, the shoulder opening hours were static during weekdays limited to peak periods and were not open during weekends.

A planning level benefit-cost ratio was calculated based on the operational and safety benefits. The ATM project had a benefit-cost ratio of 5.29, and its value was calculated by monetizing weekend operations and safety improvements on I-66. The high benefit-cost ratio shows that the ATM was a cost-efficient solution in improving operations and safety on the I-66 corridor. The thesis concludes with recommendations for additional expansion of ATM in Virginia and future research.

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CHAPTER 1: INTRODUCTION

According to the annual Urban Mobility Scorecard prepared by the Texas A&M Transportation Institute, the Washington metropolitan area is consistently ranked as the location with the worst traffic congestion in terms of delay, reliability, and fuel consumption in the United States (Schrank et al., 2015). Virginia's I-66 is a highly congested highway during both peak and off-peak hours with unpredictable traffic patterns, as it is the only interstate running east to west in the region.

A project to install an Active Traffic Management (ATM) system on the I-66 corridor from Centreville (Exit 52/US 29) to the Capital Beltway (Exit 64/I 495) officially began in early 2013 and was completed on September 2015. TransCore and Parson Brinkerhoff were the selected design-build contractors, and the approximate cost for this project was $38.6 million. The main goals of installing the ATM system on I-66 were to improve operations, roadway safety, and incident management. The project installed ATM system infrastructure including gantries with ATM signs, shoulder and lane use control signs, variable speed limit displays, emergency pull-outs, and increased coverage of traffic cameras and sensors. Gantries were spaced approximately 0.5 miles apart from each other so that continuous information can be provided to the drivers traveling on I-66 (Iteris, 2011).

1.1 I-66 Active Traffic Management System

Active Traffic Management components are defined as techniques that dynamically manage recurrent and non-recurrent congestions based on prevailing traffic conditions, optimizing capacity of the corridor and improving safety (Mirshashi et al., 2007). Some of the main ATM components implemented on I-66 include:

- Variable speed limits (VSL)
  - VSLs dynamically change the posted speed based on current traffic or roadway conditions. Variable speed limits, sometimes termed speed harmonization, encourage more uniform speed distributions that can improve traffic operations and safety by providing guidance based on real time information. They also can provide advance warning of slowed traffic ahead. For the I-66 ATM project, all VSLs are posted on signs above each lane and are advisory.
- Queue warning systems (QWS)
  - QWSs provide advanced notice to drivers of the cause of congested roadway conditions ahead on variable message signs and work in conjunction with VSLs to provide notice of slow or stopped traffic ahead. This advance notice has been found to reduce secondary crashes in other studies (Middelham, 2006).
- Hard shoulder running (HSR)
  - Prior to ATM activation, the shoulder lane on I-66 was open to travel during certain predefined peak periods. Following ATM activation, the HSR system dynamically opened or closed the shoulder lanes depending on roadway conditions, increasing capacity on I-66. A shoulder lane monitoring system (SLMS) was also installed. The SLMS uses video analytics to monitor blockages on the hard shoulder in order to facilitate quick opening or closing of the shoulder while protecting disabled motorists temporarily stopped on the shoulder. Before the ATM implementation, the hard shoulder operation hours were static from 5:30 to 11:00 AM eastbound, and 2:00 to 8:00 PM westbound.

- Lane Use Control Signs (LUCS)
  - Overhead gantries were deployed with lane use control signs to alert drivers to lane blockages. This was used for incident and work zone management.

The example below shows how ATM manages incident situations. The progression of ATM signs around an incident is shown on Figures 1-3. Figure 1 shows the shoulder lane being closed down due to a stopped vehicle using the shoulder lane as an emergency stopping lane. It should be noted that the yellow arrow sign posted by the safety patrol highwayman is not part of the ATM. The LUCS component of the ATM is shown directly on the electronic message signs on the gantry itself.

This non-recurrent congestion event caused the ATM to redirect all traffic to the open lanes by posting a red “X” on the rightmost part of the gantry signs as the shoulder lane is located on the rightmost lane of I-66.

Figure 1: Closed shoulder lane due to stopped and emergency vehicles

In response to the accident upstream, Figure 2 shows the VSL and QWS in action. The advisory roadway speeds posted on the gantry shows the optimal speeds the vehicles should travel in order to efficiently manage the flow of vehicles on the roadway. This reduction of speed may be due to roadway congestion or an incident upstream of the posted location, and the dynamic message sign displays messages that advise the drivers of the road conditions ahead.

Figure 2: Advisory speed limit in place (55 mph -> 50 mph)

Once the road conditions return back to normal, the signs on the gantry show the regulatory speed limit and the display board indicates to resume normal speed as well, which is shown on Figure 3 below.

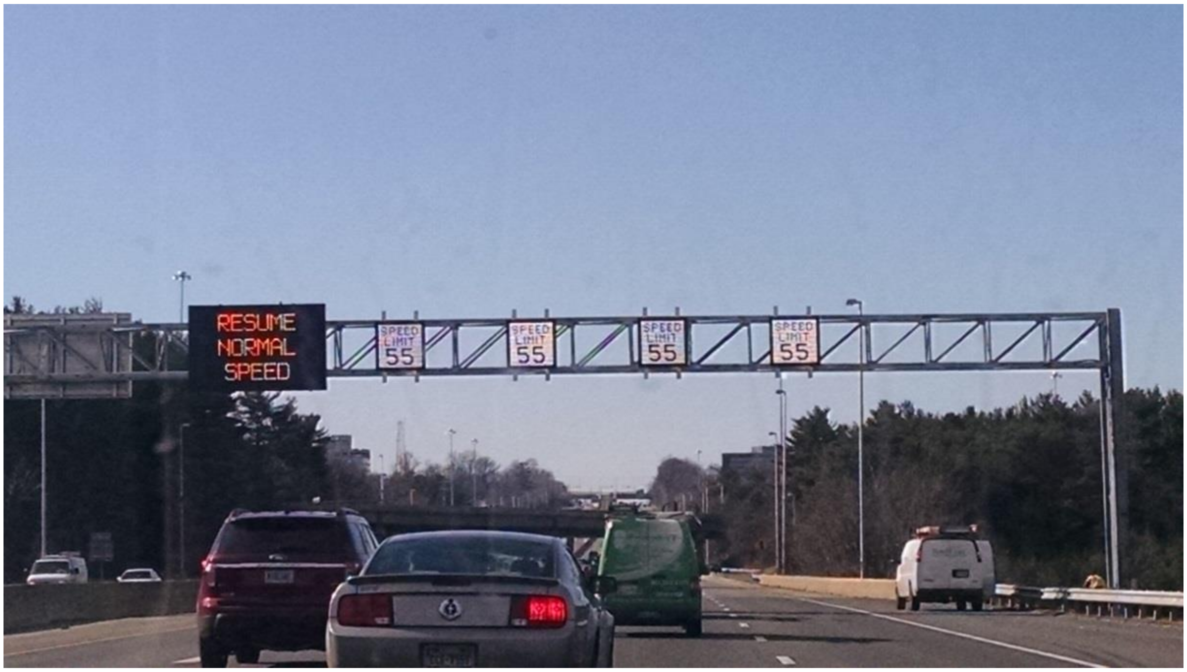


Figure 3: Returning back to normal speed

During recurrent congestion events, such as peak hour commute traffic, ATM actively increases the roadway capacity by dynamically turning on the HSR, VSL, LUCS and/or QWS whenever necessary. I-66 is a heavily congested highway during both peak and off-peak hours with unpredictable traffic patterns. However, the recurring congestion during AM and PM peak periods on I-66 can be somewhat predictable. Many of the ATM techniques such as VSL, QWS, LUCS and HSR are best applied to roadways with significant recurring congestion (Tucker et al. 2005; Fuhs 2010). Ideally, ATM on I-66 will improve flow of traffic and reduce crashes during the recurrent congestion periods as well as help improve management of non-recurrent events.

In Europe and the United States, ATM has been successful in producing safety and operational improvements on the roadway by improving safety and operational measures such as crash rates, crash severity, throughput, and travel times. Europe has had more experience with ATM implementation whereas ATM is a relatively new technology that has been just adopted in recent years in the United States. In both the United States and Europe, ATM projects tend to be implemented in urban areas where recurrent congestion is prevalent and right of way is constrained. Many of the operating characteristics of European deployments differ from those in the United States, however, and may limit the transferability of European results. For example, many European deployments use automated speed enforcement in conjunction with VSLs, which is not possible in most jurisdictions in the United States. Given the lack of data on U.S. applications of ATM, there is a need to monitor the effects of the I-66 ATM project.

1.2 Purpose and Scope

The purpose of this study is to quantify improvements in traffic operations and safety produced by the I-66 ATM system. Specific objectives include:

- Determine the utilization rate of the ATM on I-66 to identify the frequency and spatial distribution of the use of various techniques.
- Determine whether the ATM system improved average travel time and/or travel time reliability
- Assess whether the ATM system improved total traveler delay
- Determine if the ATM system improved crash characteristics, such as frequency, type, severity, and/or rate.

The scope of this study will be confined to I-66 between US 29 (Exit 52) and I-495 (Exit 64), where the most ATM components are implemented on the I-66 corridor. The thesis will focus on the macroscopic performance of the corridor, and will assess whether overall corridor-level operations and safety levels have been improved by implementing the ATM. Since the ATM system was not activated until September 2015, this thesis covers its performance from this date through the end of February 2016.

1.3 Thesis Organization

The thesis is organized into the following chapters:

- Chapter 1: INTRODUCTION
- Chapter 2: LITERATURE REVIEW
- Chapter 3: VIRGINIA I-66 ROADWAY AND ATM CHARACTERISTICS
- Chapter 4: METHODOLOGY
- Chapter 5: RESULTS
- Chapter 6: CONCLUSIONS AND RECOMMENDATIONS

CHAPTER 2: LITERATURE REVIEW

ATM has been successful in producing positive operational and safety results in many European countries, but applications in the United States are limited (Fontaine and Miller, 2012). A scan team from the Federal Highway Administration (FHWA) and the American Association of State Highway and Transportation Officials (AASHTO) visited and examined the implementation and impact of ATM in key European countries, and the scan team's conclusion from the visit was that ATM can be used to improve safety and operations in the U.S. (Mirshahi et al., 2007). Since the scan team's visit, there have been more ATM implementations, but ATM is still in its introductory stages in the U.S. In many of the U.S. implementations, preliminary evaluations of ATM have been conducted, but detailed impact analyses are limited because of limited availability of data or presence of systematic problems (Atkins Consulting, 2009 and Jacobson, 2012). Because ATM is a relatively new technology in the U.S., and the effects of ATM in Europe have been very positive, further research regarding the effects of ATM in the U.S. is necessary.

It is entirely possible that the reported European ATM implementation benefits may not be transferable to the United States, given differing operating characteristics and driver behavior. For example, many European ATM deployments incorporate automated speed enforcement, which is not legally available in most U.S. jurisdictions. Therefore, a review of past implementations and analyses was important as it shed light on the respective operating characteristics on each of the roadway where ATM was installed and analyzed. It was also important to scrutinize the methods each researcher used to design their analysis as different methodologies will result in different results. In this review, results based on field data were emphasized and analyzed since driver response to ATM is a key factor in actual field observed impacts. Results based on field data may only be relevant to the network from which they were derived, making it more important to analyze and compare the operating characteristics of the roadways (Fudala and Fontaine, 2009).

2.1 Active Traffic Management Field Studies

In many of the ATM implementation cases, implementing at least one of these techniques was considered as implementing ATM. However, these techniques are very complementary to each other and are very often installed in a synergistic manner (Fuhs, 2010). On I-66, as all of these mentioned techniques are being deployed simultaneously, the before-and-after analysis showed the effect of the combination of all ATM techniques, and not just the effect of one individual ATM technique. But as mentioned above, a review of past implementations and analyses was important as it would note the differences of operating characteristics and research methodology between the organizations. Past field deployments of ATM in Europe and the United States were briefly reviewed in this section.

2.1.1 Europe – Germany (Case of VSL Implementation)

VSL has been used in Germany to improve traffic safety and mobility since the 1970s (Fuhs, 2010). One of the motorways where Germany had implemented VSL is the A5 autobahn, which has an approximate ADT of 150,000 vehicles per day (Sparmann, 2006). The VSL implementation on the A5 autobahn between Bad Homburg and Frankfurt/West proved to be effective in improving safety, reducing accident rates by 20% when, on a comparable section of autobahn without VSL, crashes increased by 10% in the same time period (Tignor et al., 1999).

The original research that evaluated this result was not translated into English, which makes it difficult to validate these values. However, a different VSL study that evaluated the same corridor reported a comparable 27% reduction in crashes with heavy material damage and a 29% reduction in crashes with personal damage (Sparmann, 2006). Crashes with heavy material damage showed a decrease from 0.41 crashes per million vehicle-kilometer (mvkm) to 0.30 crashes per mvkm, and crashes with personal damage showed a decrease from 0.21 crashes per mvkm to 0.15 crashes per mvkm (Sparmann, 2006). However, the researchers did not mention any of the methodology they used to calculate these values. Information such as the number of years used to calculate the crash rates, the database used for analysis, the threshold for classifying crashes with light material damage from crashes with heavy material damage, and others were not identified, which are all crucial in understanding the validity of the before and after study. Also, the researchers did not mention any statistical analysis that they have conducted to determine the statistical significance of the changes.

VSLs also proved to be effective in improving traffic operations as well. An empirical study of the traffic flow effects of VSL on a 16.3km (~10 mi), three lane German A99 autobahn found that lane utilization was distributed more evenly amongst the lanes with the implementation of VSL, but at the slight cost of capacity (Weikl, 2013). This research analyzed a total of 25 weekdays with the VSL in operation and only 6 weekdays without the VSL in operation using real-time traffic information from the VSL system and the 14 dual-loop detector system (Weikl, 2013). The researchers used the bottleneck locations and activation and deactivation times to come up with a spatial-temporal analysis of the queued region, and then analyzed the difference between uncongested period and the queue discharge period of each lane, which can be described as flow recovery rate. A total of 18 bottleneck cases, 8 cases when VSL was activated and 10 cases when VSL was inactive were identified and studied using this method. It was found that when VSL was active, overall flow during the uncongested period was 4700 vehicles per hour (vph) while overall flow during the queue discharge period was 4500 vph, which resulted in a negative change of flow of approximately 4%. When VSL was inactive, the overall flow during the uncongested period was 4830 vph while the overall flow during the queue discharge period was 4690 vph, which resulted in a negative change of flow of approximately 3% (Weikl, 2013). The flow recovery rate when VSL was inactive was better than the flow recovery rate when VSL was operational, but the uniformity of reductions across the lanes showed otherwise. When VSL was active, the lane flow distribution for median, center, and right lanes were 38%, 37%, and 25% respectively. When VSL was inactive, the lane flow distribution for median, center, and right lanes were 41%, 33%, and 26% respectively. This result shows that VSL improved flow homogeneity on the bottleneck locations (Weikl, 2013). However, the researchers did not perform any statistical analysis that determines the statistical significance of the changes.

2.1.2 Europe – Germany (Case of HSR Implementation)

On the A5 autobahn in Hessen, Germany, HSR is in operation at times of extreme traffic volumes only. HSR is only available only on the Hessen, Germany region of A5 autobahn, and not in the region with VSLs discussed earlier (Sparmann, 2006). Preliminary research showed that maximum traffic flow increased from 5600 vph when HSR is inactive to 7000 vph when HSR is active on A5 autobahn (Sparmann, 2006). HSR showed similar improvements on the A4 motorway, where its average capacity increased by 20% from 4300 vph to 5200 vph after HSR implementation (Kellermann, 2000).

A more detailed study analyzing the operational effect of dynamic HSR in Germany showed that capacities of the A5 and A3 motorways increased by 20% to 25% when hard shoulders were activated during congested hours (Geistefeldt, 2012). The A5 motorway, which has very similar characteristics as A3 motorway, is an 18 km (~11 mi) motorway with three lanes in both southbound and northbound directions, high commuter traffic, and distinct peak volumes (Geistefeldt, 2012). Using 40 months of loop detector data that were collected after the implementation of HSR, capacity distribution functions were estimated for a total of four sections of the A3 and A5 motorways (Geistefeldt, 2012). In three of the four sections, the median values of the capacity were 20% to 25% higher than the capacity of comparable sections without HSR while in the remaining section was only 10% higher (Geistefeldt, 2012). The researcher did not mention the baseline capacity of the comparable section without HSR nor discuss which roadway the researcher was comparing the observed values with. The same researcher also analyzed the effects of HSR on duration of congestion on the same A5 freeway. A before-and-after analysis on the 47 sections of the motorway using 2002 and 2006 data was conducted. Total duration of congestion per year was calculated for each section by determining the number of 5 minute intervals with an average passenger car speed below a threshold speed of 70 km/h (~44 mph) (Geistefeldt, 2012). This analysis showed that the maximum duration of congestion was reduced from 640 hours per year for northbound and 450 hours per year for southbound to less than 200 hours per year in both directions (Geistefeldt, 2012).

There are many claims and counterclaims that the HSR provides improvements on roadway safety. Previous investigations in Germany have shown that crash rate on motorway sections without hard shoulders available for disabled vehicles are approximately 25% higher than those with hard shoulders available for disabled vehicles (Kellermann, 2000). This means that when hard shoulders are converted into general purpose lanes during heavily congested periods, crash rate may increase as well. However, several studies refute the claim that loss of hard shoulders leads to decrease in safety. A before-and-after safety study conducted on sections of A7 motorway showed that implementation of HSR did not necessarily cause increase in crashes on the motorway (Lemke 2010). The HSR effect on A7 motorway sections at Quickborn (10 km or ~6 mi), Neumünster (14 km or ~9 mi), and Kaltenkirchen (12 km or ~7 mi) were studied for this research, and these motorways are characterized by commuter traffic and an AADT of up to 35,000 vehicles per day (Lemke, 2010). The researcher analyzed 3 years of before-HSR and 3 years of after-HSR data for this research, and the findings show that on Quickborn, slight injury and severe property damage only (PDO) accident rate has slightly decreased from 0.09 crashes per mvkm to 0.07 crashes per mvkm after the implementation of HSR. On Kaltenkirchen, slight injury and severe PDO accident rate had increased slightly from 0.08 crashes per mvkm before HSR implementation to 0.10 crashes per mvkm after the implementation of HSR. On Neumünster, slight injury and severe PDO accident rate had decreased from 0.20 crashes per mvkm before HSR implementation to 0.13 crashes per mvkm after the implementation of HSR (Lemke, 2010). The researcher used police written reports to come up with the rate values, and no statistical analysis was performed to see if the differences were of any statistical significance. However, the results show that implementing HSR did not necessarily increase accident rates. This may have been due to the effective HSR implementation guidelines set by Germany, such as installing emergency refuge areas every 1000 meters and making the hard shoulder lanes wide enough for safe vehicle travel with the implementation of HSR (Lemke, 2010).

2.1.3 Europe – United Kingdom (Case of VSL and HSR Implementation)

The United Kingdom has also adopted ATM to innovatively improve safety and operations on its roadways. The ATM had been installed on the M42 motorway, and M42 is approximately 17 km (~11 mi) long, has 3 lanes in each direction, and has an AADT of 134,000 vehicles per day (Mott McDonald, 2008). The ATM techniques used on M42 include VSL and HSR. Activation of VSL in the three lane sections is dynamic based upon flow and speed thresholds, though operators may adjust the operation if required. The HSR is also based on predefined flow and speed thresholds. However, HSR activation is not automatic, since operators need to ensure that the hard shoulder running lane is not blocked by debris or stopped vehicles. When activated, the speed limit on the hard shoulder is 50 mph or less (Mott McDonald, 2008). Enforcement of VSL in the United Kingdom is automated, and the compliance of speed limit is very high. Compliance rates at 50, 60, and 70 mph were 94% and the compliance rate at 40 mph was 84% for the study scope (Mott McDonald, 2008). The high compliance rates may have been the result of using the Association of Chief Police Officer threshold for enforcement threshold, defined as Threshold = (1.1 x Speed Limit) + 2, to calculate compliance rate. Using this threshold means that even if a vehicle is going 79 mph on a 70 mph zone or 68 mph on a 60 mph zone, it is still considered as being compliant to the speed limit.

The construction on M42 started in 2003 and the VSL portion of the project was completed in late November 2005. The final HSR portion of the project was completed in September 2006. Data for No-ATM conditions were collected from March 2002 to February 2003 for a total of 12 months, and data for full ATM conditions were collected from October 2006 to September 2007 for a total of 12 months. September 2006 was excluded from the analysis as it was considered as settling in period for the ATM (Mott McDonald, 2008). The researcher mentions that since additional development and construction work between the ATM construction phases such as the development of a new airport near M42, the benefits of ATM were likely to be underestimated given the large increase of traffic between the analysis periods (Mott McDonald, 2008). The database used for analysis included the ATM system management database, loop detector data, police crash data, and weather data. All of the data were pre-screened to sort out outliers before they were used for any analysis. The analysis was broken down into weekday and weekend AM peak, inter-peak, and PM peak hours. The findings of this study reported that after the implementation of ATM, capacity of the motorway increased by an average of 7%, and 24-hour total flow increased by 6% and 9% on northbound and southbound directions respectively compared to No-ATM conditions (Mott McDonald, 2008). Using the TRL Journey Time Algorithm program, travel times were calculated on M42. The results showed that average travel times have increased by 9% after the implementation of ATM, and the researchers attribute this to the increase in volumes and VSL with high compliance rates. However, the variability of travel time has been reduced by 22% on both directions, allowing drivers more able to predict their overall travel times as differences between worst and best cases is reduced (Mott McDonald, 2008). The statistical t-test analysis confirmed that the results were found to be significantly significant at the 5% level (Mott McDonald, 2008).

Preliminary safety analysis was also conducted using five years of before-ATM and one year of after-ATM Personal Injury Accident (PIA) data. Measures of effectiveness used to compare the before and after values were average number of crashes per month and severity index. The average number of crashes per month was reduced from 5.08 crashes per month for No-ATM conditions to 1.83 crashes per month for ATM conditions. Severity index, which is the ratio of number of fatal and serious crashes to the total number of crashes, was 0.16 for No-ATM conditions and for 0.14 for ATM conditions (Mott McDonald, 2008). To produce more reliable conclusions, the researchers recommended longer period of data to be collected before a true before and after safety conclusion is made (Mott McDonald, 2008). Two years later, the same researchers completed a final safety study on the same corridor and found similar results. The total number of PIAs during the first 36 months of ATM operation has decreased compared to equivalent time periods during No-ATM operation. The proportion of rear-end collision crashes remained constant, but the proportion of side impact collision crashes increased from 16.1% during No-ATM conditions to 30.9% during ATM conditions on both directions. The researchers hypothesized that there were no increase in the rear-end collisions due to the increase in the length of headways, and there were increases in side impact collisions because of the opening of the new shoulder lanes, which created more lane changing maneuvers and vehicle-to-vehicle interactions (Mott McDonald, 2011). During the first 3 years of ATM operation, there were 2.25 average crashes per month for ATM conditions while there were 5.08 average crashes per month for No-ATM conditions. The severity index was 0.07 for ATM conditions compared to 0.16 for No-ATM conditions. The monthly mean number of killed or serious injured casualties had fallen from 1.15 during No-ATM conditions to 0.19 during ATM conditions as well. Finally, the two-way accident rate per billion vehicle miles traveled (bvmt) for ATM conditions were 47.98 crashes per bvmt, which was a large reduction from the 115.92 crashes per bvmt during No-ATM conditions (Mott McDonald, 2011). The conclusion on the safety analysis was that the implementation of ATM improved crash frequency, crash severity and crash rate on the M42 motorway.

2.1.4 Europe – Netherlands (Case of VSL, HSR and QWS Implementation)

VSL has been used in the Netherlands since 1970s for the purposes of creating more uniform traffic flow and managing traffic during adverse weather conditions (Fuhs, 2010). A safety assessment conducted in 1983 showed that QWS in combination with VSL improved throughput by 4-5%, and reduced primary crashes by 15-25% and secondary crashes by 40-50%. A subsequent safety assessment in 1996 confirmed the earlier results (Middelham, 2006). Also, implementing temporary shoulder use had an overall capacity increase of 7-22% by decreasing trip travel times from 1-3 minutes and increasing traffic volumes through the area up to 7% during congested periods (Taale, 2006). The methodology used to come up with these findings was not reported possibly due to the fact that the full documents were not translated into English.

2.1.5 United States – Washington (Case of VSL and QWS Implementation)

The Washington State Department of Transportation (WSDOT) has implemented ATM including VSL and QWS techniques on NB I-5 (7 miles, in operation since Aug 2010), EB and WB SR-520 (8 miles, in operation since Nov 2010), and EB and WB I-90 (9 miles, in operation since Jun 2011). However, the 6 month time lag between when a crash actually occurred when it was entered into the DOT crash database and lack of data availability resulted in no definitive assessments of the safety impact of the system as of early 2012 (Jacobson, 2012). It is important to note that the researchers considered 3 months of settling out period for future analysis.

A travel time reliability analysis had been conducted for 8 months before and after period to analyze the ATM effect on travel time reliability on a 7 mile northbound corridor of I-5 in Washington. Planning time index (PTI) and buffer index (BI) were calculated using 5 minute time intervals on each day of the week, time of day, weekday, and weekends. There were 19 loop detectors that were used for data collection, and only detector data were used for data analysis (DeGaspari, 2012). The researchers do mention that the equation they are using to calculate speed is a space-mean speed equation. However, it is important to note that loop detectors are usually used to gather time-mean speed, which does not truly represent the average speed of the vehicles on the entire roadway. Therefore, a few key assumptions, such as estimating the long vehicle percentage, were used to calculate the average speeds. For the PTI calculation, the researchers assumed free flow speed to be 60 mph, which is the speed limit for the study location (DeGaspari et al., 2012). The final results were that on weekends, PTI was improved by 17% and BI was improved by 27% after the implementation of the ATM. On Tues-Thurs, PTI improved by 12-17% and BI improved by 15-20%. Finally, on Mon and Fri, there were minor positive effects (DeGaspari et al., 2012). The researchers report that the improvements were greatest during off-peak hours during the weekdays (DeGaspari et al., 2012). The researchers have conducted a paired t-test and found all changes of PTI and BI to be significant at the 0.05 level (DeGaspari et al., 2012).

2.1.6 United States – Florida (Case of VSL Implementation)

A VSL system had been implemented on I-4 in Florida since Sept 15, 2008. The I-4 corridor is approximately 10 miles long, and has an AADT of more than 200,000 (Atkins Consulting, 2009). The Florida I-4 VSL system was evaluated by looking at data from 4PM to 6PM for 21 days before VSL activation as compared to 1 month after VSL activation (Atkins Consulting, 2009). The before data period was from Jan 1, 2008 to Jan 21, 2008 and the after data period was from Sept 15, 2008 to Oct 14, 2008 (Atkins Consulting, 2009). Analyzing only a month's worth of before and after data for analysis is problematic as it will be hard to prove the validity of the results. Also, only a 2 hour period was studied with an assumption that 4PM to 6PM was the most important time of the day as it is the PM peak hour. The data showed that speed changes were more strongly correlated with changes in occupancy than changes in the posted speed limit. The conclusion was that the VSL had no significant impact on speed compliance or mean travel speed. A simple safety impact study was also conducted but no conclusions could be drawn because of limited data (Atkins Consulting, 2009).

2.1.7 United States – Missouri (Case of VSL Implementation)

In May 2008, total of 65 VSL signs were installed along 38 miles of I-260 and I-255 in St. Louis, Missouri (Kianfar et al., 2010). Using inductive loop and acoustic detectors, the researchers analyzed the effect of VSL on three major bottlenecks along the corridor. Data from Missouri deployment were evaluated using 150 days of before the VSL deployment and 150 days of after the VSL deployment, excluding the ten days of post-VSL data for driver normalization (Kianfar et al., 2010). Using both parametric and non-parametric statistical analysis, the researchers found that pre-queue flow (free flow) decreased by up to 4.5% and queue discharge flow (congested flow) decreased by up to 7.7% on the three bottleneck locations. Average speeds fluctuated, but speed variance declined at all bottleneck locations (Kianfar et al., 2010). Because this study was conducted for bottleneck locations only, the findings of this research may not represent the effects of VSL on the entire roadway.

2.1.8 United States – Minnesota (Case of VSL Implementation)

VSL was activated in late July 2010 on the I-35W and I-94 corridors located in Minneapolis/St. Paul, Minnesota. The researchers used single loop detector and speed sign activation records to analyze the operational effects of VSL system on I-35W. The researchers mentioned the limitation of using data collected from loop detectors, which may be critical in analyzing the operational effects of VSL. Since loop detectors are location specific, the length of the entire corridor is represented by extrapolating specific point locations only. Peak periods for weekdays and weekends were analyzed using 2 years' worth of data from Nov 2009 to Dec 2011. One of the performance measures for operational analysis was congestion rate, calculated by counting the number of certain speed drop thresholds (i.e. speed drop below 10mph, 15mph, …, 45 mph) for each 30 second segment counts divided by the total number of total possible segment counts. The other performance measure was to analyze the volume-occupancy trends on heaviest traffic locations. Generally, the volume-occupancy trends showed that during the onset of congestion, the after VSL implementation conditions showed much more gradual decreases in speeds. This meant that the impact of traffic shockwave is reduced and mean speeds increased. Analyzing congestion rate, northbound AM peak period experienced an average of over 17% less congestion with the VSL implementation for speed drop of 25 mph or more. The entire VSL region had 7.6 minutes less congestion during the average AM peak. The researchers mention that the southbound PM peaks are less distinct and the VSL had less impact than that of the northbound AM peak. No statistical significance testing was conducted in this research (Hourdos et al., 2013).

The same research group also conducted a VSL safety study on problematic crash areas of I-94. I-94 is a highway that intersects and joins I-35W, and has AADT over 160,000. Before and after analysis was conducted to examine crash rates for crashes and near-crashes using visually identified events within video data and Minnesota State Patrol crash records. Video footage from the problematic locations recorded between 10AM and 9PM during all weekdays were analyzed for before period (April 2012 to late-Sept 2012) and after period (late-Sept 2012 to fall of 2013). Crashes and near-crashes were recorded for each hour and averaged across the days for the before and after periods. In total, the crash rates dropped slightly from approximately 116 to 107 crashes per 100 million VMT. However, when adjusted for non-winter months, the crash rate increased from 129 to 132 crashes per 100 million VMT. No statistical significance testing was conducted in this research. The researchers concluded that there were no significant changes in safety along the corridor due to the VSL system (Hourdos and Zitzow, 2014).

Different researchers analyzed different performance measures to evaluate the operational effects of VSL on the same I-35W corridor (Kwon and Park, 2015). The researchers mention that traffic detector data were used. Average maximum deceleration and travel time buffer index were analyzed for before and after data for the AM peak period (7-8AM) for September-November of 2009, 2010, and 2011. September-November of 2009 was considered as the before-VSL implementation period, and September-November of 2010 and 2011 were considered as the after-VSL implementation period. Average maximum deceleration was determined to be the average of the maximum 1 minute decelerations between two detector stations for a given hour. The average maximum deceleration decreased by 22% when 2009 conditions were compared to that of 2010, and the difference was statistically significant at the 95% confidence level. Also, the average maximum deceleration decreased by 10% when 2009 conditions were compared to that of 2011, and the difference was statistically significant at the

95% confidence level. Travel time buffer index improved by 24-32% after VSL activation, which was also significant at the 95% confidence level. The average buffer index values were 0.25 in 2009, 0.19 in 2010 and 0.17 in 2011. April-June in 2010, 2011 and 2012 were also analyzed, and while average maximum deceleration changes were not statistically significant, the travel time buffer index showed improvements of 17-25% at the 95% significance level. The average buffer index values were 0.25 in 2010, 0.20 in 2011, and 0.18 in 2012 (Kwon and Park, 2015). The researched showed that the implementation of VSL produced improvements in travel time reliability on this corridor.

2.2 Literature Review Summary

From the literature review, it is evident that ATM could have operational and safety benefits. In Europe, the evaluations showed that travel times, traffic flow, crash rates, and crash severity were improved by implementation of one or more ATM technique. In the United States, while the implementation of ATM is in its beginning stages, it showed potential in improving operations and safety on the corridor. Tables 1-3 below show the summary of the major European and American implementation of ATM.

Many of the research regarding ATM techniques were conducted by using detector data, which shows traffic information on a specific point on a roadway. In this thesis, INRIX real-time probe-based travel time data will be used to analyze operational effects of ATM. By using real-time probe-based data, the travel time conditions of the corridor were better represented as the measured travel times and speeds were for the length of the entire corridor (space-mean-speeds). Also in Virginia, the implementation of ATM consisted of HSR, VSL, LUCS and QWS techniques. The ATM system on I-66 is a very comprehensive implementation on a unique corridor as Virginia I-66 has HOV lanes, shoulder lanes, and even metro rail running in the middle of the corridor. As a result, it was initially unclear how well past results would translate to the I-66 installation.

Table 1. Summary of ATM implementations in Germany

Location	ATM studied	Roadway Characteristics	Research Design	ATM on Operations	ATM on Safety	Research Problems or Comments
Germany, A5 (Sparmann, 2006)	VSL	• ADT of 150,000	N/A	N/A	• 27% reduction in crashes with heavy material damage •29% reduction in crashes with personal damage	• No methodology provided
Germany, A99 (Weikl, 2013)	VSL	• 16.3 km (~10 mi) section of A99 • 3 lanes each direction	• VSL system • 14 dual-loop detectors • 18 bottleneck cases	• Lane utilization of the roadway distributed more evenly at the slight cost of capacity • Flow change reduction of 4% when VSL was on and flow change reduction of 3% when VSL was off	N/A	• Gathered only total of 31 weekdays (25 days when VSL-ON and 6 days when VSL-OFF) for data analysis
Germany, A5 and A3 (Geistefeldt, 2012)	HSR	• 18 km (~11 mi) • 3 lanes each direction • high commuter traffic • distinct peak volumes	• 40 months of loop detector data • 47 sections of the roadway analyzed for duration of congestion analysis	• Median values of the capacity 10-25% higher than the capacity of comparable sections without HSR • Duration of congestion reduced from 640 hours/year and 450 hours/year for NB and SB respectively to less than 200 hours/year in both directions	N/A	• Did not mention information of the comparing sections
Germany A7 (Lemke, 2010)	HSR	• Three sections of the roadway which summed up to 36 km or ~22 mi •AADT of 35,000 on each of the three sections	• Original hand written police reports • 3 years of before and 3 years for after data analyzed	N/A	• Crash rates did not necessarily increase in all cases	N/A

Table 2. Summary of ATM implementation in the United Kingdom

Location	ATM studied	Roadway Characteristics	Research Design	ATM on Operations	ATM on Safety	Research Problems or Comments
U.K., M42 (Mott McDonald, 2008)	VSL, HSR	• 17 km (~11 mi) • Total AADT in both directions is 134,000 • 3 lanes wide each direction	• 12 months of before and 12 months of after data analyzed • 1 month of settling in period	• Average capacity increase of 7% • Total flow increase of 6% and 9% on NB and SB directions respectively • Average travel time increase of 9% • Variability of travel time reduced by 22% on both directions	• Average number of crashes per month reduced from 5.08 to 1.83 after ATM implementation • Severity index reduced from .16 to .14 after ATM implementation	• Additional development and construction work between ATM construction phases which may underestimate benefit of ATM • Preliminary safety analysis
U.K., M42 (Mott McDonald, 2011)	VSL, HSR	• 17 km (~11 mi) • Total AADT in both directions is 134,000 • 3 lanes wide each direction	• 36 months of before and 36 months of after data analyzed • 1 month of settling in period	N/A	• Average number of crashes per month reduced from 5.08 to 2.25 after ATM implementation • Severity index reduced from .16 to .07 • Monthly mean number of killed or serious injured casualties reduced from 1.15 to .19 • Two-way accident rate per billion vehicle miles traveled reduced from 115.92 to 47.98 • Proportion of rear-end collision crashes remained constant • Proportion of side impact collision crashes increased from 16.1% to 30.9%	• Final safety analysis

Table 3. Summary of ATM implementations in the United States

Location	ATM studied	Roadway Characteristics	Research Design	ATM on Operations	ATM on Safety	Research Problems or Comments
I-5, Washington (DeGaspari et al., 2012)	VSL, QWS	• 7 mile NB	• Total of 8 month before and after period • 19 loop detectors	• Planning time index improved by 17% - 31% • Buffer index improved by 15 - 27%	N/A	• Solely depended on detector data for analysis of entire roadway • Peak times seem odd
I-4, Florida (Atkins Consulting, 2009)	VSL	• 10 miles • AADT of 200,000	• Study period from 4PM to 6PM • 21 days of before VSL data and 30 days of after VSL data analyzed	• Speed changes were correlated with changes in occupancy than changes in the posted speed limit	N/A	• Study period way too short • Before and after periods do not match in season • Studying only 4PM to 6PM could result in problems
I-260 and I-255, Missouri (Kianfar et al., 2010)	VSL	• Total of 38 miles • Three bottleneck locations	• Inductive loop and acoustic detectors • 150 days of before VSL and 150 days of after VSL data analyzed • 10 days in between before and after VSL deployment for driver normalization	• Pre-queue flow decreased by up to 4.5% • Queue discharge flow decreased by up to 7.7% • Average speed fluctuated, but speed variance declined at all bottleneck locations	N/A	• Findings true for bottleneck locations only. Not plausible to conclude that the results apply to the entire roadway
I-35W and I-94, Minnesota (Hourdos et al., 2013), (Hourdos and Zitzow, 2014)	VSL	• AADT of over 160,000	• Single loop detectors, video recordings, crash records • 9 months of before VSL data, 17 months of after VSL data analyzed for operational analysis • 6 months of before VSL data, 6 months of after VSL data analyzed for safety analysis	• During AM peak period, 17% less congestion with the VSL system in operation for speed drop thresholds of 25 mph or more • 7.6 minute less congestion during the average AM peak	• Traffic pattern shows gradual decrease in speeds during the onset of congestion • No change in crash rates	• Solely depended on single loop detector data for analysis of entire roadway
I-35W, Minnesota (Kwon and Park, 2015)	VSL	• Urban location	• Traffic detector data • Sept-Nov of 2009 (before), 2010 (after), and 2011 (after) • Apr-Jun of 2010 (before), 2011 (after), and 2012 (after)	• Average travel time buffer index improved by 17-32%	• Maximum deceleration decreased by 10-22%	• Analysis of 6 months' worth of data may not show the full effects of VSL

CHAPTER 3: VIRGINIA I-66 ROADWAY AND ATM CHARACTERISTICS

Before discussing the methodology and results of this research, it is useful to fully describe the site conditions on I-66 given its unique characteristics. The ATM system implemented on I-66 used different ATM components along its length. Tables 4-5 show description summary of the roadway and operational characteristics of the study sections. Segments C and D from Table 4 were focus for this thesis, as these were the segments with the most ATM components implemented. The total length of these segments was approximately 13 miles on each EB and WB direction, with a regulatory speed limit of 55 mph. As shown in Table 5, Segments C and D were further subdivided into six segments. The division points were based on major interchanges along the corridor of study. On the segments without hard shoulder running (Segments 1-3), there was a HOV-2 lane and three general purpose lanes. On the segments with hard shoulder running (Segments 4-6), there was a HOV-2 lane, two general purpose lanes, and a shoulder lane available for travel using HSR. For EB and WB, the 2015 AADT varied by segment, ranging from 61,000 to 93,000.

3.1 I-66 ROADWAY CHARACTERISTICS

Discussions with VDOT Northern Regional Operations (NRO) staff on July 3, 2014 noted that analysis of the ATM system should focus on Segments C and D of the deployment since improvements to the other segments were more focused on improved monitoring. Early deployments of ATM strategies in Segments A and B will be limited, and the dynamic ramp metering on Segment E is scheduled to be activated at a later date. The impact of dynamic ramp metering and other ATM techniques on Segment E may be analyzed in a different study once all construction has been completed in that segment. As a result, Segments C and D were the focus of this research, and they were further broken down into six finer segments for detailed analysis based on NRO feedback. Table 5 and Figures 4-5 show the physical roadway characteristics and ATM characteristics of the six sub-segments of I-66 that were analyzed. It is important to note that there are different combinations of ATM techniques being implemented on different segments. For example, segments 1-3 do not employ the HSR component of the ATM. Also, since multiple techniques are being deployed simultaneously within a section, the before-and-after analysis will show the net effect of the combinations of all ATM techniques for each section.

Table 4. Characteristics of I-66 Roadway Segments.

Segment	Location	Length (mi.)	AADT (2012)	ATM techniques being implemented	Additional features	Physical roadway characteristics
A	US-15 (Exit 40) to US-29 Gainesville (Exit 43)	2.6	EB: 30,000 WB: 29,000	-	Increased CCTV camera, sensor, and dynamic message sign coverage	Currently in construction to improve from two to four lanes each direction. Upon completion of widening, HOV-2 rules will also apply on this segment
B	US-29 (Exit 43) to US-29 Centreville (Exit 52)	8.2	EB: 55,000 to 65,000 WB: 53,000 to 55,000	-	Increased CCTV camera, sensor, dynamic message sign coverage, and enhanced emergency pull-out zones	Four lanes each direction. HOV-2 rules still applies on this segment
C	**US-29|Lee HWY (Exit 52) to US-50 (Exit 57)**	**5.8**	**EB: 64,000 to 71,000** **WB: 62,000 to 72,000**	**VSL, LUCS, QWS, LUCs**	**Increased CCTV camera, sensor, dynamic message sign coverage, and enhanced emergency pull-out zones**	**Four lanes each direction. HOV-2 rules still applies on this segment**
D	**US-50 (Exit 57) to I-495 (Exit 64)**	**7.2**	**EB: 76,000 to 91,000** **WB: 84,000 to 86,000**	**VSL, LUCS, QWS, HSR**	**Increased CCTV camera, sensor, dynamic message sign coverage, and enhanced emergency pull-out zones**	**Three lanes + shoulder lane both direction. Right shoulder lane used as travel lane during respective peak hours to maintain three general travel lanes while leftmost lane acts as HOV-2 lane. Median is used by heavy rail in sections of this segment**
E	I-495 (Exit 64) to DC Line (~Exit 75)	10.2	EB: 33,000 to 65,000 WB: 34,000 to 65,000	Dynamic ramp metering	Increased CCTV camera, sensor, dynamic message sign coverage, and enhanced emergency pull-out zones	Two lanes both direction, additional lane for entry/exit through selected segments. Entire roadway reserved for HOV-2 eastbound in the morning and westbound in the afternoon

Table 5. I-66 Final Segments for Analysis

Segment	**Location**	**Length (mi.)**	**Speed Limit (mph)**	**AADT (2015)**	**ATM techniques**	**Physical roadway characteristics**
1	US-29 (Exit 52) to VA-28 (Exit 53)	1.3	55	EB: 68,000 WB: 66,000	VSL, LUCS, QWS	Four lanes each direction. HOV-2 rules still applies on this segment
2	VA-28 (Exit 53) to VA-286 (Exit 55)	1.9	55	EB: 80,000 WB: 82,000	VSL, LUCS, QWS	Four lanes each direction. HOV-2 rules still applies on this segment
3	VA-286 (Exit 55) to US-50 (Exit 57)	2.6	55	EB: 65,000 WB: 61,000	VSL, LUCs, QWS	Four lanes each direction. HOV-2 rules still applies on this segment
4	US-50 (Exit 57) to VA-123 (Exit 60)	1.9	55	EB: 90,000 WB: 93,000	VSL, LUCS, QWS, HSR	Three lanes + shoulder lane both direction. Right shoulder lane used as travel lane during respective peak hours to maintain three general travel lanes while leftmost lane acts as HOV-2 lane
5	VA-123 (Exit 60) to VA-243 (Exit 62)	2.1	55	EB: 93,000 WB: 81,000	VSL, LUCS, QWS, HSR	Three lanes + shoulder lane both direction. Right shoulder lane used as travel lane during respective peak hours to maintain three general travel lanes while leftmost lane acts as HOV-2 lane
6	VA 243 (Exit 62) to I-495 (Exit 64)	3.2	55	EB: 82,000 WB: 86,000	VSL, LUCS, QWS, HSR	Three lanes + shoulder lane both direction. Right shoulder lane used as travel lane during respective peak hours to maintain three general travel lanes while leftmost lane acts as HOV-2 lane. Median is used by heavy rail (Metrorail)

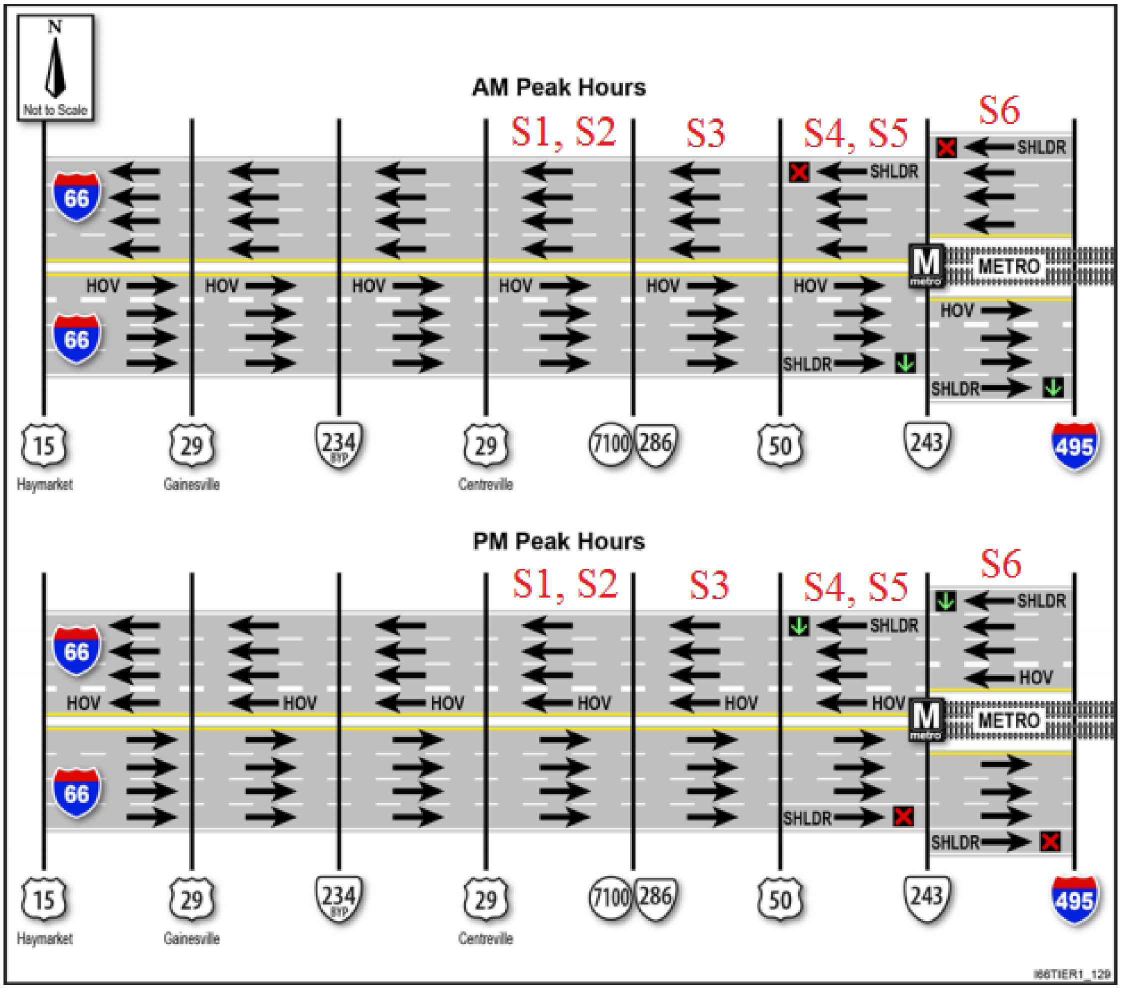


Figure 4. Physical Roadway Characteristics of I-66, with Segments from Table 5

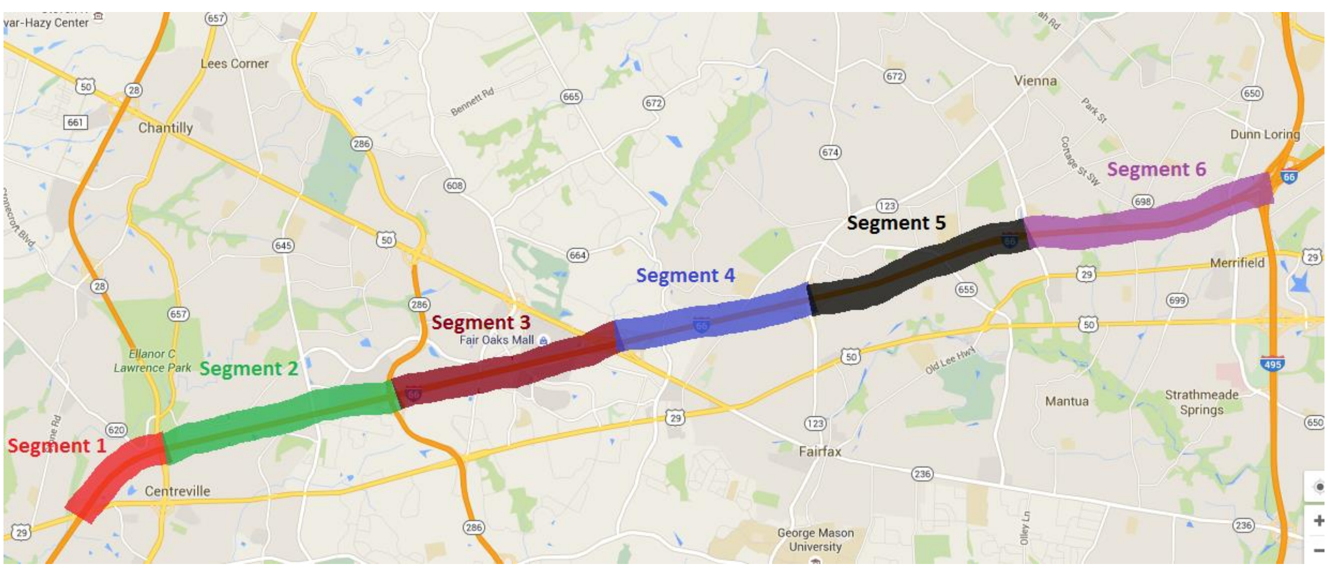


Figure 5. Map of ATM Implementation Segments from Table 5

Most, if not all segments of the corridor of I-66 experienced steady traffic volume growth throughout the years on both average weekdays and weekends. Table 6 shows the generally increasing trend of the traffic volume increase on I-66 from 2012 to 2015. The corridor-level AADT growth rate calculated by using weighted averages by length of segment showed an average annual volume growth rate from 2012 to 2015 of approximately 2 percent on weekdays and 1 percent on weekends. With the increase in traffic volumes, the average travel times for the corresponding years have increased as well. Generally before the implementation of ATM, the average travel times along the corridor have increased on peak, midday, and even on off-peak periods. The overnight period was the only period without much average travel time change. Figures 7-8 show the general increasing average travel time trends over the years prior to ATM activation.

Table 6. AADT for 2012-2015 on EB and WB I-66 Segments

Direction	Segment	AADT - Average Weekday				AADT - Average Weekend			
		2012	2013	2014	2015	2012	2013	2014	2015
EB	1	66000	70000	70000	71000	59000	56000	56000	60500
	2	74000	83000	82000	85000	63500	65500	68000	67500
	3	68000	68000	67000	68000	61000	57500	60000	57500
	4	94000	93000	92000	94000	76500	75500	74500	80000
	5	97000	96000	96000	99000	76000	78500	75000	78000
	6	80000	79000	79000	86000	66000	68500	65000	72000
WB	1	66000	69000	68000	70000	52000	55000	54000	56000
	2	76000	88000	87000	87000	62000	70500	69500	69500
	3	67000	66000	65000	65000	53000	55500	54500	51000
	4	91000	90000	89000	98000	73500	72500	71500	80500
	5	91000	90000	89000	85000	73500	72500	71500	71000
	6	87000	86000	86000	91000	76500	75500	72000	73500

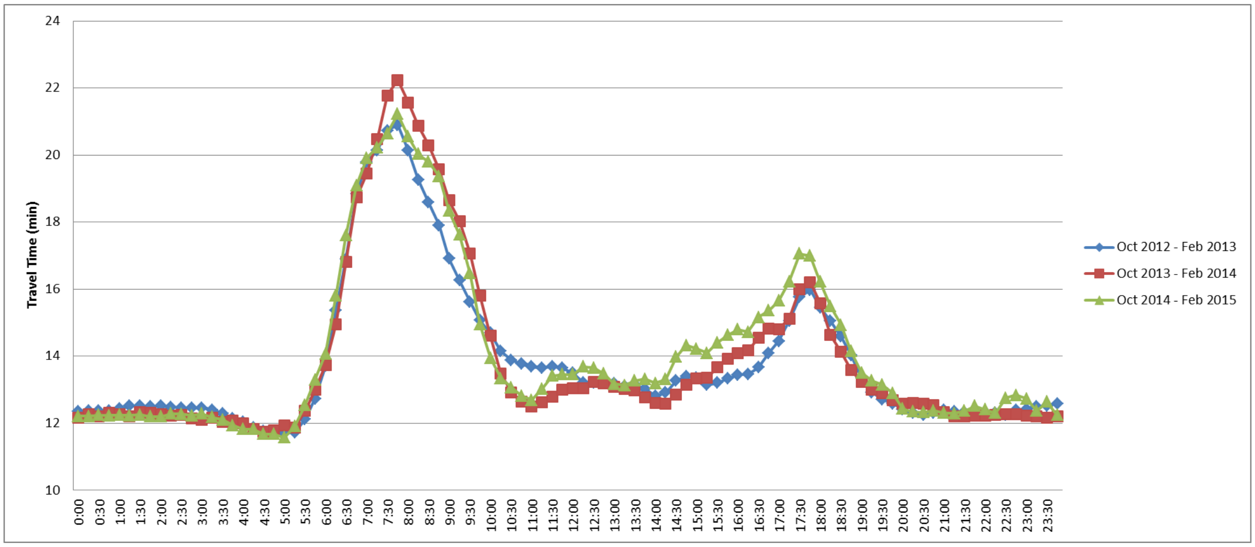


Figure 6. EB Average Weekday Average Travel Time Trend (Corridor-level)

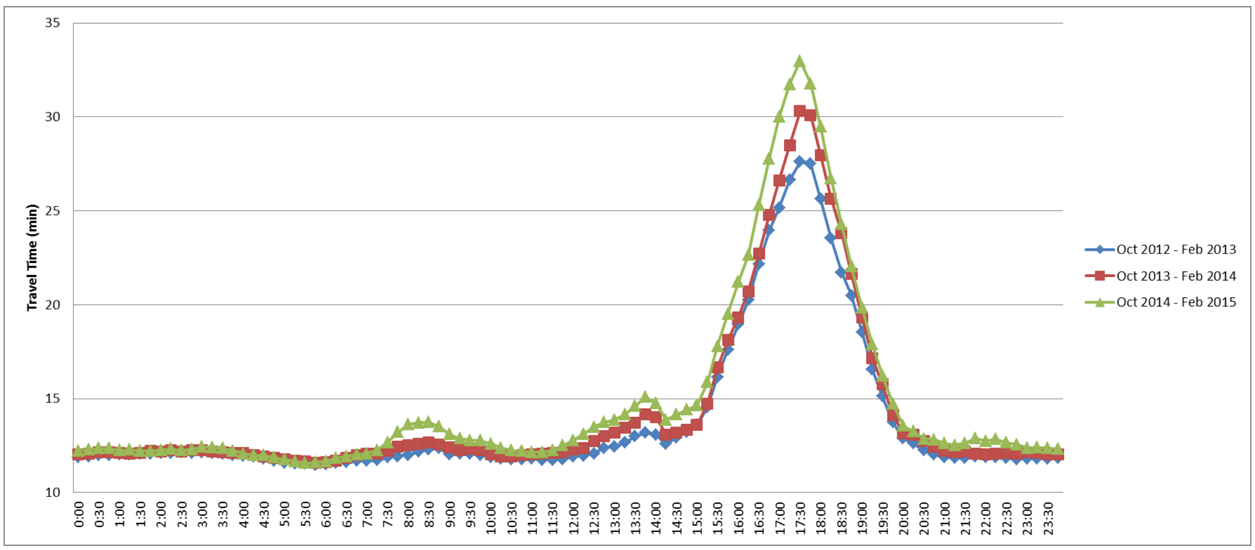


Figure 7. WB Average Weekday Average Travel Time Trend (Corridor-level)

3.2 CONDITIONS ON I-66 BEFORE AND AFTER ATM IMPLEMENTATION

This section describes key traffic control characteristics that were present in the study corridor, including both the before and after ATM periods. The ATM system became active on September 16, 2015. The VSLs were initially activated, but taken off line after a week of operation due to issues with the control algorithm. They were subsequently reactivated in mid-January 2016.

3.2.1 HOV Restrictions on I-66

An HOV-2 lane is present in both directions of the study section. The HOV-2 hours did not change for both before-and-after ATM conditions. Outside of the I-495 Beltway, the HOV-2 hours are:

- Eastbound: 5:30 am – 9:30 am
- Westbound: 3:00 pm – 7:00 pm

As shown in Figure 4, HOV lanes were present throughout the study section, and were only separated by pavement markings.

3.2.2 Shoulder Opening Hours

Before the implementation of ATM, the shoulders on Segments 4-6 were open only during set fixed peak periods. The before-ATM static shoulder opening hours were as follows:

- Eastbound: 5:30 am – 11:00 am

- Westbound: 2:00 pm – 8:00 pm

The shoulders were only open on weekdays, and were not opened during federal holidays. After the implementation of ATM, shoulders continued to be open during the before-ATM peak period, but were also opened whenever there were needs for additional road capacity. Thus, a major change is that during weekday non-peak hours and weekends the shoulders can now be opened when an increase in roadway capacity was warranted.   This should allow the ATM system to add capacity to better handle traffic demands during incidents, work zones, or unusual fluctuations in demand.

3.2.3 Characteristics of Variable Speed Limits

VSL signs were deployed on overhead gantries throughout the corridor once ATM was installed. Inconsistencies with the VSL algorithm caused the VSL component of ATM to be deactivated after one week of initial operation for fine tuning. The VSL component was reactivated in mid-January 2016 with an enhanced algorithm. The algorithm deployed was developed by Delcan, and a specific evaluation of the mechanics of the algorithm was not in the scope of this evaluation. Generally speaking, the algorithm examines real time speed data, and then smooths and troops adjacent signs to develop easy transitions into and out of congested conditions. Speeds are gradually lowered approaching congestion, hopefully reducing conflicts.

The speed limits for VSL can go low as 35 mph. As the VSL speed limits are advisory, the police cannot enforce the VSL speed limits although they can write citations for failure to comply with traffic control.

3.2.4 Lane Use Control Signs

The LUCS control roadways to manage roadway incidents and work zones, and were implemented on overhead gantries throughout the corridor. Figure 9 shows the signs that LUCS use for roadway management, and Figure 10 shows an example of LUCS activation on I-66.

Figure 8. Available Lane Control Signs on I-66

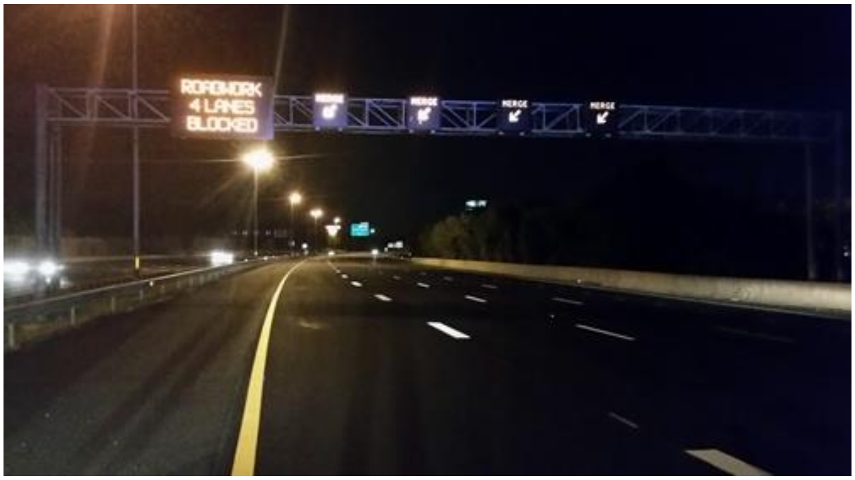


Figure 9. Example of LUCS in Operation

Figure 10 shows an example of gantries in sequences that are working to resolve a crash event that is blocking the right lane (L3) during normal peak period (Iteris, 2011). The signs redirect flow of upstream vehicles out of the problematic lane as well as the shoulder beforehand to reduce the effect of bottleneck as much as possible. QWS is also activated to alert upstream vehicles of the problematic crash event ahead.

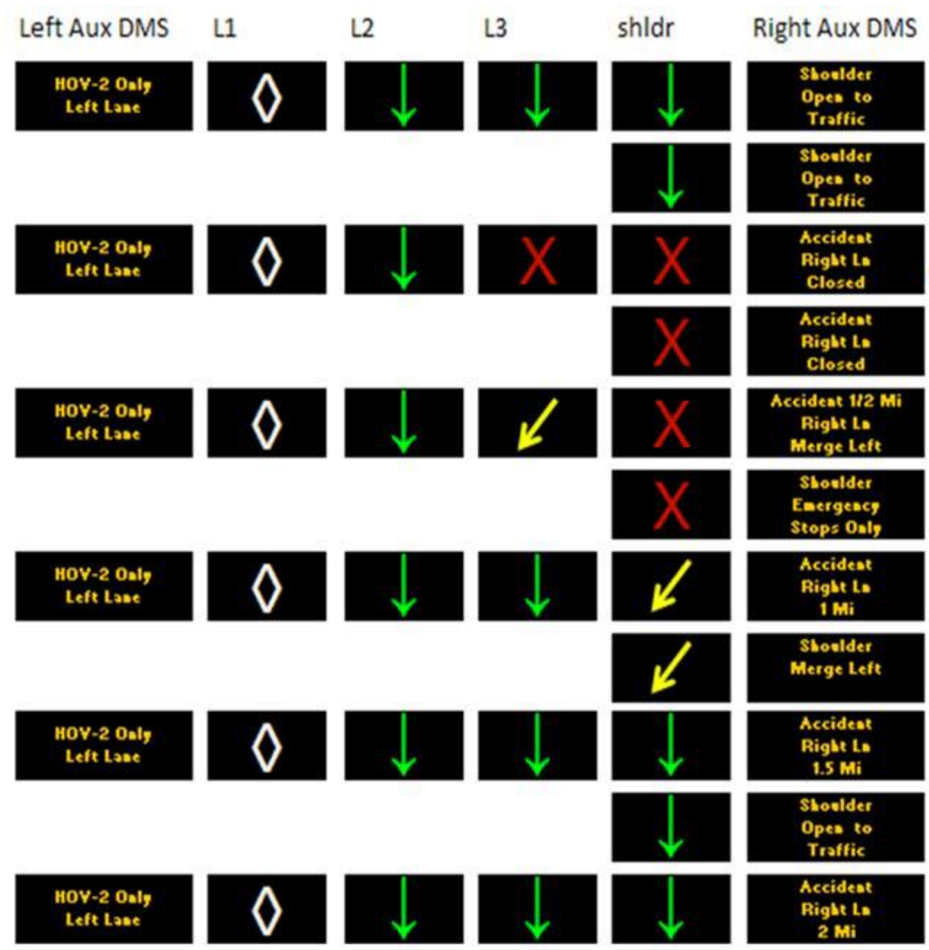


Figure 10. Example of Lane Use Control Signs in Operation when Accident on Right Lane of Roadway

Since the diagonal yellow arrow is not a MUTCD standard indication, a different study is analyzing the effects of yellow arrow signs on I-66. That study will evaluate the microscopic effects of the approaching vehicle behaviors due to the LUCS.

3.2.5 ATM Gantry Locations

New infrastructure was constructed to implement the ATM systems on I-66. A total of 21 new gantries were constructed in each direction, and the approximate average distance between gantries was 0.5 miles apart. Each gantry contained lane use control signals that could be used to indicate whether drivers were approaching closed lanes, as well as to post reduced VSLs. Figures 11-12 below shows the milepost locations of the new gantries that were constructed, and whether the new gantry was used for HSR or not.

Figure 11. Gantry Locations for Segments 1-3 (HSR not present)

Figure 12. Gantry Locations for Segments 4-6 (HSR present)

Each gantry contained dynamic message signs over each lane which could display the VSL, LUCS, HSR, and or QWS. Figure 13 shows an example of a gantry installed on the I-66 corridor. This particular gantry employs all of the components of the ATM.

Figure 13. Example of Gantry with ATM Techniques

CHAPTER 4: METHODOLOGY

The methodology consisted of the following tasks:

- Literature Review
- Review of I-66 Characteristics and ATM Project Documentation
- Before-and-after ATM Analysis (Operations and Safety)
- Benefit-cost Analysis

These tasks are discussed in more detail on the following pages.

4.1 LITERATURE REVIEW

ATM-related literature from both Europe and the United States were reviewed for this thesis. The literature review focused on the operations and safety impact of similar ATM field deployments, with a particular emphasis on the methods used to evaluate these systems and the results. The literature review focused on observed, empirical studies of ATM field deployments rather than simulation studies. Since driver reactions to ATM will significantly impact the overall system effectiveness, field studies were examined in detail to determine how actual drivers reacted to ATM techniques.

In most cases, the ATM implementations from the literature review were not as expansive as the I-66 ATM implementation. The I-66 ATM implementation included many ATM techniques such as VSL, HSR, QWS, and LUCS while the ATM implementations from the literature review often mention having only one or two of the mentioned techniques. As a result, comparisons between past studies and I-66 will have to account for varying system configurations.

4.2 REVIEW I-66 CHARACTERISTICS AND ATM PROJECT DOCUMENTATION

The characteristics and features of the I-66 ATM project were examined. Some of the steps associated with this task include:

- Identifying basic project characteristics (e.g. project location on I-66 corridor, ATM characteristics)
- Identifying other projects that are under way that may affect operations and safety data (such as major work zones)
- Identifying recurring congestion time periods
- Identifying exact locations where ATM techniques will be implemented (e.g. gantry locations, DMS locations)
- Determining sensor locations and data elements to be collected

The goal of this task was to thoroughly document the ATM system, and identify site characteristics that will influence the before-after analysis.

Maintaining constant communication and discussions with VDOT Norther Region Operations (NRO) helped to narrow the scope of the research and guided the project to become more relevant to all parties. The project location was determined to be the section of I-66 with the heaviest implementation of ATM and with the most traffic volume, between US 29 in Centreville and I-495.

4.3 DATA ANALYSIS OF BEFORE-AND-AFTER ATM CONDITIONS

Traffic safety and operational data for before-ATM and after-ATM implementation conditions were analyzed. The safety and operational effects of the ATM on I-66 corridor were analyzed at a corridor-level as well as at a segment-level, as the segments of I-66 implement different combinations of ATM techniques. Table 7 below shows the measures of effectiveness that will be analyzed, as well as the data sources that will be utilized to conduct the before-and-after ATM evaluation.

Table 7. Operations and Safety Measures of Effectiveness for ATM analysis

	Measure of Effectiveness	**Data Sources Used for Calculation**
Operation	Average travel time	INRIX
	Travel time reliability (i.e. buffer index, planning time index)	INRIX
	Recurrent congestion levels	INRIX + limited point sensors
	Non-Recurrent congestion levels	INRIX + limited point sensors
	Utilization of ATM system (post deployment only)	Traffic operations center (TOC) logs
Safety	Crash frequency, severity and rate	VDOT Roadway Network Systems (RNS) + limited point sensors
	Safety surrogate measure (speed drop events)	INRIX + VDOT RNS

Many operations and safety performance measures (e.g. average volume, maximum throughput, speed limit compliance rate, and speed variance) were initially identified as primary evaluation metrics. These metrics relied on having detailed point detector data from the ATM system detectors. Unfortunately, these data could not be examined due to technical problems with the detector data archive. Configuration problems related to the detector archive resulted in losses of data for the after-ATM period initially. Subsequent technical problems with the detector data archive and contractual disputes between VDOT and the ATM vendor on how to fix the archive made it impossible to query ATM point detectors during the course of this thesis. While these measures would have been very valuable for this study, they could not be obtained.

4.3.1 Data Used for Analysis

The analysis of traffic operations will be performed using a combination of INRIX travel time data, limited point sensor data, and TOC ATM utilization log records. For the safety analysis, INRIX travel time data and RNS police crash records will be used.

4.3.1.1 INRIX Data

VDOT has access to INRIX real-time probe-based travel time data throughout the I-66 corridor. INRIX is a private company that determines speed and travel time data by mining GPS data from smartphones and commercial fleet management systems (Haghani et al., 2009). INRIX processes this GPS probe data to estimate speeds, which are reported spatially using TMC links. TMC links are spatial representations developed by digital mapping companies for reporting traffic data, and consist of homogeneous segments of roadways. On freeways, TMCs typically end at ramp junctions or at locations where the number of mainline lanes change. There were 14 TMCs summed up to total of 12.414 miles in the EB direction and 14 TMCs summed up to 12.345 miles in the WB direction. The length of each TMC varied from 0.22 to 1.85 miles. The data available from INRIX include average travel time, length of traffic message channel link (TMC), and average speed. It is important to note that analysis using INRIX data can be sensitive to temporal (e.g. 5-minute interval vs. 1-hour interval) and spatial (e.g. a TMC that is 0.5 mile long vs. a TMC that is 1 mile long) aggregations. The INRIX data provides wide spatial coverage throughout the corridor, which will allow a comprehensive examination of travel times (Fontaine et al., 2013). Since INRIX is calculating segment speeds using GPS probe data, it represents the space mean speed over a segment of road, which is a deviation from prior studies that relied on time mean speeds from point detectors. The validity of INRIX freeway travel time data has been previously established by the I-95 Corridor Coalition Vehicle Probe Project through a comparison with Bluetooth travel time data (Haghani et al., 2009). VDOT currently uses INRIX data to support a variety of performance measurement and traveler information applications.

Since the INRIX data relies on vehicle probes, real-time data may not be available continuously, especially during low flow periods. INRIX provides confidence scores for each 1-minute interval, with a confidence score of 30 representing real-time data and scores of 10 and 20 representing historic data during overnight and daytime periods, respectively. For the purposes of this analysis, average travel times were determined for every 15-minute interval, and that 15-minute travel time interval had to have an average confidence score of 26.67 or higher for at least 85% of the TMC length to be retained for analysis. These thresholds were derived from VDOT travel time business rules, and time periods that did not meet this threshold were discarded from analysis. The 15 minute time interval balances the need to examine short-term changes in performance with the time required to process data. Longer time intervals would dampen reliability effects, and shorter time intervals would require more resources to analyze and suffer from more individual periods with no data. It is important to have a balance of both data quantity and detail for the sake of time and accuracy respectively.

4.3.1.2 Volume Data

Since the INRIX probe vehicles represent a small portion of the total vehicles on the roadway, INRIX does not provide volume data. As noted earlier, configuration problems related to the ATM detector archive made querying the existing database impossible. As a result, real-time traffic volume counts during and after-ATM activation were not available from VDOT for this analysis. However, it was possible to obtain limited archived real time before-ATM traffic volume data from the RITIS detector tools database. AADT estimates were also available from VDOT along the corridor throughout the study period, although real time counts following ATM deployment were not available. For some performance measures, the before-ATM traffic volume distributions were used to estimate performance measures in the after period by assuming a traffic growth rate based on AADT changes. While it is possible that hourly distributions of traffic did change after ATM installation, no data was available from VDOT to determine whether this was the case. Given observed operational data, especially on weekdays, it was expected that this was a reasonable assumption, however.

4.3.1.3 Traffic Operations Center (TOC) Logs

TOC logs were reviewed to determine the times when hard shoulders were opened to travel, as well as the time periods when VSL and LUCS were posted. The TOC logs consisted of information on the sign message, time stamp when the message was posted, and a location identifier for the sign. Thus, the specific message being displayed on every individual LUCS could be tracked over time. This was used to determine the amount of additional time that shoulders were opened to travel, as well as the duration and times of day when VSL and LUCS were used.

4.3.1.4 Crash Data

VDOT has records of police crash reports along the corridor in a database called the Roadway Network System (RNS). However, the police reports are transmitted onto RNS on a rolling basis with a lag time of 3-4 months. Therefore, the most recent crash reports could not be analyzed in this thesis. Only crashes through the end of December 2015 were available for analysis for this thesis. Information on crash frequency, severity, crash type, and location was collected from this database.

4.3.2 Time Periods Analyzed

The INRIX database contains travel time data from 2010 to the present, which means that there are data for at least three years of pre-ATM installation conditions. However, road characteristics (such as increasing traffic volume) on I-66 have changed over the course of time and INRIX data quality continues to improve over time. As a result, analyzing all three years' worth of data may not provide the most accurate information on pre-deployment conditions. For example, part of segment 4 of I-66 was widened from two lanes in both directions to four lanes between the VA-234 Bypass and US-29 in Gainesville in 2010. This widening may have decreased the average travel time of traffic because of the physical road capacity increase. Also, according to the discussion with NRO staff on July 30, 2014, the recent opening of the Phase I of Metro's Silver Line on July 26, 2014 may have created significant traffic pattern changes on I-66. The Phase I of Metro's Silver Line was a $2.9 billion project that extends the existing Metrorail system towards Reston, VA by 23 miles (Metropolitan Washington Airports Authority, 2012). As a result, before-after comparisons in this thesis are primarily focused on the conditions on I-66 after the opening of the Silver Line for the before period.

ATM on I-66 was first activated in September 2015. However, drivers will be unfamiliar with the new system initially and their behavior may change over time as they become more comfortable with the new system on I-66. Therefore, drivers need some time to acclimate themselves to this new system, and this acclimation period will help stabilize the after-ATM traffic pattern. The acclimation period following activation of the ATM system on I-66 was defined to be approximately 2.5 weeks, from 9/16/2015 to 10/4/2015. Also, the two extreme non-recurrent events, which were the Pope's visit to Washington D.C. on 9/23/2015 to 9/25/2015, and the arrival of Hurricane Joaquin in Virginia on 10/2/2015 to 10/4/2015, were the other contributing factors in the selection of this 2.5 week acclimation period. In total, 21 weeks' worth of after-ATM data, from 10/5/2015 to 2/28/2016 was examined in this paper. This results in approximately 21 Mondays-Sundays being analyzed for this paper, and 21 weeks of before-ATM data (Oct 2014 – Feb 2015) are compared with 21 weeks of after-ATM data (Oct 2015 – Feb 2016). While 2012-2014 average travel time and crash data were not analyzed for the before-and-after analysis, they were analyzed to review the operations and safety trends throughout the years before the implementation of ATM. This provided an indication as to whether the post-ATM data revealed changes in trends in crashes or safety from what was being experienced prior to system installation.

Analysis was segregated by day of week and time of day (i.e. AM peak, midday, PM peak, overnight). The time of day periods were defined based on the pre-ATM shoulder opening hours so that operational results could be fairly compared. Also, the corridor was divided up into six segments for the segment-level analysis, with the segments ranging between 1.3 to 2.6 miles. Both segment-level and corridor-level analysis were conducted. The segment details are shown in Chapter 3 of this thesis. Since the ATM implementation, the HSR component of the ATM has been the most active system in operation. To emphasize the effects of the HSR, Segments 4-6, where shoulders are present, were analyzed individually from the entire corridor (Segments 1-6) analysis.

4.3.3 Operations Analysis – ATM Utilization

It was possible to analyze the utilization rates of the ATM techniques using the activation logs stored at the traffic operations center. The activation log contained detailed records of ATM usage by each gantry and by individual lane use control sign. In general, the ATM techniques showed two characteristics during activation:

1. The ATM technique was activated, then was deactivated
2. The ATM technique was activated, then was changed to another message

Since not all gantries are located where shoulders are present, it was necessary to filter out gantries that were not used for HSR for the HSR utilization analysis. It was found that 11 out of 21 gantries were used for HSR on EB, and 9 out of 21 gantries were used for HSR on WB. HSR utilization analysis was divided into direction and day of the week (i.e. average weekday, average weekend). HSR utilization rates were calculated by adding up the total time of HSR activation per each gantry then dividing up the total by the number of days in the analysis period. This utilization rate represents average HSR utilization rate per day for each gantry.

All gantries were included for the VSL utilization analysis. VSL was deactivated shortly after first activation for fining tuning and was re-activated in mid-January 2016 with an enhanced algorithm. Therefore, VSL utilization analysis was performed with only mid-January to February data. VSL utilization analysis was divided into direction and day of the week as VSL. VSL utilization rates were calculated by adding up the total time of VSL activation per each gantry then dividing up the total by the number of days in the analysis period. This utilization rate represents average VSL utilization rate per day for each gantry. Also, use of different speed reduction signs was analyzed by evaluating the utilization of each speed reduction speeds (i.e. 35-50 mph in 5 mph increments).

All gantries were included for the LUCS utilization analysis. LUCS utilization analysis was divided into direction and day of the week. The utilization rate of LUCS is less frequent than the activation of VSL or HSR. Therefore, it made more sense to analyze LUCS for the frequency and total duration of activation per gantry.

4.3.4 Operations Analysis - Average Travel Times

INRIX travel time data were acquired in 15-minute temporal aggregation, data quality screening measures were conducted, and travel times were segregated by appropriate segments, days of the week, and peak and non-peak periods. Using the average travel time data, average travel time profiles were constructed using comparable months for 3 years of before-ATM and 1 year of after-ATM period. Paired t-tests were conducted at $\alpha = 0.05$ level to determine if the changes were statistically significant between October 2014-February 2015 and October 2015 and February 2016. For each day of the week and average weekday and weekend, the 15-minute average times were divided up into time of day for both before-and-after ATM periods to set up the paired t-test. These groups of average travel times were then matched up by their appropriate before-and-after periods. For example, all of the 15-minute average travel times for weekday AM peak period from 5:30 to 11am for the before-ATM period was paired, and then compared to that of the after-ATM period. This guaranteed one-to-one match for the paired t-test as the number of days for the before-and-after analysis was the same.

4.3.5 Operations Analysis – Travel Time Reliability

In addition to examining changes in mean travel time, changes in travel time reliability were also examined using the planning time index (PTI) and buffer index (BI). The planning time index value shows the total time travelers should account for in order to be on-time 95% of the time relative to free flow speeds. The buffer index value shows the extra time travelers should add to their average travel time in order to ensure they are on-time 95% of the time. Travel time reliability measures were derived directly from INRIX travel time data for both before-and-after ATM periods. The equations used to calculate PTI and BI are as follows:

$$\text{Buffer Index} = \frac{95th\ percentile\ average\ travel\ time - average\ travel\ time}{average\ travel\ time}$$

Equation 1. Buffer Index

$$\text{Planning Time Index} = \frac{95th\ percentile\ average\ travel\ time}{free\ flow\ average\ travel\ time}$$

Equation 2. Planning Time Index

For PTI calculations, free flow average travel times were calculated by using 55 mph as the free flow speed, which is the posted regulatory speed limit. Paired t-tests were conducted at $\alpha = 0.05$ level to analyze the statistical significance of the PTI and BI changes.

Since travelers are usually going faster than the speed limit during low traffic flow hours, it is possible to have a PTI value of less than 1. For buffer index, the baseline average travel time value changes, unlike the planning time index. Before and after buffer index values use respective before and after average travel time values as the denominator. This means that in most cases, the after-ATM buffer index value is calculated by the improved after-ATM average travel time, and the calculated after-ATM buffer index value is a conservative number compared to the calculated before-ATM buffer index value. Reductions in PTI and/or BI would show that the ATM system has contributed to a more predictable, consistent trip for drivers. Since many of the ATM system's components may have a greater impact on mitigating the effects of non-recurring congestion, reliability changes may be greater than changes in mean travel time.

4.3.6 Operations Analysis – Recurrent Congestion/Non-Recurrent Congestion Analysis

A recent publication from Old Dominion University (ODU) identified K-Nearest Neighbor (K-NN) classification as a method of quantifying the magnitude of recurrent and non-recurrent congestion on a roadway (Cetin et al., 2014). The ODU recurrent congestion analysis was conducted on a 3.5 mile Hampton Roads Bridge Tunnel (HRBT) corridor. It also used INRIX travel time data to create a reference travel time profile, and this reference travel time profile was compared with the free flow travel time profile and traffic volume data to determine the magnitude of recurrent congestion. This reference travel time profile is considered to be a travel time profile during a typical day for the corridor, provided that the incident-affected portion of all the observed travel time profile is removed (Cetin et al., 2014). I-66 is constantly plagued with non-recurrent congestion events, such as vehicle incidents, which makes it difficult to separate recurring and non-recurring events when developing a reference travel time profile. Because of these reasons, a different approach must be taken when implementing the ODU paper's methods of quantifying the magnitude of recurrent and non-recurrent congestions on a roadway. Unlike the ODU's methodology, non-recurrent congestion events are considered when formulating the reference travel time profile on I-66, as they are so common on I-66.

K-Nearest Neighbors classification method is a nonparametric method used for classification and also in regression, in which closest similarities are measured among the k nearest observations in Euclidean space to the new unclassified observation (Ripley, 1994). The K-NN algorithm is a method of classifying cases based on their similarity to other cases, and it is used to recognize patterns of data without requiring an exact match to any stored patterns or cases. Similar cases are near each other (neighbors) and dissimilar cases (still neighbors, but not so much) are distant from each other, and the distance between two cases, in Euclidean space, is a measure of their dissimilarity (IBM, 2014). The k value in this analysis means the number of nearest neighbors to examine. However, greater number of neighbors will not necessarily result in a more accurate model (IBM, 2014). Like clustering analysis methods described above, the distance metric used to measure the similarity of cases will be Euclidean distance. K-value sensitivity analysis was done for a preliminary analysis, and it was concluded that the root mean square error was the least when k = 9. This k-value result was consistent with the ODU report's result.

Using the SPSS Statistical software, the K-NN classification method was used to create appropriate reference travel time profiles by inputting the average travel times and respective 15-minute time interval values into the software. The variable that SPSS will output is called the *KNN_PredictedValue*, which is a predicted value for a scale dependent variable, or in this case, reference travel time value (IBM, 2014).

The magnitude of recurrent and non-recurrent congestion can be determined by calculating the following equation for each of the 15-minute interval average travel times:

$$Recurrent\ Congestion = \begin{cases} 0 & if\ FFTTP \geq RTTP \\ (RTTP - FFTTP) \times Volume & if\ FFTTP < RTTP \end{cases}$$

Equation 3. Recurrent Congestion

$$NonRecurrent\ Congestion = \begin{cases} 0 & if\ RTTP \geq ATTP \\ (RTTP - FFTTP) \times Volume & if\ RTTP < ATTP \end{cases}$$

Equation 4. Non-recurrent Congestion

- Recurrent Congestion and Non-Recurrent Congestion is in minutes
- FFTTP = Free Flow Travel Time Profile, which is defined as the travel time through the corridor at a constant 55 mph speed. Speeds faster than 55 mph result in 0 delay, not negative delay
- RTTP = Reference Travel Time Profile, which is defined using the k-NN profile with k = 9
- ATTP = Average Travel Time Profile

Since volume data was not available for after-ATM conditions, some assumptions had to be made. Since volumes are so different for the non-shoulder running and shoulder running sections of I-66, they were analyzed separately. The daily volume distributions for before-and-after ATM conditions were assumed to be the same. AADT for after-ATM were developed using before-ATM conditions and using average weighted average (weighted by length of segment) growth rates across the segments.

Once all of the recurrent congestion and non-recurrent congestion values were calculated for each 15-minute interval, the summation of the respective values represent the average daily magnitude recurrent and non-recurrent congestion levels. This total delay was examined to determine if the system produced a net benefit on operations, including if it slowed the rate of travel time delay increases from pre-ATM conditions.

4.3.7 Safety Analysis - Data

Since RNS police crash reports were not available from January 2016 – February 2016, only October 2015 – December 2015 data were analyzed for the safety analysis. Although only

3 months of post-ATM data are available, this limited data may provide some preliminary insight into the safety effects of the system. These results will not be conclusive, but may help provide insight into performance, particularly when viewed in parallel with operational data.

4.3.8 Safety Analysis - Crash Rate Analysis

Crash rates were analyzed at a corridor-level by using weighted average AADT values (weighted by length of segment). Both total crash and rear-end and sideswipe crash cases were analyzed, and severity was separated into Property Damage Only (PDO) and Injury + Fatal types for this analysis. The crash rate, expressed as crashes per 100 million vehicle-miles of travel, is calculated by using the following equation:

$$\text{Crash Rate} = \frac{Crash\ Frequency \times 100{,}000{,}000}{AADT \times Days \times Length}$$

Equation 5. Crash Rate

In order to analyze the crash rate trends, October to December of 2012, 2013, 2014, and 2015 crashes were analyzed for the crash rate analysis.

4.3.9 Safety Analysis - Crash Surrogate Analysis

One of the goals of the ATM project is to reduce the number of crashes occurring at locations where vehicles transition from high speeds to congested flow. At these locations, approaching drivers may have to rapidly reduce their speed as they approach the end of the queue. ATM may alleviate this potential safety problem by notifying upstream drivers that there may be possible congestion ahead, and also by actively promoting speed harmonization along the entire freeway (Mirshahi et al., 2007).

Although it would be ideal to assess the safety impact of the I-66 project using actual crash data from after ATM implementation, it takes time to accumulate enough data to perform a statistically robust evaluation. VDOT indicated that they would like to have very quick feedback on whether the system is having the intended positive safety effect. As a result, there was interest in determining whether safety surrogate measures could be well correlated with past safety at the site. In response, research was conducted to investigate the relationship between the frequency of mainline speed drop events at interchanges and crash frequency to determine whether speed drop events are a reliable surrogate measure for crash frequency (Chun and Fontaine, 2016). Interchange areas were selected for evaluation due to their shorter TMC lengths. Initial investigations revealed that there was a strong linear relationship between speed drop events and crash frequency near interchanges. INRIX travel time data and RNS crash report data were analyzed for all of 2012-2014, and it was determined that the Pearson's Correlation value for speed drop events at speed threshold of 10 mph and rear-end and sideswipe crash frequency was 0.826, which shows a strong linear relationship. This was the first attempt at correlating probe data results at the TMC level with crash frequency. Instead of waiting three full years to collect after-ATM crash data for a typical before and after safety analysis, these relationships were used to perform a rapid evaluation of potential safety changes following ATM activation.

Janson et al. performed a test of crash frequency differences in successive 0.05 mile sections both upstream and downstream of ramps in order to determine the relationship between the extent of ramp influence zone and crash frequency. The researchers examined the number of relevant crashes in adjacent 0.05 mile sections in order to determine where crash frequencies became asymptotic. Using this method, the researchers found that ramp influence zones typically extended 0.25 miles upstream of the tip of an exit ramp taper to 0.15 miles downstream of the end of an entrance ramp taper (Janson et al., 1998). This ramp influence zone threshold was implemented for this thesis and the, interchange influence area was defined to be 0.25 miles upstream of the tip of the exit ramp, the interchange section in between the on and off ramps, and the 0.15 miles downstream of the end of an entrance ramp taper. Using this ramp influence zone definition, approximately 70% of all crashes that occurred on I-66 between mileposts 52 and 64 occurred within the ramp influence zones during the study period. In total, the ramp influence areas included approximately 17 miles of the 24 mile analysis corridor, representing approximately 70% coverage. Crashes and speed drop events that occurred around the interchange influence area were used for the analysis. An example of a ramp influence zone can be seen on Figure 14.

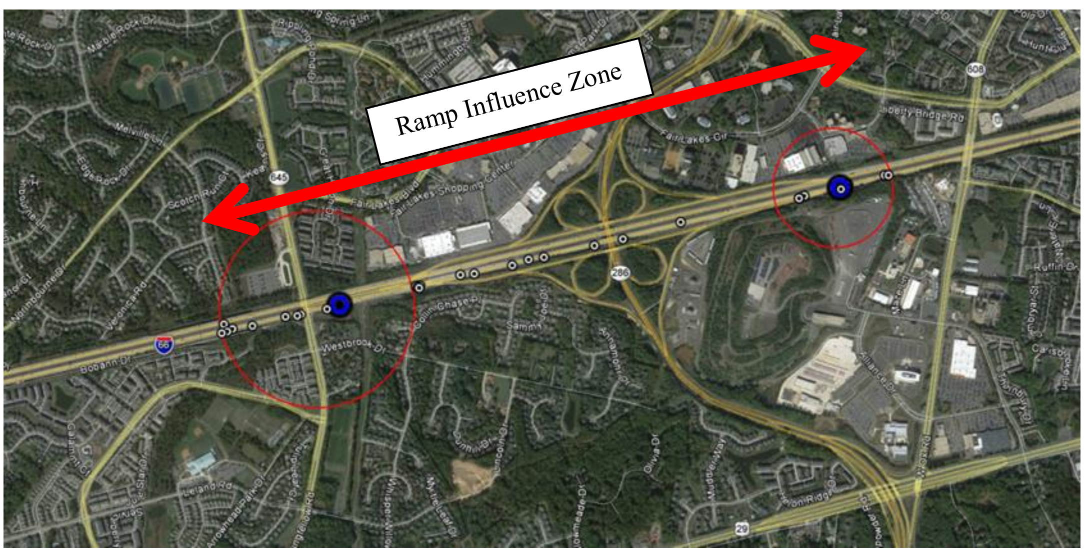


Figure 14. Ramp Influence Zone at EB, VA-286 Interchange

Segment speed drop events were determined by examining the speed differences between adjacent TMC segments at a moment in time. The number of 15-minute periods where the difference in speed between adjacent TMCs exceeded a predefined threshold was counted.

Speed drop thresholds of 10 mph were all examined to determine correlations with crashes. Only speed decreases were analyzed in this paper since the initial findings identified rapid deceleration as one of the more significant causal factors for crashes, and large speed increases were not problematic. The speed drop events represent the number of day occasions when there was more than 10 mph speed drop between two adjacent TMCs in each 15-minute period during 2012-2014. As speed drop events and crash frequency should not be strongly location-dependent, these numbers were total counts in both directions of travel across the six interchanges.

The relationship between speed drop events and the number of rear end and sideswipe crashes were used to develop the average weekday and weekend safety surrogate models using stepwise regression. SPSS was used to develop models using a randomly selected sample of 70% of the data. The remaining 30% of the data were used to validate the model.

There were some assumptions and limitations with the safety surrogate analysis. In some cases, the TMC locations did not line up with the ramp influence zones and some bias could be introduced to the analysis. Most likely, the bias will result in smaller speed changes between TMCs since boundaries would not align with natural speed transition areas. Also, since INRIX is calculating segment speeds using GPS probe data, it represents the space mean speed over a segment of road. The speed differences are between two TMC segments that have different segment lengths, and the space mean speeds are influenced by the length of the TMC segment. The longer the length of the TMC segment, the more difficult it would be to capture localized speed drop locations, as those localized speed drops will be washed out by the other non-affected portions of the TMC segment.

4.4 BENEFIT-COST ANALYSIS

There have been limited quantitative analyses that have examined the operational and safety impacts of U.S. ATM deployments. A benefit-cost ratio was calculated to show the value of the ATM project. This provided valuable information that can be used when assessing the feasibility of implementing additional ATM deployments on other corridors or expanding ATM on I-66 corridor itself.

Benefit-cost analysis was conducted by assigning appropriate monetary value to traffic mobility and safety effects resulting from the implementation of ATM. Values of time from commonly used references were consulted, and crash costs from the Highway Safety Manual were used to monetize safety benefits. Also, differing values of time for freight and passenger vehicles were explicitly considered. These values were combined with the traveler delay analysis and project costs to estimate an overall benefit-cost ratio for the project.

CHAPTER 5: RESULTS

This chapter discusses both the corridor-level and segment-level before-and-after operations and safety analysis of the ATM on I-66. The corridor-level analysis is the focal point of this analysis that evaluates the effectiveness of the ATM system as a whole, with the segment-level analysis supplementing the fact that the HSR was the most beneficial component of the ATM system. For the operations analysis, ATM utilization rate, average travel times, travel time reliability measures, and total travel delay were the performance measures. For the safety analysis, crash rates and estimation of crash frequency using the surrogate measure of speed drop events were the performance measures. Lastly, benefit-cost ratio analysis was conducted to analyze the cost-efficiency of the ATM implementation.

5.1 CORRIDOR-LEVEL OPERATIONS ANALYSIS

5.1.1 Corridor-level Utilization Analysis: HSR

Before ATM was implemented, HSR was only activated during predefined peak periods on weekdays. After ATM implementation, HSR was dynamically opened in response to congestion, in addition to being opened during the regular peak travel times. After-ATM in the EB direction, the average weekday HSR operational hours increased from 5.5 hours/day to 7.99 hours/day per gantry. After-ATM in the WB direction, the average weekday HSR operational hours decreased from 6 hours/day to 5.94 hours/day per gantry. On weekends, EB and WB saw average weekend HSR operational hours increasing to 2.37 hours/day and 2.04 hours/day per gantry, respectively, versus not being opened at all during the before period. It should be noted that these average durations are skewed by the large number of holidays present during the analysis interval. Since HSR is not activated during federal holidays, the average in the WB direction actually declined slightly from the 6 hour baseline from before ATM was activated. This may mean that the increase in the HSR utilization rate may be a conservative value, and the long-term actual HSR utilization rate will be higher.

Some gantries had more hours of HSR activation than others, and these gantries were located on segments with higher AADTs (approximately from milepost 57 to 62 or segment 4 to 5). This is probably not surprising since demand for additional capacity is likely to be highest where volumes are the greatest. Tables 8-11 show the average weekday and weekend HSR utilization results for each gantry and both EB and WB.

Table 8. EB Weekday Before-and-after HSR Utilization by Gantry per Day

EB Weekday HSR Utilization		
Gantry Milepost	**Average Operational Hours - Before (hr/day)**	**Average Operational Hours - After (hr/day)**
58.37	5.50	9.53
58.75	5.50	8.99
59.21	5.50	10.07
59.98	5.50	10.09
60.62	5.50	10.12
61.09	5.50	10.00
61.55	5.50	10.25
62.03	5.50	4.71
62.62	5.50	4.73
63.16	5.50	4.73
63.84	5.50	4.68
Average	**5.50**	**7.99**

Table 9. WB Weekday Before-and-after HSR Utilization by Gantry per Day

WB Weekday HSR Utilization		
Gantry Milepost	**Average Operational Hours - Before (hr/day)**	**Average Operational Hours - After (hr)**
59.42	6.00	7.07
60.01	6.00	7.13
60.9	6.00	7.15
61.27	6.00	7.13
61.59	6.00	8.05
62.08	6.00	6.73
62.62	6.00	3.37
63.16	6.00	3.39
63.84	6.00	3.43
Average	**6.00**	**5.94**

Table 10. EB Weekend Before-and-after HSR Utilization by Gantry per Day

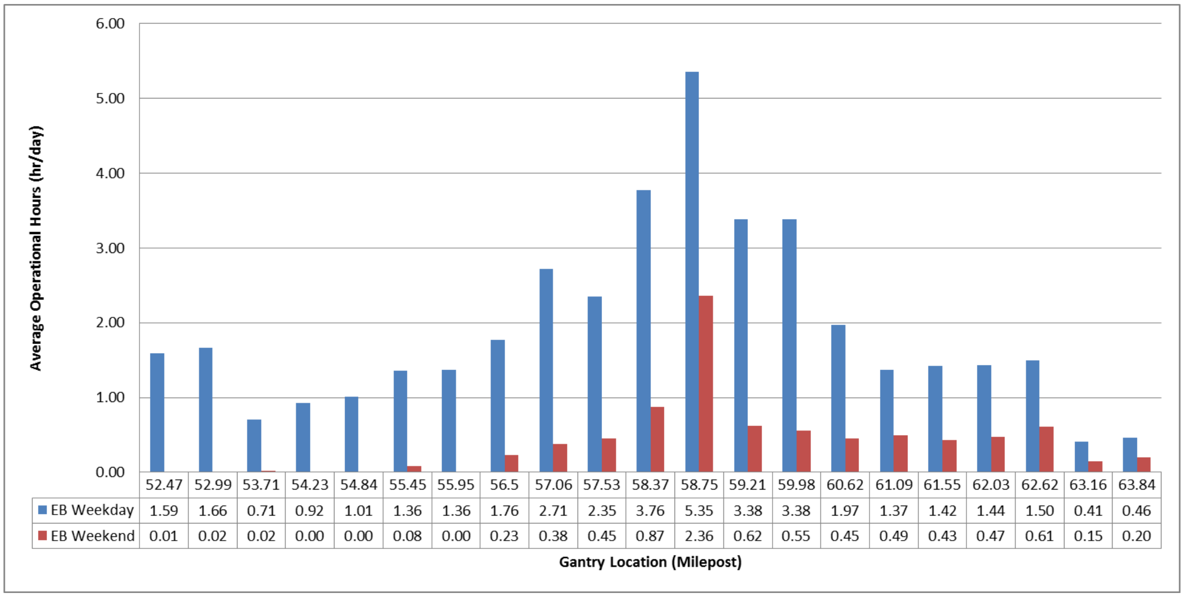
EB Weekend HSR Utilization		
Gantry Milepost	Average Operational Hours - Before (hr/day)	Average Operational Hours - After (hr/day)
58.37	0.00	2.68
58.75	0.00	2.66
59.21	0.00	2.84
59.98	0.00	2.80
60.62	0.00	2.87
61.09	0.00	4.26
61.55	0.00	2.87
62.03	0.00	1.26
62.62	0.00	1.26
63.16	0.00	1.27
63.84	0.00	1.25
Average	**0.00**	**2.37**

Table 11. WB Weekend Before-and-after HSR Utilization by Gantry per Day

WB Weekend HSR Utilization		
Gantry Milepost	Average Operational Hours - Before (hr/day)	Average Operational Hours - After (hr/day)
59.42	0.00	2.58
60.01	0.00	2.58
60.9	0.00	2.58
61.27	0.00	2.58
61.59	0.00	2.44
62.08	0.00	2.45
62.62	0.00	1.06
63.16	0.00	1.06
63.84	0.00	1.06
Average	**0.00**	**2.04**

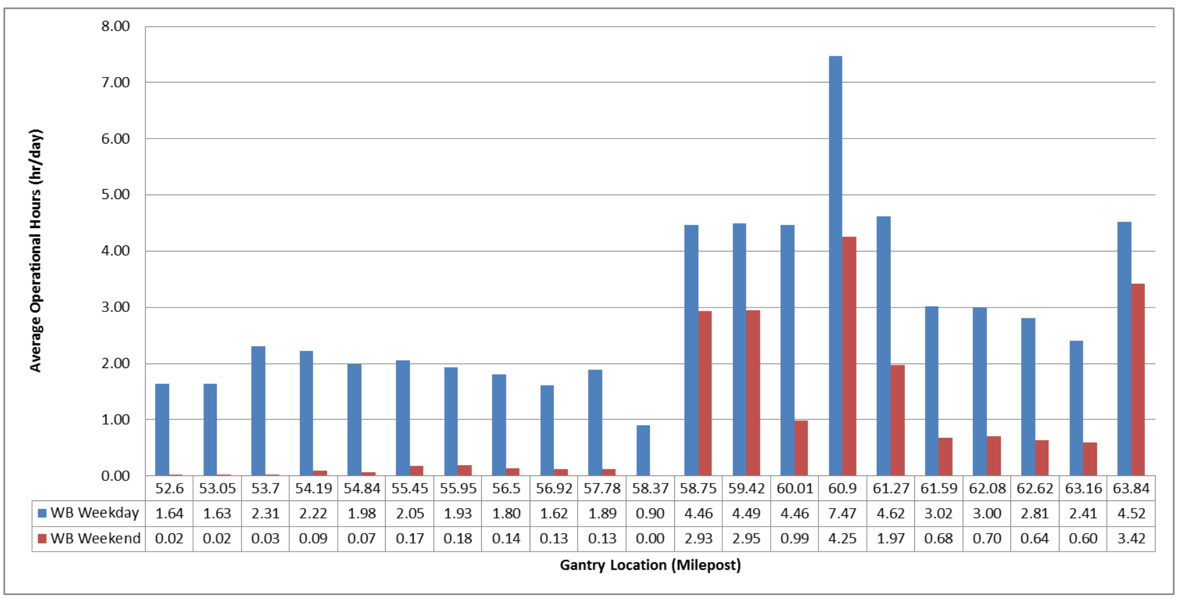
5.1.2 Corridor-level Utilization Analysis: VSL

There are more gantries that are operating the VSL than HSR since VSL is operated on all of the study segments of the I-66 corridor. Since VSLs with enhanced algorithms were re-activated in mid-January, only mid-January to February data were analyzed for the VSL utilization analysis. The VSLs are activated whenever the system detects slowdowns in traffic in order to smooth flow into a reduced speed zone. The average weekday VSL operational durations for EB and WB were 1.90 hours and 2.92 hours respectively. The average weekend VSL operation hours for EB and WB were 0.40 hours and 0.96 hours respectively. Like HSR, some gantries had more hours of VSL activation than others, and these gantries were located on segments with higher AADTs (approximately from milepost 57 to 62 or segment 4 to 5). Figures 15 and 16 show the average weekday and weekend VSL utilization results for each gantry.



	52.47	52.99	53.71	54.23	54.84	55.45	55.95	56.5	57.06	57.53	58.37	58.75	59.21	59.98	60.62	61.09	61.55	62.03	62.62	63.16	63.84
EB Weekday	1.59	1.66	0.71	0.92	1.01	1.36	1.36	1.76	2.71	2.35	3.76	5.35	3.38	3.38	1.97	1.37	1.42	1.44	1.50	0.41	0.46
EB Weekend	0.01	0.02	0.02	0.00	0.00	0.08	0.00	0.23	0.38	0.45	0.87	2.36	0.62	0.55	0.45	0.49	0.43	0.47	0.61	0.15	0.20

Figure 15. EB VSL Utilization by Gantry for Weekday and Weekend



	52.6	53.05	53.7	54.19	54.84	55.45	55.95	56.5	56.92	57.78	58.37	58.75	59.42	60.01	60.9	61.27	61.59	62.08	62.62	63.16	63.84
WB Weekday	1.64	1.63	2.31	2.22	1.98	2.05	1.93	1.80	1.62	1.89	0.90	4.46	4.49	4.46	7.47	4.62	3.02	3.00	2.81	2.41	4.52
WB Weekend	0.02	0.02	0.03	0.09	0.07	0.17	0.18	0.14	0.13	0.13	0.00	2.93	2.95	0.99	4.25	1.97	0.68	0.70	0.64	0.60	3.42

Figure 16. WB VSL Utilization by Gantry per Weekday and Weekend

Figure 17 shows the distribution of reduced speeds that were posted on the VSLs when they were active based on duration of the display. For weekdays, the percentages of total time when the gantries indicated 35, 40, 45, and 50 mph speed reduction were very similar to each other, each ranging from 20 to 30% for both EB and WB. For EB and WB weekends, the percentages of total time when the gantries indicated 50 mph was the highest at 47% and 64% for EB and WB directions respectively. What this may represent is that during weekends, the traffic flow is at good enough conditions for the average speeds to remain high, and there weren't many time periods that required VSLs to show 45 mph or lower. This reflects the better quality of flow that was generally present on the weekends. However, during weekdays, the traffic flow is not as good, therefore the VSLs are posting lower speeds going into the congested areas of the highway.

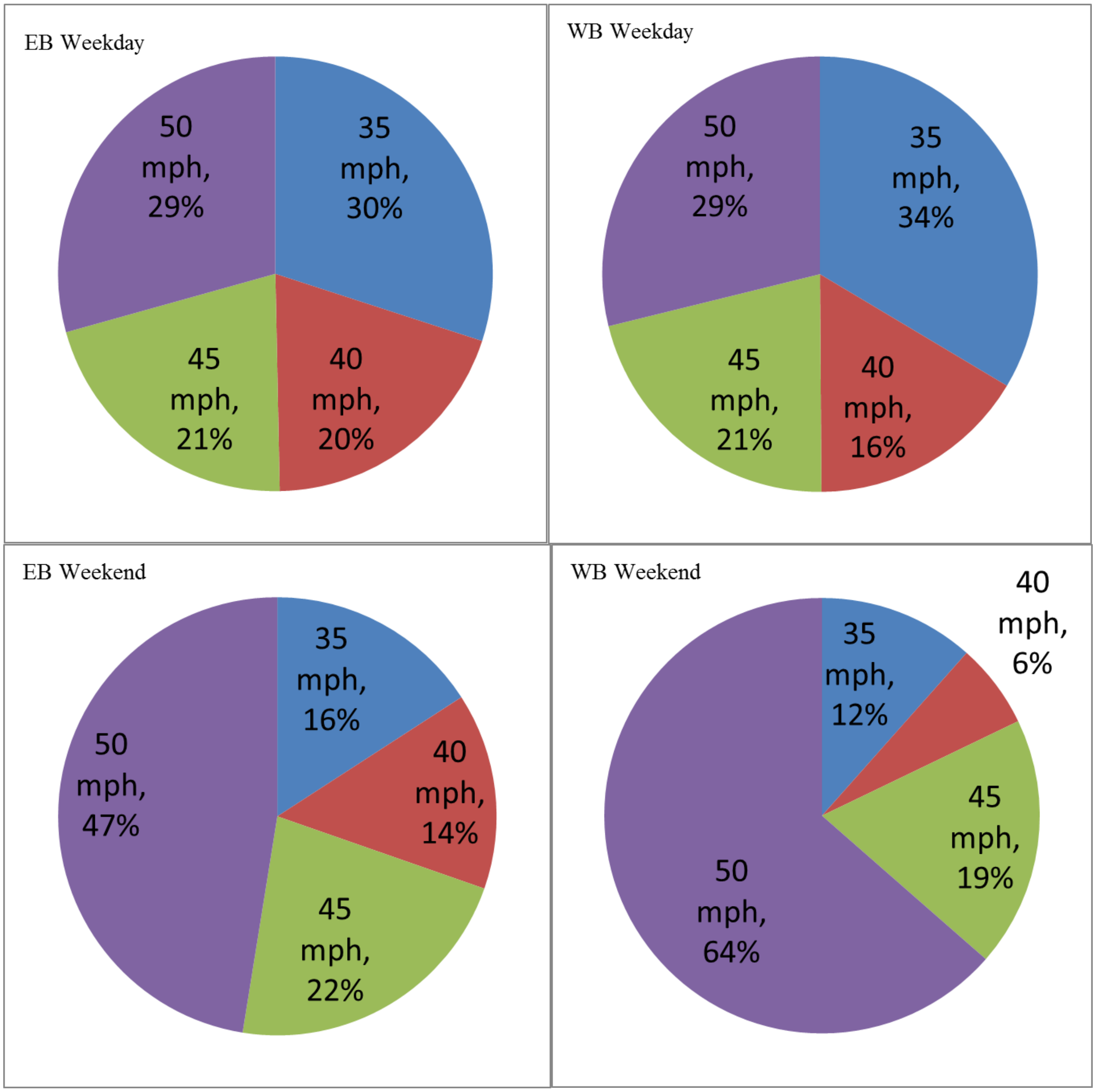


Figure 17. Speed Reduction Sign Type Utilization for EB and WB Weekdays and Weekends

5.1.3 Corridor-level Utilization Analysis: LUCS

LUCS were activated when there were lane blockages on the corridor due to incidents, crashes, or work zones. On the problematic lanes, the electronic message signs showed diagonal yellow arrow signs that were used to reroute the regular traffic into the open lanes. Red “X” indications were then used to indicate closed lanes. Since LUCS are activated only when there is a problem on the roadway (e.g. disabled vehicle, crashes, work zones), it does not activate as frequently as HSR or VSL. Since electronic message signs on each lane can be activated or deactivated independently to appropriately mitigate non-recurrent congestion events, the utilization of the LUCS were analyzed by individual lane per gantry. The full LUCS utilization results are shown on Tables 12-15. As expected, the total duration and instances of LUCS activation for EB and WB weekday and weekend periods were low. While the TOC operators have anecdotally indicated that the LUCS have provided some incident management benefits, they are not operated very frequently as compared to the VSL or HSR. As a result, it is difficult to assign specific benefits to these systems. A parallel VTRC study is currently investigating the microscopic benefits of the LUCS in a more detailed manner.

Table 12. EB Weekday Total Duration and Instances of LUCS Activation

EB Weekday LUCs Utilization								
Gantry Milepost	Sign on Lane 1 Utilization		Sign on Lane 2 Utilization		Sign on Lane 3 Utilization		Sign on Lane 4 Utilization	
	Total Minutes of Operation	Total Number of Activation	Total Minutes of Operation	Total Number of Activation	Total Minutes of Operation	Total Number of Activation	Total Minutes of Operation	Total Number of Activation
52.47	14.85	1	15.88	1	0.00	0	2.32	1
52.99	0.00	0	1.03	1	24.12	1	43.15	1
53.71	0.00	0	0.00	0	51.00	1	61.22	2
54.23	14.52	1	0.00	0	31.20	4	147.83	6
54.84	38.80	2	21.10	2	0.00	0	0.00	0
55.45	21.33	1	0.00	0	2.55	1	7.28	2
55.95	29.18	4	3.97	1	5.23	2	9.28	4
56.5	35.52	3	0.00	0	2.80	2	2.80	2
57.06	29.40	3	19.80	3	1.60	1	0.00	0
57.53	128.75	5	37.37	1	73.65	3	131.78	3
58.37	4.47	1	28.75	4	49.95	4	125.22	9
58.75	80.73	4	96.73	6	72.88	5	146.98	8
59.21	232.48	10	75.25	8	58.50	6	135.87	11
59.98	86.37	4	38.50	4	0.00	0	17.87	4
60.62	15.33	2	38.80	3	312.50	6	157.95	11
61.09	55.48	5	41.23	4	296.53	4	350.10	17
61.55	14.97	2	8.95	1	52.03	4	64.10	4
62.03	72.05	3	1.87	2	22.20	6	16.58	4
62.62	2.38	2	37.25	4	49.63	5	18.30	3
63.16	0.12	1	47.82	5	190.02	10	416.42	13
63.84	0.35	2	11.08	4	11.20	4	74.63	2
Total	**877.08**	**56**	**525.38**	**54**	**1307.60**	**69**	**1929.68**	**107**

Table 13. WB Weekday Total Duration and Instances of LUCS Activation

WB Weekday LUCs Utilization								
Gantry Milepost	Sign on Lane 1 Utilization		Sign on Lane 2 Utilization		Sign on Lane 3 Utilization		Sign on Lane 4 Utilization	
	Total Minutes of Operation	Total Number of Activation	Total Minutes of Operation	Total Number of Activation	Total Minutes of Operation	Total Number of Activation	Total Minutes of Operation	Total Number of Activation
52.47	0.00	0	0.00	0	0.00	0	0.00	0
52.99	0.00	0	28.82	1	2.93	1	0.00	0
53.71	42.62	1	0.00	0	0.00	0	0.00	0
54.23	113.23	9	90.38	6	40.95	4	12.27	3
54.84	93.13	4	99.75	3	46.75	2	74.13	3
55.45	5.48	3	0.00	0	0.00	0	240.03	3
55.95	68.35	5	4.38	2	3.02	1	11.10	1
56.5	35.87	2	45.38	2	40.52	2	38.88	1
57.06	9.52	1	152.13	3	105.72	2	0.00	0
57.53	75.40	1	32.93	1	0.00	0	0.00	0
58.37	84.78	2	68.37	2	6.22	1	0.00	0
58.75	50.30	1	0.00	0	0.00	0	31.37	1
59.21	51.55	3	0.00	0	0.00	0	2.73	1
59.98	32.35	2	472.02	1	2.02	1	148.25	7
60.62	82.47	2	0.00	0	29.28	1	58.95	5
61.09	130.37	4	43.72	2	0.00	0	54.60	2
61.55	14.00	1	0.00	0	44.47	1	119.63	4
62.03	141.48	1	0.00	0	0.00	0	46.93	6
62.62	75.82	4	30.00	5	59.15	5	70.45	4
63.16	63.17	6	0.42	3	0.38	3	50.42	6
63.84	11.38	3	36.38	8	37.08	8	271.07	6
Total	**1181.27**	**55**	**1104.68**	**39**	**418.48**	**32**	**1230.82**	**53**

Table 14. EB Weekend Total Duration and Instances of LUCS Activation

EB Weekend LUCs Utilization								
	Sign on Lane 1 Utilization		Sign on Lane 2 Utilization		Sign on Lane 3 Utilization		Sign on Lane 4 Utilization	
Gantry Milepost	Total Minutes of Operation	Total Number of Activation	Total Minutes of Operation	Total Number of Activation	Total Minutes of Operation	Total Number of Activation	Total Minutes of Operation	Total Number of Activation
52.47	0.00	0	29.93	1	42.20	1	67.17	1
52.99	0.00	0	0.00	0	0.00	0	4.77	1
53.71	0.00	0	0.00	0	0.00	0	46.47	3
54.23	0.00	0	0.00	0	0.93	1	36.37	1
54.84	0.00	0	0.00	0	32.95	1	32.95	1
55.45	0.00	0	0.00	0	32.95	1	0.00	0
55.95	0.00	0	0.00	0	0.00	0	0.33	1
56.5	0.00	0	0.00	0	0.00	0	42.52	1
57.06	0.00	0	0.00	0	0.00	0	0.00	0
57.53	0.00	0	0.00	0	0.00	0	0.00	0
58.37	0.00	0	0.00	0	0.00	0	46.23	2
58.75	9.47	1	0.00	0	65.25	1	0.00	0
59.21	2.07	1	0.00	0	0.00	0	1.12	1
59.98	9.62	2	10.30	2	4.37	1	0.00	0
60.62	7.83	1	0.00	0	0.00	0	0.00	0
61.09	44.28	1	71.45	2	84.57	4	75.85	4
61.55	3.07	1	26.45	1	40.12	3	20.92	4
62.03	11.22	1	25.90	1	25.90	1	7.32	1
62.62	84.30	1	147.75	2	0.00	0	0.00	0
63.16	0.00	0	0.00	0	0.00	0	89.68	2
63.84	0.00	0	0.00	0	0.00	0	0.00	0
Total	**171.85**	**9**	**311.78**	**9**	**329.23**	**14**	**471.68**	**23**

Table 15. WB Weekend Total Duration and Instances of LUCS Activation

WB Weekend LUCs Utilization								
Gantry Milepost	Sign on Lane 1 Utilization		Sign on Lane 2 Utilization		Sign on Lane 3 Utilization		Sign on Lane 4 Utilization	
	Total Minutes of Operation	Total Number of Activation	Total Minutes of Operation	Total Number of Activation	Total Minutes of Operation	Total Number of Activation	Total Minutes of Operation	Total Number of Activation
52.47	0.00	0	0.00	0	0.00	0	0.00	0
52.99	13.32	1	13.32	1	0.00	0	0.00	0
53.71	0.00	0	0.00	0	0.00	0	0.00	0
54.23	10.93	2	7.22	2	0.00	0	0.00	0
54.84	15.48	1	6.23	1	64.28	1	93.88	2
55.45	0.00	0	0.00	0	36.17	2	43.00	2
55.95	0.00	0	0.00	0	0.00	0	0.00	0
56.5	0.00	0	0.00	0	0.00	0	0.00	0
57.06	0.00	0	3.25	1	0.00	0	0.00	0
57.53	0.00	0	10.92	1	0.00	0	0.00	0
58.37	0.00	0	1.18	1	0.00	0	0.00	0
58.75	0.00	0	4.20	1	0.00	0	22.98	1
59.21	0.00	0	0.00	0	0.00	0	0.00	0
59.98	64.08	1	0.00	0	0.52	1	15.48	3
60.62	73.85	1	0.00	0	0.00	0	0.00	0
61.09	0.00	0	0.00	0	0.00	0	0.00	0
61.55	73.72	1	0.00	0	0.00	0	0.00	0
62.03	2.38	1	0.00	0	0.00	0	0.00	0
62.62	0.00	0	0.00	0	0.00	0	0.00	0
63.16	0.00	0	0.00	0	0.00	0	0.00	0
63.84	0.00	0	0.00	0	0.00	0	0.00	0
Total	**253.77**	**8**	**46.32**	**8**	**100.97**	**4**	**175.35**	**8**

5.1.4 Weekday Corridor-level Average Travel Time Analysis

For weekday average travel times, there were small, but statistically significant degradations at $\alpha = 0.05$ between after-ATM and before-ATM average times during peak periods while traveling in the peak directions (AM for EB, PM for WB). For the EB AM peak period, weekday average travel times increased from 17.03 minutes to 18.19 minutes (6.80% increase) and for the WB PM peak period, weekday average travel times increased from 21.65 minutes to 22.54 minutes (4.12% increase). This trend was generally consistent across most days of the week (Mon – Fri). This increase in weekday average travel times during peak periods was expected, as peak period weekday average travel time profiles for both EB and WB, shown on Figures 18 and 20, have been generally increasing during the past 3 years of before-ATM periods, from 2012-2015 (Oct – Feb only). Table 16 shows the general increasing trend of the average travel times for several years prior to the ATM implementation for average weekdays. The WB Peak average travel time trend was that prior to the implementation of ATM, the average travel time percent changes continually increased (deteriorated). However, after the implementation of ATM, the percent change increased, but the magnitude of it had decreased. For EB, the case was not strong as the average travel time percent change between Oct '13 - Feb '14 and Oct '14 - Feb '15 improved, possibly due to the impact of the opening of the Metro Silver Line which may have removed traffic from I-66 that previously access Metro in Vienna.

Table 16. Average Travel Time Percent Changes for 2012-2016 (Oct – Feb Only, Peak Directions)

Average Travel Time % Changes for 2012-2016 (Oct - Feb Only)			
Direction	Weekday		
	Oct '12 - Feb '13 to Oct '13 - Feb '14	Oct '13 - Feb '14 to Oct '14 - Feb '15	Oct '14 - Feb '15 to Oct '15 - Feb '16 (After-ATM)
EM Peak (AM)	3.2%	-1.1%	6.8%
WB PM Peak (PM)	5.2%	7.1%	4.1%

Also in this case, the shoulders were already open to travel in the peak direction before the ATM system was deployed, the ATM system did not offer any additional capacity beyond what was already in use during pre-ATM conditions.

Figures 19 and 21 show the EB and WB corridor-level average travel time profiles on average weekdays for the before-and-after ATM periods. The error bars represent the confidence interval of the average travel time values at the 95% confidence level. Reliability measures will be discussed in more detail later in this thesis.

For the off-peak directions (PM for EB, AM for WB), there were statistically significant improvements in weekday average travel times even though the off-peak period weekday average travel times for both EB and WB have been increasing during the past 3 years of before-ATM periods. For the EB PM off-peak period, weekday average travel times decreased from 14.66 minutes to 13.73 minutes (6.35% improvement) and for the WB AM off-peak period, average weekday average travel times decreased from 12.57 minutes to 12.29 minutes (2.20% improvement). For the midday transition period, there were also small, but statistically significant improvements in weekday average travel times in both EB and WB directions. For the EB midday period, average weekday travel times decreased from 13.31 minutes to 13.16 minutes (1.17% improvement) and for the WB midday period, average weekday average travel times decreased from 13.33 minutes to 12.70 minutes (4.66% improvement). For these off peak and midday transition periods when the roadway was not operating at maximum capacity, the dynamic opening of the shoulders may have contributed to faster travel times along the corridor and mitigated any incident and non-recurring congestion impacts. The improvements in weekday average travel times were generally consistent across the weekday days for both off-peak and midday transition periods. Once again, the reductions observed in the off peak and midday periods represented a reversal from the year-over-year increases that were observed in the 3 years prior to ATM activation.

The weekday average travel time changes during the overnight period were negligible as average travel times were free-flow for both before and after conditions. The full average weekday average travel time results are shown on Tables 17 and 18.

Table 17. Weekday Before-and-after Average Travel Time Comparisons (Entire Corridor) – AM and PM Peaks

Weekday		Average Travel Times (min)									
Direction	Day	AM Peak Period (5:30am - 11am) Oct '14 - Feb '15	AM Peak Period (5:30am - 11am) Oct '15 - Feb '16	Change in AM Peak Period (#)	Change in AM Peak Period (%)	Statistical Significance at α = 0.05	PM Peak Period (2pm - 8pm) Oct '14 - Feb '15	PM Peak Period (2pm - 8pm) Oct '15 - Feb '16	Change in PM Peak Period (#)	Change in PM Peak Period (%)	Statistical Significance at α = 0.05
EB	Mon	16.370	17.068	0.698	4.264	Sig (<0.05)	14.873	13.848	1.025	6.892	Sig (<0.05)
	Tues	18.176	20.178	2.002	11.015	Sig (<0.05)	14.084	13.119	0.965	6.852	Sig (<0.05)
	Wed	17.620	18.806	1.186	6.731	Sig (<0.05)	13.718	14.911	1.193	8.697	Sig (<0.05)
	Thurs	18.751	19.855	1.104	5.888	Sig (<0.05)	15.001	13.105	1.896	12.639	Sig (<0.05)
	Fri	14.316	14.959	0.643	4.491	Sig (<0.05)	15.580	13.663	1.917	12.304	Sig (<0.05)
	Average	**17.034**	**18.192**	1.158	6.798	**Sig (<0.05)**	**14.656**	**13.725**	0.931	6.352	**Sig (<0.05)**
WB	Mon	12.331	12.282	0.049	0.397	Not Sig (0.324)	20.392	22.194	1.802	8.837	Sig (<0.05)
	Tues	13.118	12.415	0.703	5.359	Sig (<0.05)	20.202	22.538	2.336	11.563	Sig (<0.05)
	Wed	12.868	12.340	0.528	4.103	Sig (<0.05)	21.773	23.028	1.255	5.764	Sig (<0.05)
	Thurs	12.454	12.193	0.261	2.096	Sig (<0.05)	23.227	22.769	0.458	1.972	Not Sig (0.155)
	Fri	12.095	12.220	0.125	1.033	Not Sig (0.094)	22.624	22.179	0.445	1.967	Not Sig (0.137)
	Average	**12.567**	**12.290**	0.277	2.204	**Sig (<0.05)**	**21.653**	**22.544**	0.891	4.115	**Sig (<0.05)**

Table 18. Weekday Before-and-after Average Travel Time Comparisons (Entire Corridor) – Midday and Overnight

Weekday		Average Travel Times (min)									
Direction	Day	Midday Period (11am - 2pm) Oct '14 - Feb '15	Midday Period (11am - 2pm) Oct '15 - Feb '16	Change in Midday Period (#)	Change in Midday Period (%)	Statistical Significance at α = 0.05	Overnight Period (8pm - 5:30am) Oct '14 - Feb '15	Overnight Period (8pm - 5:30am) Oct '15 - Feb '16	Change in Overnight Period (#)	Change in Overnight Period (%)	Statistical Significance at α = 0.05
EB	Mon	14.093	12.946	1.147	8.139	Sig (<0.05)	12.240	12.476	0.236	1.928	Sig (<0.05)
	Tues	13.039	13.050	0.011	0.084	Not Sig (0.393)	12.194	12.777	0.583	4.781	Sig (<0.05)
	Wed	12.788	13.380	0.592	4.629	Sig (<0.05)	12.174	13.583	1.409	11.574	Sig (<0.05)
	Thurs	13.766	13.410	0.356	2.586	Not Sig (0.085)	12.283	12.593	0.310	2.524	Sig (<0.05)
	Fri	12.862	12.987	0.125	0.972	Not Sig (0.166)	12.283	12.395	0.112	0.912	Not Sig (0.128)
	Average	**13.312**	**13.156**	0.156	1.172	**Sig (<0.05)**	**12.238**	**12.768**	0.530	4.331	**Sig (<0.05)**
WB	Mon	12.554	12.459	0.095	0.757	Not Sig (0.329)	12.363	12.292	0.071	0.574	Sig (<0.05)
	Tues	13.390	12.512	0.878	6.557	Sig (<0.05)	12.483	12.450	0.033	0.264	Not Sig (0.205)
	Wed	14.361	12.503	1.858	12.938	Sig (<0.05)	12.240	12.589	0.349	2.851	Sig (<0.05)
	Thurs	12.466	12.873	0.407	3.265	Sig (<0.05)	12.556	12.380	0.176	1.402	Sig (<0.05)
	Fri	13.853	13.193	0.660	4.764	Sig (<0.05)	12.399	12.250	0.149	1.202	Sig (<0.05)
	Average	**13.325**	**12.704**	0.621	4.660	**Sig (<0.05)**	**12.410**	**12.390**	0.020	0.161	**Not Sig (0.202)**

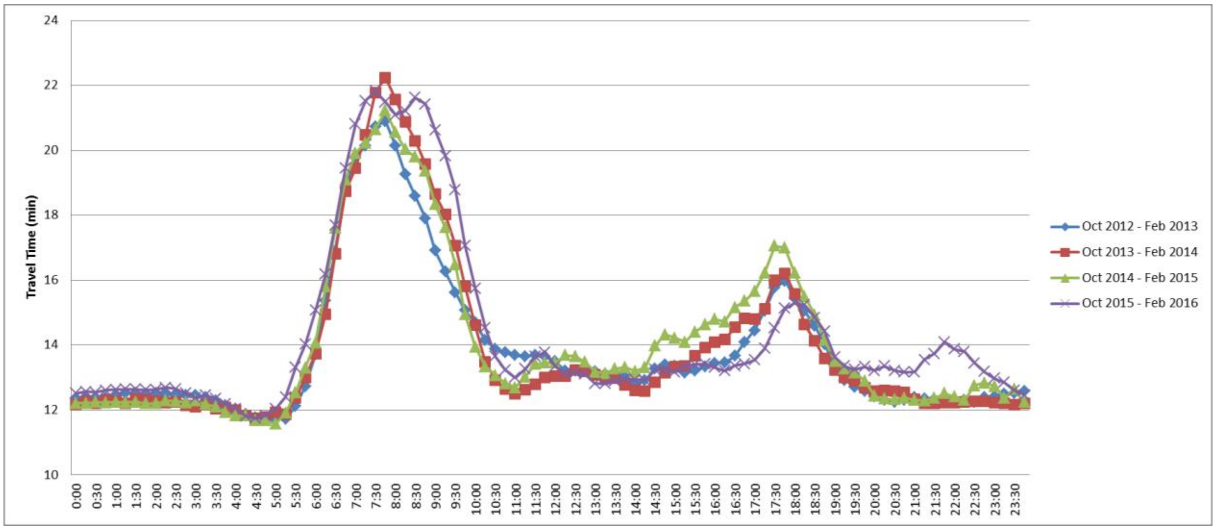


Figure 18: EB Average Weekday Average Travel Time Trend (Corridor-level)

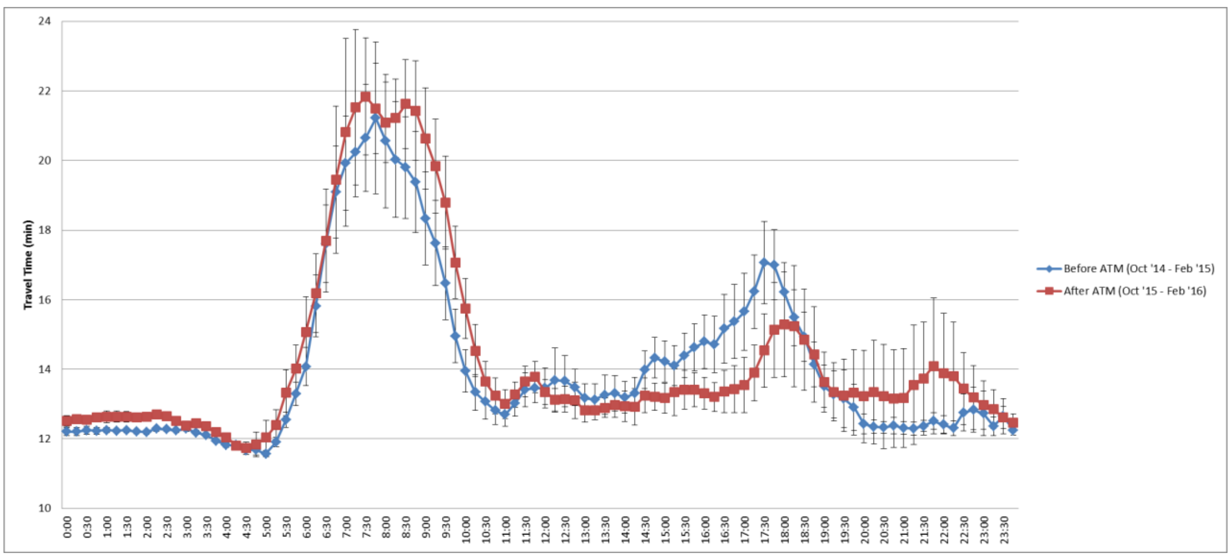


Figure 19. EB Before-and-After Average Weekday Average Travel Time Profile (Corridor-level)

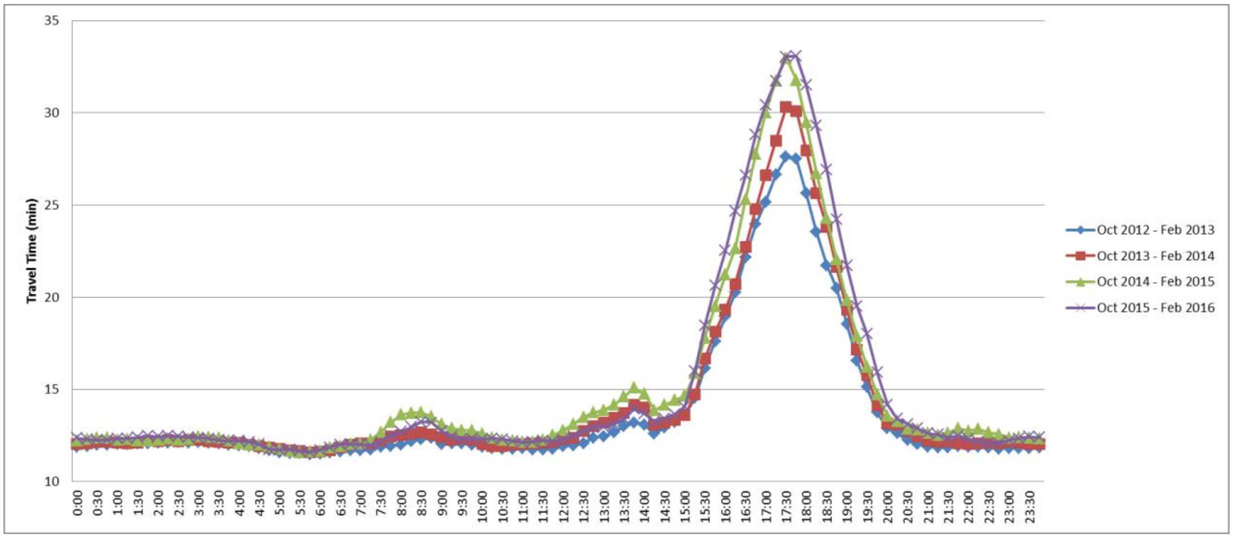


Figure 20: WB Average Weekday Average Travel Time Trend (Corridor-level)

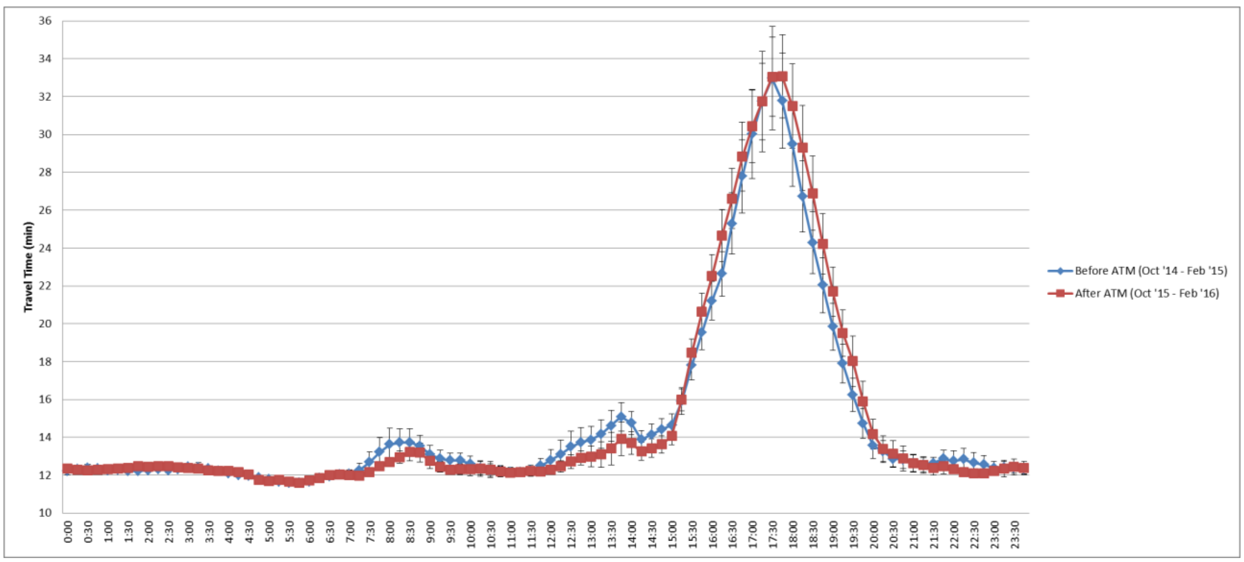


Figure 21. WB Before-and-After Average Weekday Average Travel Time Profile (Corridor-level)

5.1.5 Weekend Corridor-level Average Travel Time Analysis

The weekend peak period is significantly different than that of the weekday peak periods for both EB and WB directions. Table 19 shows that for both the EB and WB weekend peak period, there were statistically significant improvements in travel times. For the EB direction, the weekend average travel times were reduced from 14.53 minutes to 13.06 minutes (10.13% improvement). In the WB direction, the average weekend travel times were reduced from 13.71 minutes to 12.25 minutes (10.66% improvement). These improvements were both statistically significant. Before the ATM system was implemented, the shoulders were not used during the weekends even if there was demand for increased roadway capacity. However, after the ATM system was implemented, shoulders were being opened for travel on the weekend whenever demands for additional capacity were warranted. This additional roadway capacity brought on by the HSR likely contributed to the improvements in travel times along the corridor. For both the EB and WB directions, the travel times now often approach free-flow during the weekend peak periods, which can be seen from the yearly weekend average travel time trends on Figures 22 and 24. The improvements in weekend average travel times were consistent across the weekend days for both peak and off-peak periods.

The weekend average travel time changes during overnight off-peak period are negligible, as average travel times were already free-flow for both before and after conditions.

Figures 23 to 25 show the corridor-level average travel time profiles on average weekends for the before-and-after ATM periods. The error bars represent the confidence interval of the average travel time values at the 95% confidence level. Visual analysis showed strong evidence that the confidence intervals tightened during the average weekday conditions after the implementation of the ATM. The tightened confidence intervals represent less variance of travel times, indicating more reliable trips. This is discussed in more detail later in this thesis.

Table 19. Weekend Before-and-after Average Travel Time Comparisons (Entire Corridor) – Peak and Off-peak

Weekend		Average Travel Times (min)									
Direction	**Day**	**Peak Period (10am - 8pm) Oct '14 - Feb '15**	**Peak Period (10am - 8pm) Oct '15 - Feb '16**	**Change in Peak Period (#)**	**Change in Peak Period (%)**	**Statistical Significance at α = 0.05**	**Off-Peak Period (8pm - 10am) Oct '14 - Feb '15**	**Off-Peak Period (8pm - 10am) Oct '15 - Feb '16**	**Change in Off-Peak Period (#)**	**Change in Off-Peak Period (%)**	**Statistical Significance at α = 0.05**
EB	Sun	13.617	12.663	0.954	7.006	Sig (<0.05)	12.068	12.351	0.283	2.345	Sig (<0.05)
	Sat	15.487	13.481	2.006	12.953	Sig (<0.05)	12.128	12.227	0.099	0.816	Sig (<0.05)
	Average	**14.534**	**13.062**	1.472	10.128	**Sig (<0.05)**	**12.098**	**12.287**	0.189	1.562	**Sig (<0.05)**
WB	Sun	12.460	11.971	0.489	3.925	Sig (<0.05)	12.000	12.076	0.076	0.633	Not Sig (0.115)
	Sat	14.991	12.544	2.447	16.323	Sig (<0.05)	11.988	12.043	0.055	0.459	Sig (<0.05)
	Average	**13.710**	**12.249**	1.461	10.656	**Sig (<0.05)**	**11.995**	**12.055**	0.060	0.500	**Sig (<0.05)**

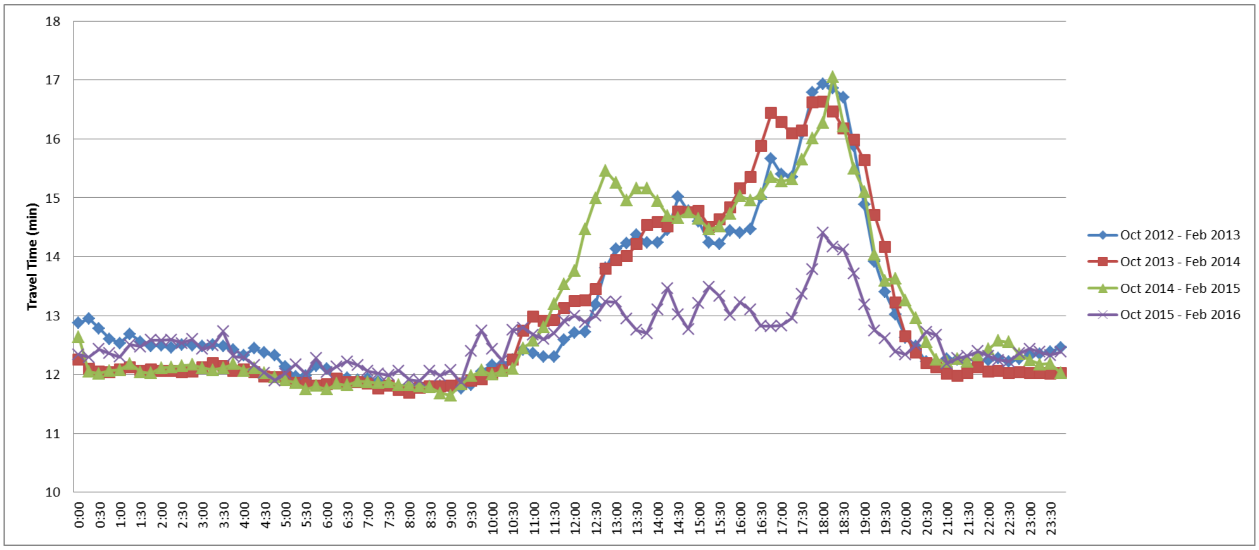


Figure 22: EB Average Weekend Average Travel Time Trend (Corridor-level)

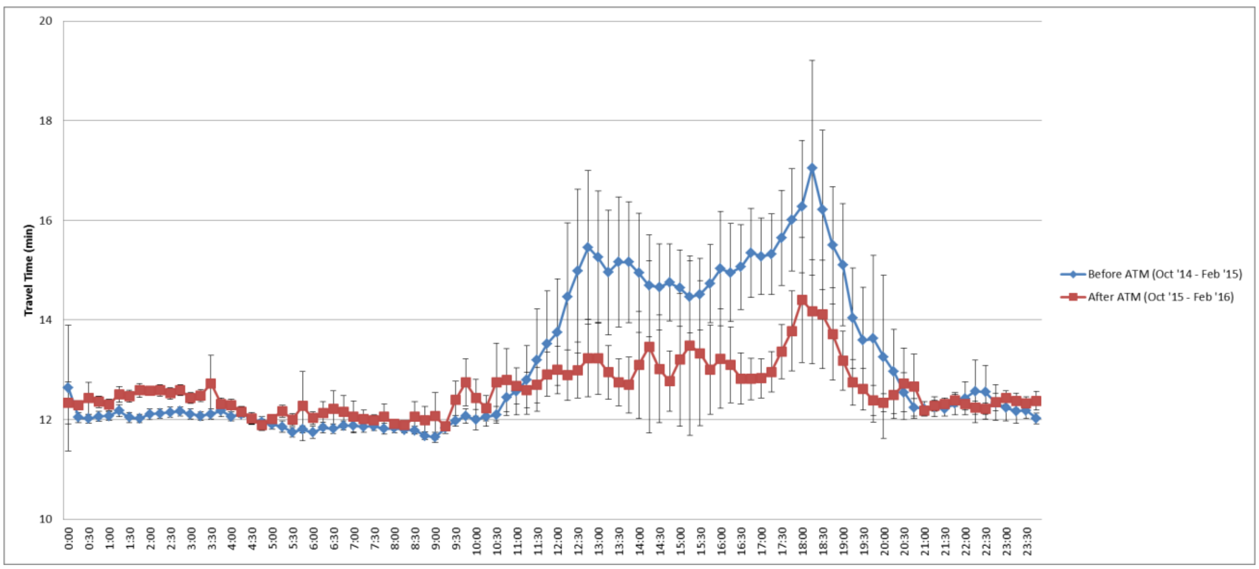


Figure 23. EB Before-and-After Average Weekend Average Travel Time Profile (Corridor-level)

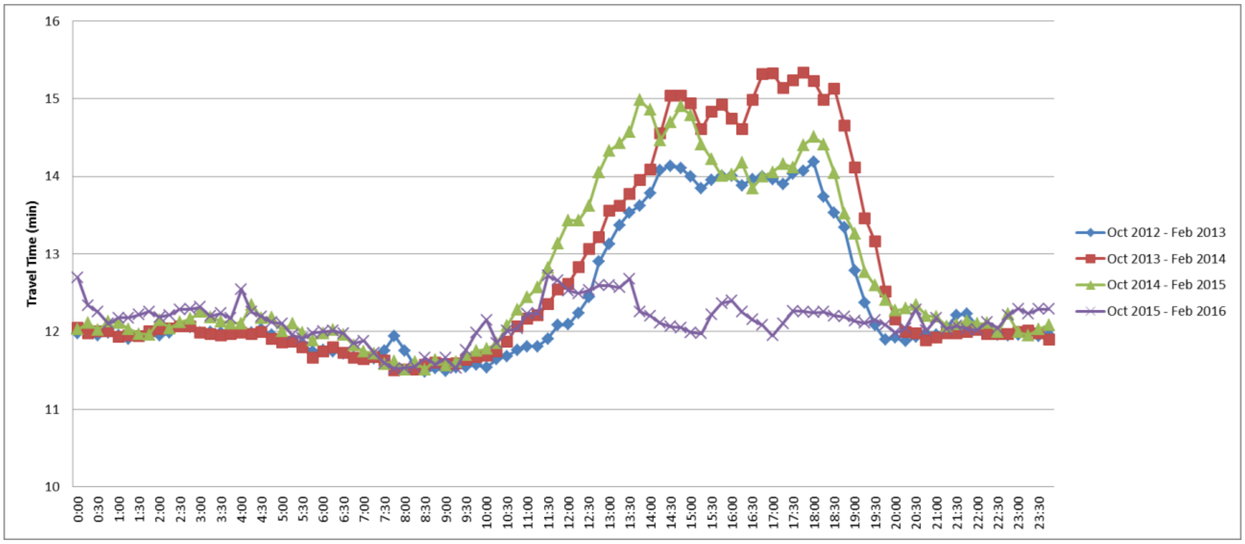


Figure 24: WB Average Weekend Average Travel Time Trend (Corridor-level)

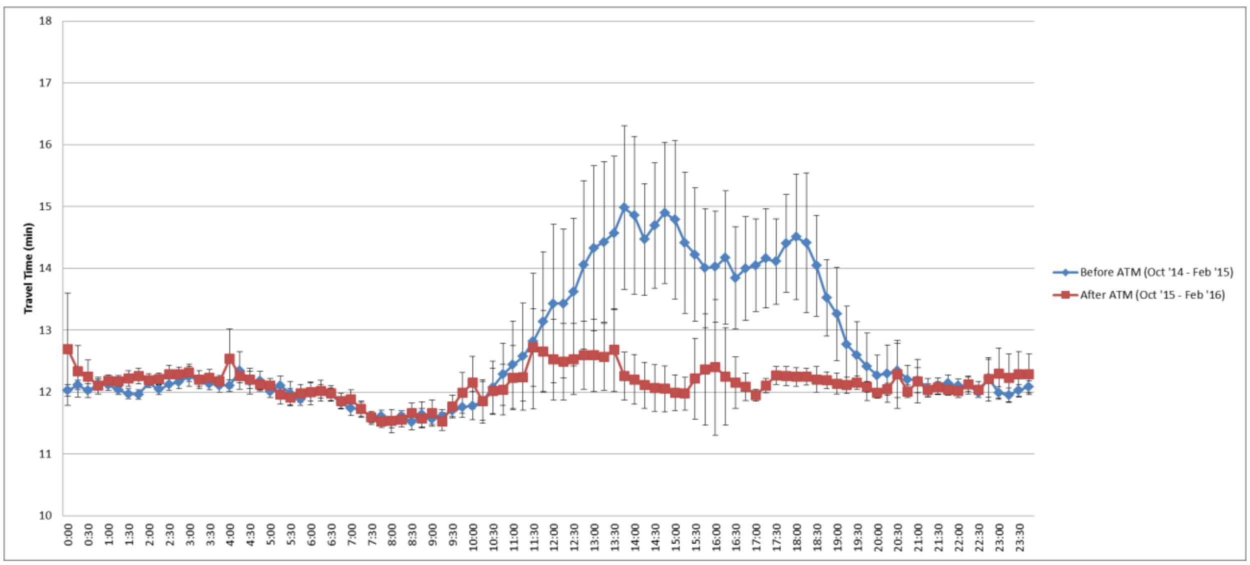


Figure 25. WB Before-and-After Average Weekend Average Travel Time Profile (Corridor-level)

5.1.6 Weekday Corridor-level Travel Time Reliability Analysis

The travel time reliability results were similar to the average travel time results for the respective peak, midday, off-peak, and overnight periods. For the EB AM peak period, average weekday PTI and BI deteriorated by 0.10 (7.48%) and 0.01 (13.33%) respectively. For the WB PM peak period, average weekday PTI and BI deteriorated by 0.07 (3.81%) and <0.01 (3.45%) respectively. The changes, while mostly small, were statistically significant at $\alpha = 0.05$. This trend was consistent across most days of the week. These results were to be expected as average travel times deteriorate, travel time reliability would deteriorate as greater congestion creates less reliable conditions.

Generally for the off-peak directions (PM for EB, AM for WB), there were statistically significant improvements in PTI and BI, which is expected as the average travel times improved for off-peak directions. For the EB PM off-peak period, average weekday PTI improved by 0.06 (5.45%), but average weekday BI deteriorated by 0.01 (17.65%). For the WB AM off-peak period, average weekday PTI and BI improved by 0.03 (3.33%) and 0.01 (36.67%) respectively. For the midday transition period, there were also small, but statistically significant improvements in average PTI and BI. For the EB midday period, average weekday PTI and BI improved by 0.02 (2.25%) and 0.01 (28.21%) respectively. For the WB midday period, average weekday PTI and BI improved by 0.06 (5.62%) and 0.01 (25.00%) respectively. The magnitudes of the off-peak and midday travel time reliability changes were minimal or practically insignificant as the PTI values were close to 1 or less than 1 during these time periods for both EB and WB.

The average weekday PTI and BI changes during overnight period are negligible as average travel times were free-flow-like for both before and after conditions. The full average weekday PTI and BI results and trends are shown on Tables 20-23 and Figures 26-29.

Table 20. Weekday Before-and-after Average PTI Comparisons (Entire Corridor) – AM and PM Peaks

Weekday		Planning Time Index									
Direction	Day	AM Peak Period (5:30am - 11am) Oct '14 - Feb '15	AM Peak Period (5:30am - 11am) Oct '15 - Feb '16	Change in AM Peak Period (#)	Change in AM Peak Period (%)	Statistical Significance at α = 0.05	PM Peak Period (2pm - 8pm) Oct '14 - Feb '15	PM Peak Period (2pm - 8pm) Oct '15 - Feb '16	Change in PM Peak Period (#)	Change in PM Peak Period (%)	Statistical Significance at α = 0.05
EB	Mon	1.357	1.394	0.037	2.727	Sig (<0.05)	1.254	1.147	0.107	8.533	Sig (<0.05)
	Tues	1.534	1.748	0.214	13.950	Sig (<0.05)	1.115	1.035	0.080	7.175	Sig (<0.05)
	Wed	1.446	1.574	0.128	8.852	Sig (<0.05)	1.078	1.312	0.234	21.707	Sig (<0.05)
	Thurs	1.648	1.732	0.084	5.097	Not Sig (0.140)	1.276	1.046	0.230	18.025	Sig (<0.05)
	Fri	1.133	1.197	0.064	5.649	Sig (<0.05)	1.269	1.146	0.123	9.693	Sig (<0.05)
	Average	**1.337**	**1.437**	0.100	7.479	**Sig (<0.05)**	**1.138**	**1.076**	0.062	5.448	**Sig (<0.05)**
WB	Mon	0.956	0.958	0.002	0.209	Not Sig (0.357)	1.705	1.918	0.213	12.493	Sig (<0.05)
	Tues	1.075	0.959	0.116	10.791	Sig (<0.05)	1.709	1.872	0.163	9.538	Sig (<0.05)
	Wed	1.036	0.948	0.088	8.494	Sig (<0.05)	1.856	1.936	0.080	4.310	Sig (<0.05)
	Thurs	0.967	0.928	0.039	4.033	Sig (<0.05)	1.988	1.915	0.073	3.672	Sig (<0.05)
	Fri	0.923	0.946	0.023	2.492	Not Sig (0.071)	1.919	1.864	0.055	2.866	Not Sig (0.079)
	Average	**0.962**	**0.930**	0.032	3.326	**Sig (<0.05)**	**1.708**	**1.773**	0.065	3.806	**Sig (<0.05)**

Table 21. Weekday Before-and-after Average BI Comparisons (Entire Corridor) – AM and PM Peaks

Weekday		Buffer Index									
Direction	Day	AM Peak Period (5:30am - 11am) Oct '14 - Feb '15	AM Peak Period (5:30am - 11am) Oct '15 - Feb '16	Change in AM Peak Period (#)	Change in AM Peak Period (%)	Statistical Significance at α = 0.05	PM Peak Period (2pm - 8pm) Oct '14 - Feb '15	PM Peak Period (2pm - 8pm) Oct '15 - Feb '16	Change in PM Peak Period (#)	Change in PM Peak Period (%)	Statistical Significance at α = 0.05
EB	Mon	0.116	0.101	0.015	12.931	Sig (<0.05)	0.140	0.117	0.023	16.429	Not Sig (0.078)
	Tues	0.138	0.171	0.033	23.913	Sig (<0.05)	0.070	0.068	0.002	2.857	Not Sig (0.396)
	Wed	0.107	0.133	0.026	24.299	Sig (<0.05)	0.063	0.183	0.120	190.476	Sig (<0.05)
	Thurs	0.174	0.164	0.010	5.747	Not Sig (0.362)	0.145	0.080	0.065	44.828	Sig (<0.05)
	Fri	0.066	0.081	0.015	22.727	Sig (<0.05)	0.103	0.127	0.024	23.301	Not Sig (0.054)
	Average	**0.060**	**0.068**	0.008	13.333	**Not Sig (0.078)**	**0.051**	**0.060**	0.009	17.647	**Sig (<0.05)**
WB	Mon	0.043	0.050	0.007	16.279	Not Sig (0.061)	0.122	0.149	0.027	22.131	Sig (<0.05)
	Tues	0.101	0.038	0.063	62.376	Sig (<0.05)	0.128	0.114	0.014	10.938	Sig (<0.05)
	Wed	0.080	0.034	0.046	57.500	Sig (<0.05)	0.140	0.132	0.008	5.714	Not Sig (0.212)
	Thurs	0.044	0.025	0.019	43.182	Sig (<0.05)	0.136	0.120	0.016	11.765	Sig (<0.05)
	Fri	0.027	0.041	0.014	51.852	Not Sig (0.055)	0.133	0.120	0.013	9.774	Sig (<0.05)
	Average	**0.030**	**0.019**	0.011	36.667	**Sig (<0.05)**	**0.058**	**0.056**	0.002	3.448	**Not Sig (0.136)**

Table 22. Weekday Before-and-after Average PTI Comparisons (Entire Corridor) – Midday and Overnight

Weekday		Planning Time Index									
Direction	Day	Midday Period (11am - 2pm) Oct '14 - Feb '15	Midday Period (11am - 2pm) Oct '15 - Feb '16	Change in Midday Period (#)	Change in Midday Period (%)	Statistical Significance at α = 0.05	Overnight Period (8pm - 5:30am) Oct '14 - Feb '15	Overnight Period (8pm - 5:30am) Oct '15 - Feb '16	Change in Overnight Period (#)	Change in Overnight Period (%)	Statistical Significance at α = 0.05
EB	Mon	1.176	1.009	0.167	14.201	Sig (<0.05)	0.927	0.950	0.023	2.481	Sig (<0.05)
	Tues	1.020	1.030	0.010	0.980	Not Sig (0.255)	0.920	1.004	0.084	9.130	Sig (<0.05)
	Wed	0.984	1.062	0.078	7.927	Sig (<0.05)	0.914	1.200	0.286	31.291	Sig (<0.05)
	Thurs	1.130	1.057	0.073	6.460	Sig (<0.05)	0.933	0.980	0.047	5.038	Sig (<0.05)
	Fri	0.985	0.999	0.014	1.421	Not Sig (0.223)	0.942	0.948	0.006	0.637	Not Sig (0.381)
	Average	**1.022**	**0.999**	0.023	2.250	**Sig (<0.05)**	**0.917**	**0.987**	0.070	7.634	**Sig (<0.05)**
WB	Mon	0.999	0.983	0.016	1.602	Not Sig (0.246)	0.954	0.939	0.015	1.572	Not Sig (0.107)
	Tues	1.094	0.971	0.123	11.243	Sig (<0.05)	0.972	0.964	0.008	0.823	Not Sig (0.201)
	Wed	1.235	0.963	0.272	22.024	Sig (<0.05)	0.934	0.989	0.055	5.889	Sig (<0.05)
	Thurs	0.965	1.020	0.055	5.699	Sig (<0.05)	0.978	0.959	0.019	1.943	Not Sig (0.151)
	Fri	1.094	1.094	0.000	0.000	Not Sig (0.481)	0.961	0.934	0.027	2.810	Sig (<0.05)
	Average	**1.033**	**0.975**	0.058	5.615	**Sig (<0.05)**	**0.940**	**0.939**	0.001	0.106	**Not Sig (0.380)**

Table 23. Weekday Before-and-after Average BI Comparisons (Entire Corridor) – Midday and Overnight

Weekday		Buffer Index									
Direction	Day	Midday Period (11am - 2pm) Oct '14 - Feb '15	Midday Period (11am - 2pm) Oct '15 - Feb '16	Change in Midday Period (#)	Change in Midday Period (%)	Statistical Significance at α = 0.05	Overnight Period (8pm - 5:30am) Oct '14 - Feb '15	Overnight Period (8pm - 5:30am) Oct '15 - Feb '16	Change in Overnight Period (#)	Change in Overnight Period (%)	Statistical Significance at α = 0.05
EB	Mon	0.127	0.055	0.072	56.693	Sig (<0.05)	0.026	0.031	0.005	19.231	Not Sig (0.104)
	Tues	0.059	0.069	0.010	16.949	Not Sig (0.132)	0.021	0.060	0.039	185.714	Sig (<0.05)
	Wed	0.042	0.074	0.032	76.190	Sig (<0.05)	0.017	0.175	0.158	929.412	Sig (<0.05)
	Thurs	0.111	0.067	0.044	39.640	Sig (<0.05)	0.028	0.051	0.023	82.143	Sig (<0.05)
	Fri	0.037	0.041	0.004	10.811	Not Sig (0.147)	0.036	0.033	0.003	8.333	Not Sig (0.403)
	Average	**0.039**	**0.028**	0.011	28.205	**Sig (<0.05)**	**0.014**	**0.046**	0.032	228.571	**Sig (<0.05)**
WB	Mon	0.069	0.062	0.007	10.145	Not Sig (0.305)	0.038	0.028	0.010	26.316	Not Sig (0.089)
	Tues	0.100	0.044	0.056	56.000	Sig (<0.05)	0.048	0.041	0.007	14.583	Not Sig (0.104)
	Wed	0.154	0.037	0.117	75.974	Sig (<0.05)	0.027	0.055	0.028	103.704	Sig (<0.05)
	Thurs	0.041	0.067	0.026	63.415	Sig (<0.05)	0.047	0.042	0.005	10.638	Not Sig (0.340)
	Fri	0.061	0.111	0.050	81.967	Sig (<0.05)	0.043	0.027	0.016	37.209	Sig (<0.05)
	Average	**0.044**	**0.033**	0.011	25.000	**Sig (<0.05)**	**0.020**	**0.020**	0.000	0.000	**Not Sig (0.460)**

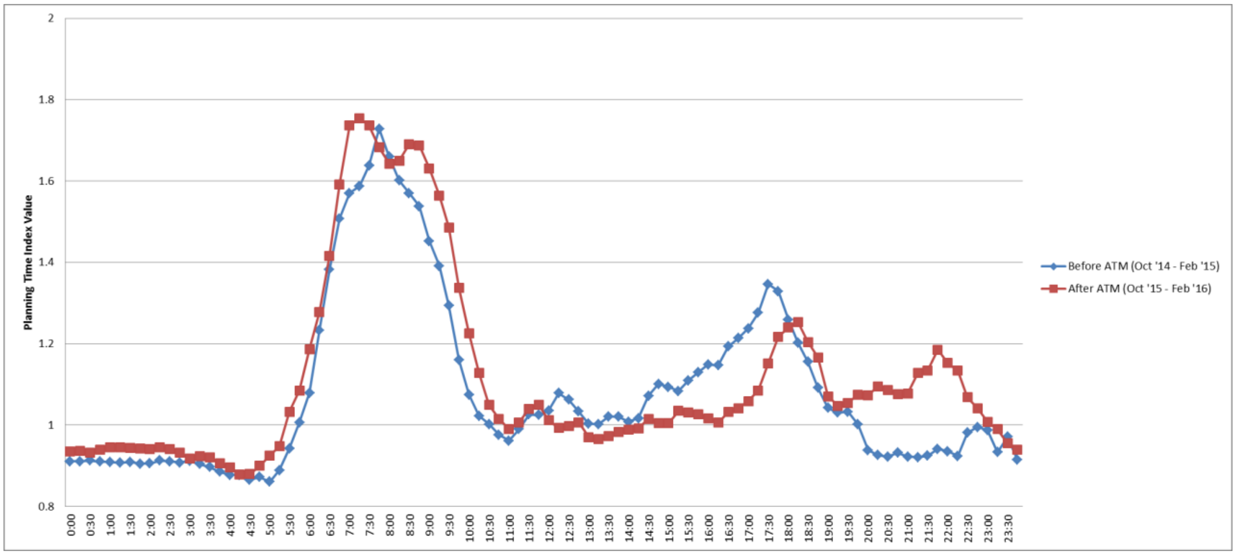


Figure 26. EB Before-and-after Average Weekday PTI Profile (Corridor-level)

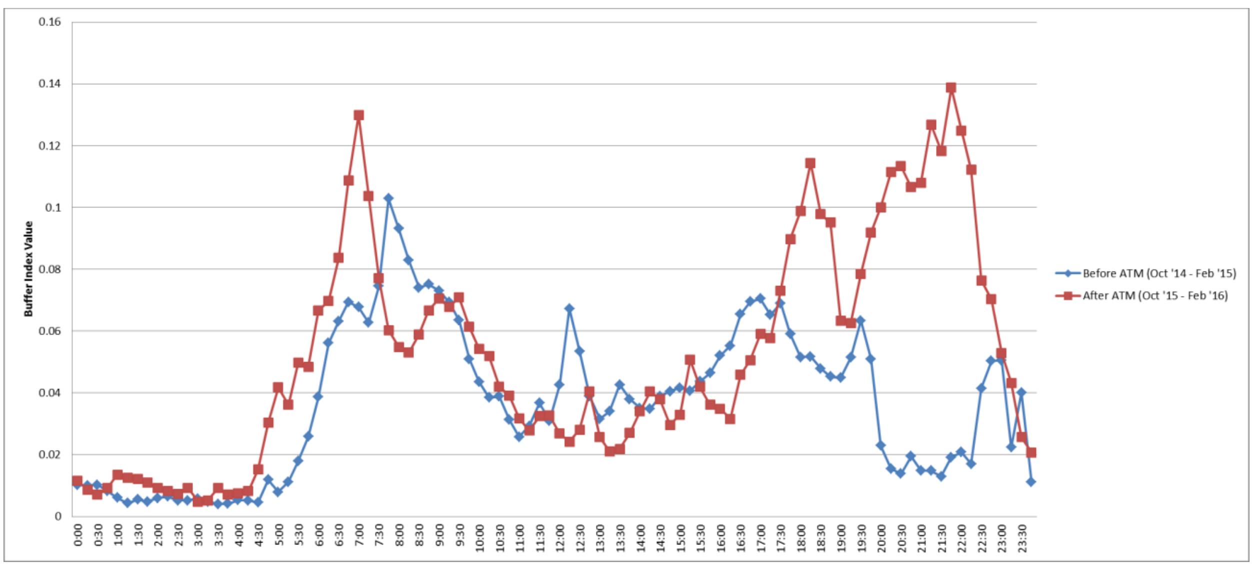


Figure 27. EB Before-and-after Average Weekday BI Profile (Corridor-level)

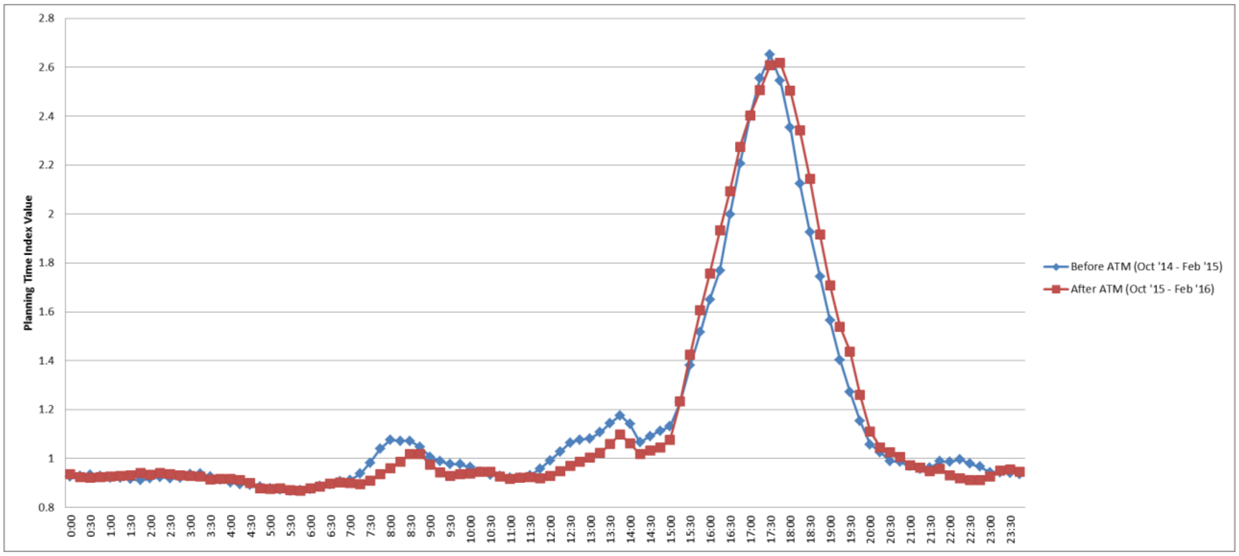


Figure 28. WB Before-and-after Average Weekday PTI Profile (Corridor-level)

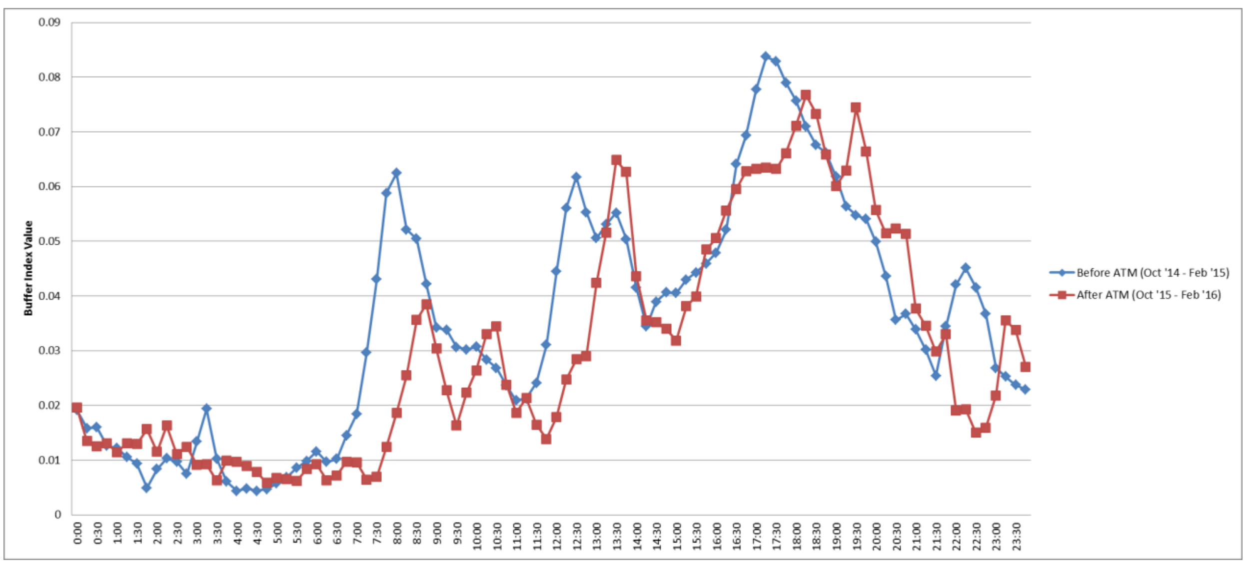


Figure 29. WB Before-and-after Average Weekday BI Profile (Corridor-level)

5.1.7 Weekend Corridor-level Travel Time Reliability Analysis

The travel time reliability for the weekend peak period improved the most out of all periods for both EB and WB directions. This statistically significant improvement is credible since weekend peak periods had the most improvements in average travel times. On EB, the average peak period PTI and BI improved by 0.13 (11.32%) and .01 (19.12%) respectively. On WB, the average peak period PTI and BI improved by 0.15 (13.62%) and 0.03 (50.75%) respectively. The average weekend PTI were reduced from above 1 to close to or less than 1, which represents that the travel time reliability during peak period has become like that of travel time reliability during free-flow conditions. The improvements were shown across all days of the week for both EB and WB conditions, which can be seen on Tables 22-23

The average weekend PTI and BI changes during the overnight off-peak period are negligible as average travel times were already approaching free-flow for both before and after conditions. The full average weekend PTI and BI results and trends are shown on Tables 24-25 and Figures 30-33.

Table 24. Weekend Before-and-after Average PTI Comparisons (Entire Corridor) – Peak and Off-peak

Weekend		Planning Time Index									
Direction	Day	Peak Period (10am - 8pm) Oct '14 - Feb '15	Peak Period (10am - 8pm) Oct '15 - Feb '16	Change in Peak Period (#)	Change in Peak Period (%)	Statistical Significance at α = 0.05	Off-Peak Period (8pm - 10am) Oct '14 - Feb '15	Off-Peak Period (8pm - 10am) Oct '15 - Feb '16	Change in Off-Peak Period (#)	Change in Off-Peak Period (%)	Statistical Significance at α = 0.05
EB	Sun	1.094	0.989	0.105	9.598	Sig (<0.05)	0.909	0.936	0.027	2.970	Sig (<0.05)
	Sat	1.257	1.083	0.174	13.842	Sig (<0.05)	0.917	0.920	0.003	0.327	Not Sig (0.380)
	Average	**1.148**	**1.018**	0.130	11.324	**Sig (<0.05)**	**0.909**	**0.923**	0.014	1.540	**Sig (<0.05)**
WB	Sun	0.978	0.912	0.066	6.748	Sig (<0.05)	0.904	0.915	0.011	1.217	Sig (<0.05)
	Sat	1.227	0.986	0.241	19.641	Sig (<0.05)	0.905	0.914	0.009	0.994	Sig (<0.05)
	Average	**1.087**	**0.939**	0.148	13.615	**Sig (<0.05)**	**0.901**	**0.909**	0.008	0.888	**Sig (<0.05)**

Table 25. Weekend Before-and-after Average BI Comparisons (Entire Corridor) – Peak and Off-peak

Weekend		Buffer Index									
Direction	Day	Peak Period (10am - 8pm) Oct '14 - Feb '15	Peak Period (10am - 8pm) Oct '15 - Feb '16	Change in Peak Period (#)	Change in Peak Period (%)	Statistical Significance at α = 0.05	Off-Peak Period (8pm - 10am) Oct '14 - Feb '15	Off-Peak Period (8pm - 10am) Oct '15 - Feb '16	Change in Off-Peak Period (#)	Change in Off-Peak Period (%)	Statistical Significance at α = 0.05
EB	Sun	0.086	0.056	0.030	34.884	Sig (<0.05)	0.020	0.026	0.006	30.000	Sig (<0.05)
	Sat	0.097	0.086	0.011	11.340	Not Sig (0.171)	0.023	0.019	0.004	17.391	Not Sig (0.236)
	Average	**0.068**	**0.055**	0.013	19.118	**Sig (<0.05)**	**0.017**	**0.017**	0.000	0.000	**Not Sig (0.401)**
WB	Sun	0.055	0.026	0.029	52.727	Sig (<0.05)	0.015	0.020	0.005	33.333	Sig (<0.05)
	Sat	0.102	0.058	0.044	43.137	Sig (<0.05)	0.017	0.022	0.005	29.412	Not Sig (0.073)
	Average	**0.067**	**0.033**	0.034	50.746	**Sig (<0.05)**	**0.012**	**0.016**	0.004	33.333	**Sig (<0.05)**

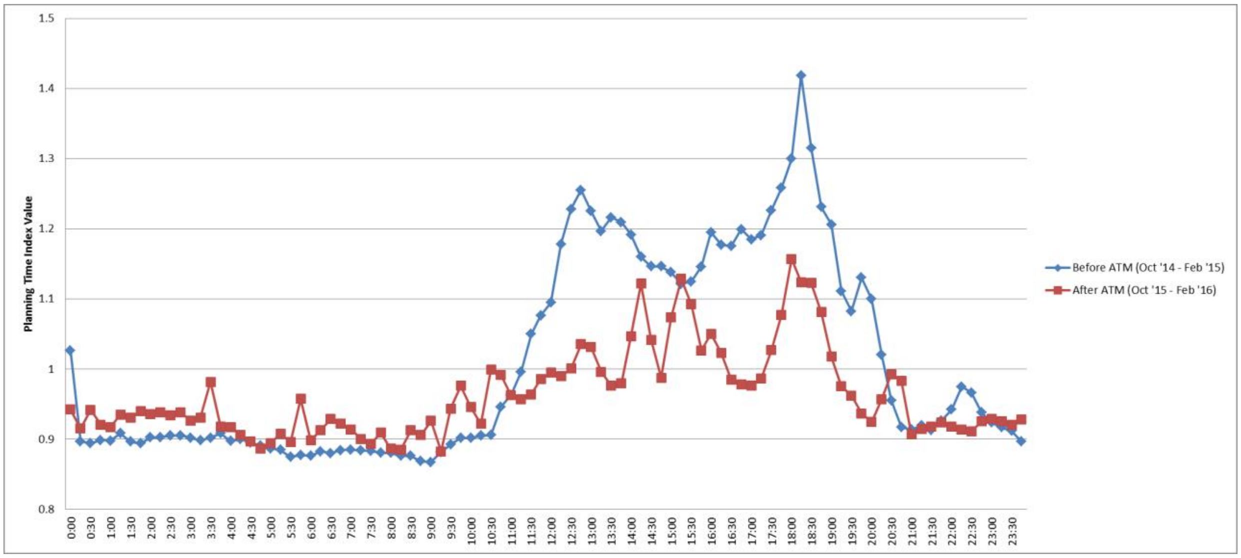


Figure 30. EB Before-and-after Average Weekend PTI Profile (Corridor-level)

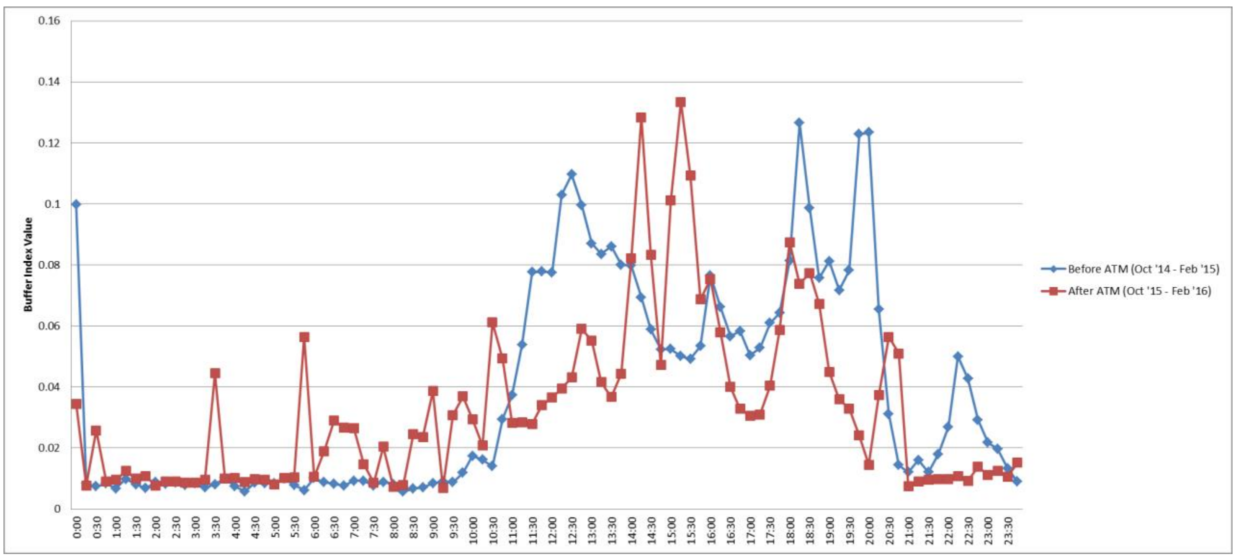


Figure 31. EB Before-and-after Average Weekend BI Profile (Corridor-level)

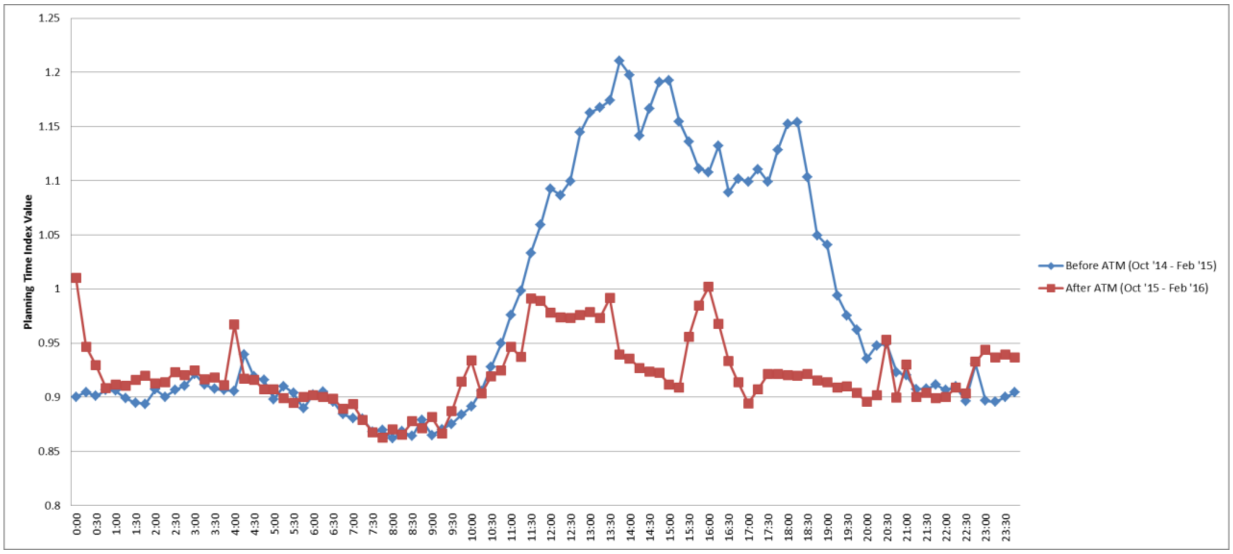


Figure 32. WB Before-and-after Average Weekend PTI Profile (Corridor-level)

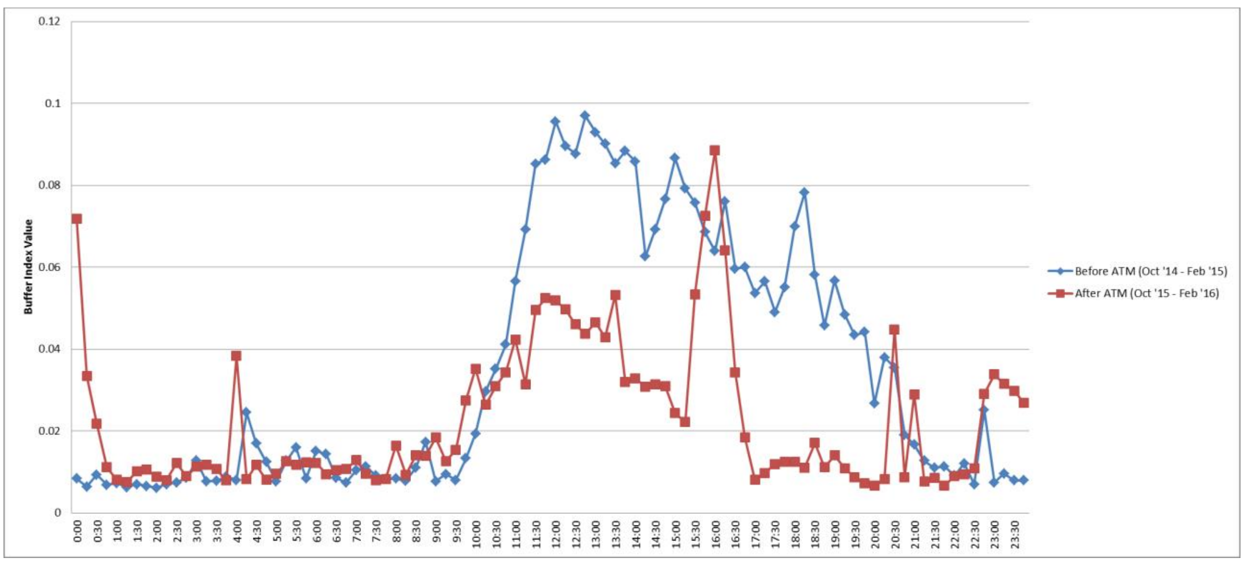


Figure 33. WB Before-and-after Average Weekend BI Profile (Corridor-level)

5.1.8 Corridor-level Total Traveler Delay Analysis

The total traveler delay was considered to be the combination of both recurrent and non-recurrent congestion levels. Following the procedure from the methodology, this section discusses the results of the travel delay analysis.

The EB and WB reference average travel time profiles developed for Oct – Feb of 2012-2013, 2013-2014, 2014-2015 and 2016 are shown on Figures 34, 35, 36 and 37 respectively. Before the implementation of ATM, the reference average travel time profiles show a deteriorating trend over the years during the peak periods, with Oct – Feb 2013-2014 having the worst performing reference travel time profile for both EB and WB. This could mean there was unusual number of non-recurrent congestion events or the magnitude of the non-recurrent congestion events was very high during Oct - Feb 2013-2014, resulting in them being included in the “typical” congestion profile being generated using k-NN. Reference travel time profiles were developed by considering both recurrent and non-recurrent congestion events. If the non-recurrent congestion events are so frequent, the average travel times from the non-recurrent congestion events could be considered as being the average travel times from the recurrent-congestion events when the reference travel time profiles were developed. Thus, there were difficulties in truly isolating recurring versus nonrecurring congestion on the corridor due to the large number of non-recurring events. In some cases, the nonrecurring congestion impacts appear to have been screened out, while in other cases they appear to have influenced the reference travel time profile. As a result, estimated recurring and nonrecurring congestion are combined to indicate a total delay on the corridor, which is considered to be a more reliable and stable metric of system performance. After the activation of ATM, the reference travel times profile showed a dramatic improvement for the weekend EB and WB directions. Like the average travel time profiles during average weekends, the reference travel time profile showed free-flow-like conditions during average weekends. However, the weekday reference travel time profiles for after-ATM period showed mixed results, with EB improving and WB deteriorating from the before-ATM year.

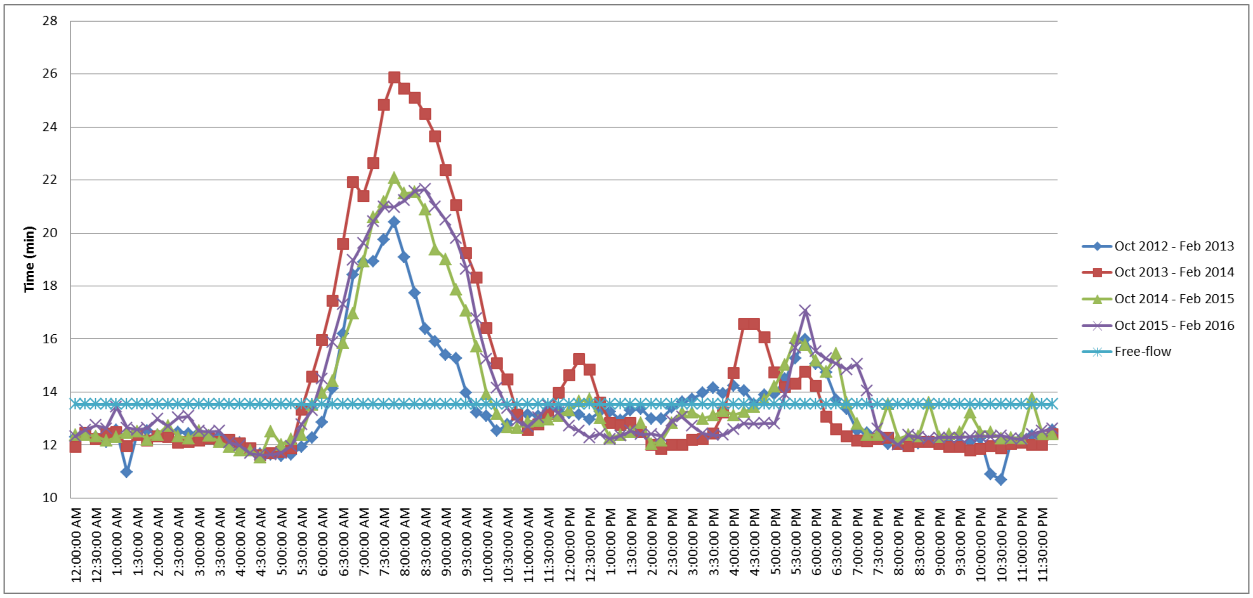


Figure 34. EB Average Weekday Reference Travel Time Profiles (Corridor-level)

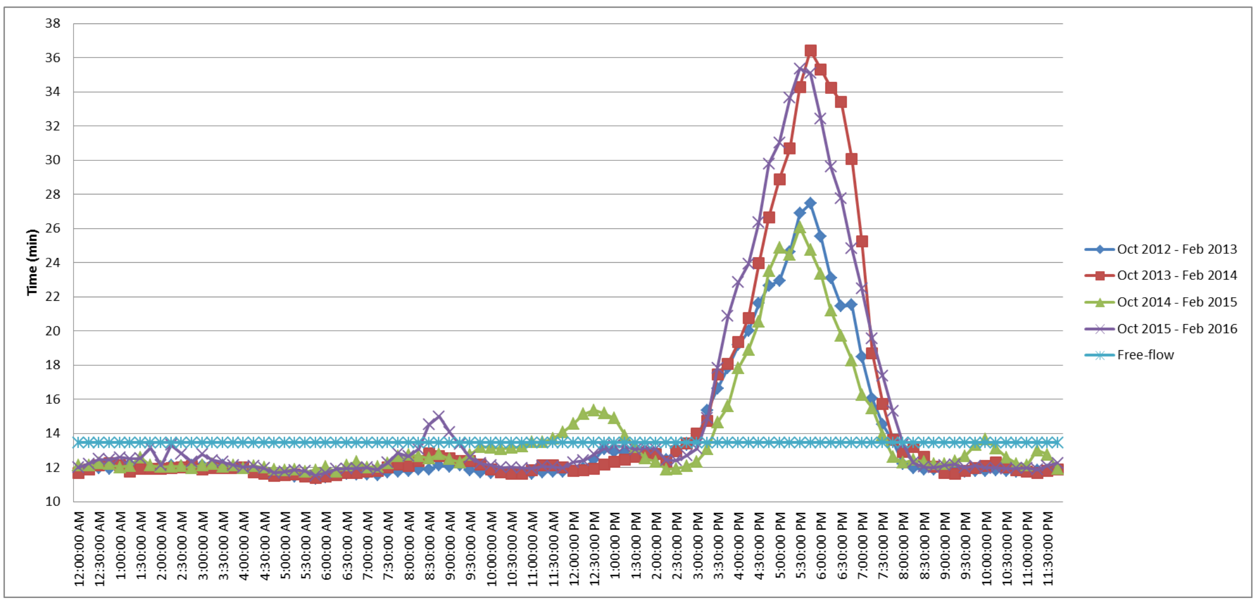


Figure 35. WB Average Weekday Reference Travel Time Profiles (Corridor-level)

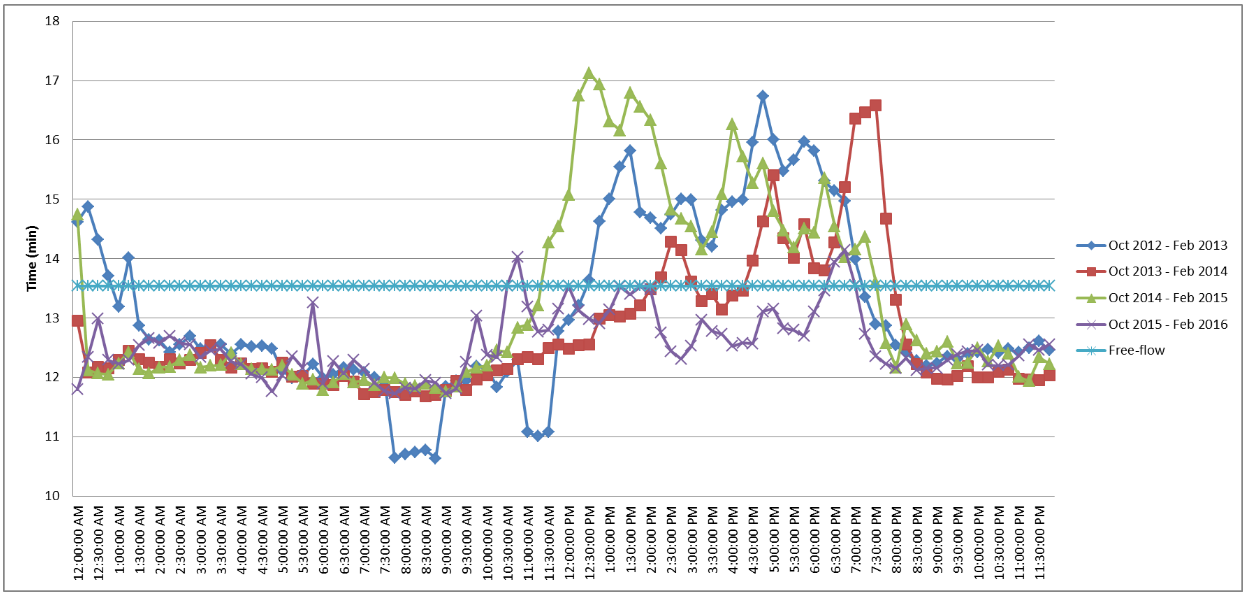


Figure 36. EB Average Weekend Reference Travel Time Profiles (Corridor-level)

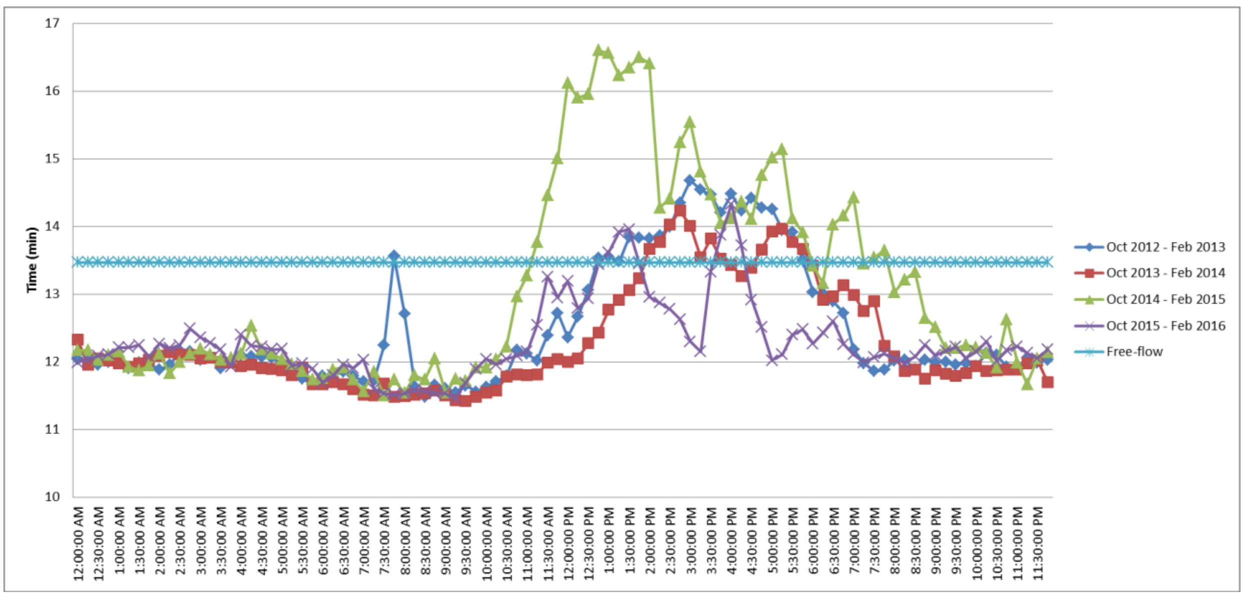


Figure 37. WB Average Weekend Reference Travel Time Profiles (Corridor-level)

The next components required to calculate travel delay were the yearly AADT and average 15-min volume distribution. Since 2016 AADT is not available, AADT growth rates from 2014-2015 was used to estimate the 2016 AADT, and the calculation results are shown in Table 26. The weekday weighted average growth rate by length of segment was 3.92% and 2.86% for EB and WB, respectively. The weekend weighted average growth rate by length of segment was 4.13% and 1.59% for EB and WB, respectively. The EB and WB average 15-min volume distribution profiles for both non-shoulder and shoulder segments are shown on Figures 38-45.

Table 26: Observed AADT (2014-2015) and Estimated AADT (2016)

Route Label	Link Length	Segment	2014 Weekday AADT	2015 Weekday AADT	% Change	Weighted Avg % Change	est. 2016 Weekday AADT	2014 Weekend AADT	2015 Weekend AADT	% Change	Weighted Avg % Change	est. 2016 Weekend AADT
I-66 EB	1.25	1	70000	71000	1.43	3.92	73783	56000	60500	8.04	4.13	62999
	1.86	2	82000	85000	3.66		88332	68000	67500	-0.74		70288
	2.57	3	67000	68000	1.49		70666	60000	57500	-4.17		59875
	1.85	4	92000	94000	2.17		97685	74500	80000	7.38		83304
	2.13	5	96000	99000	3.13		102881	75000	78000	4.00		81221
	2.98	6	79000	86000	8.86		89371	65000	72000	10.77		74974
I-66 WB	0.83	1	68000	70000	2.94	2.86	72002	54000	56000	3.70	1.59	56890
	3.03	2	87000	87000	0.00		89488	69500	69500	0.00		70605
	2.20	3	65000	65000	0.00		66859	54500	51000	-6.42		51811
	2.01	4	89000	98000	10.11		100803	71500	80500	12.59		81780
	1.41	5	89000	85000	-4.49		87431	71500	71000	-0.70		72129
	3.62	6	86000	91000	5.81		93603	72000	73500	2.08		74669

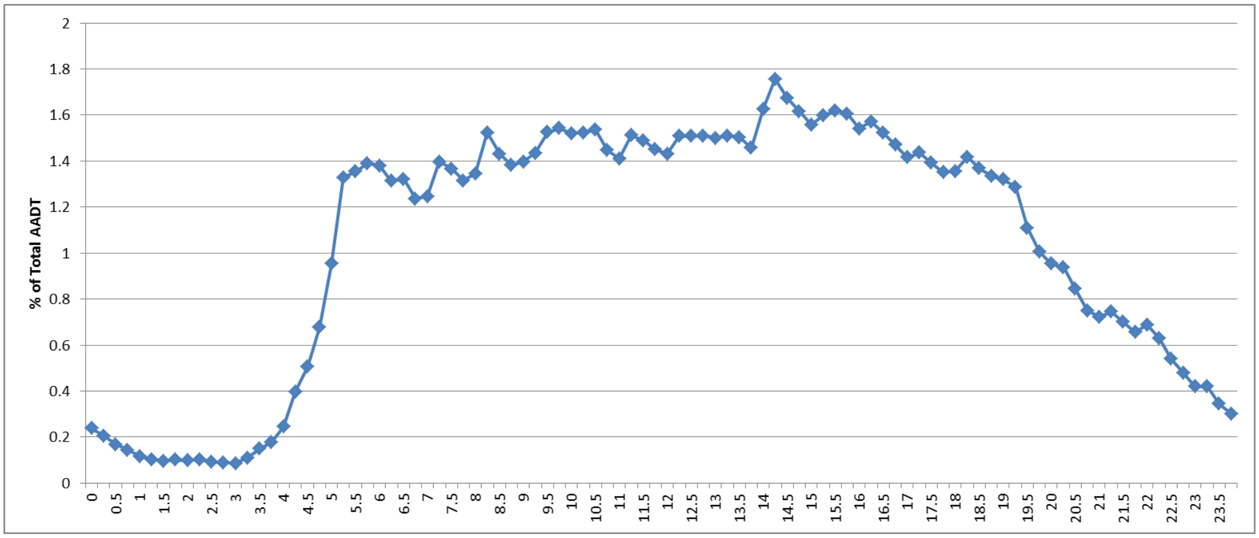


Figure 38. EB Weekday Volume Distribution Profile for Non-Shoulder Segments

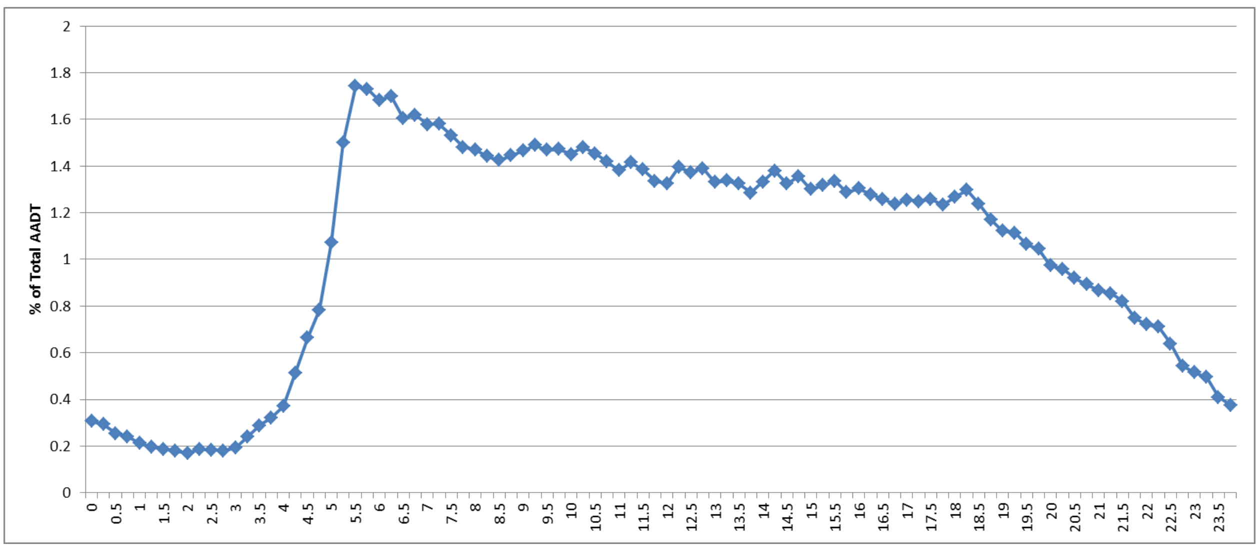


Figure 39. EB Weekday Volume Distribution Profile for Shoulder Segments

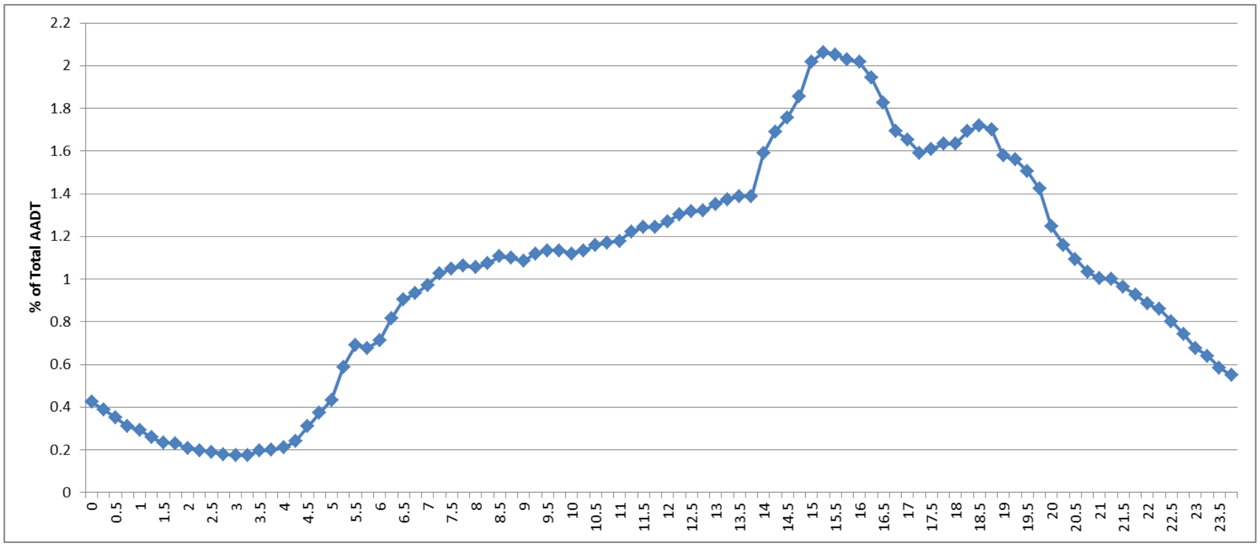


Figure 40. WB Weekday Volume Distribution Profile for Non-Shoulder Segments

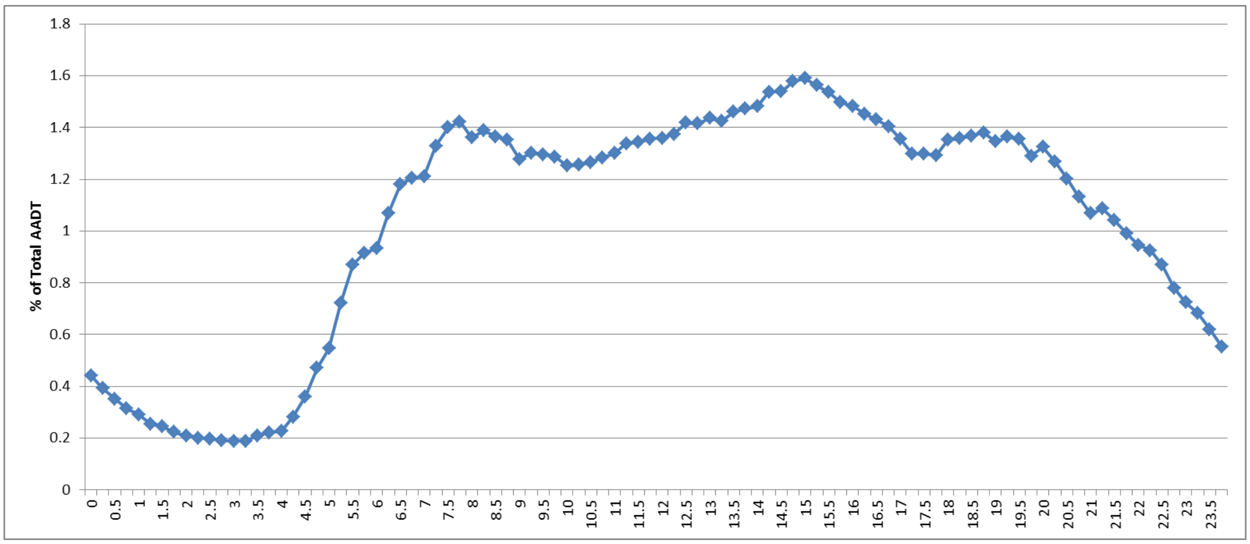


Figure 41. WB Weekday Volume Distribution Profile for Shoulder Segments

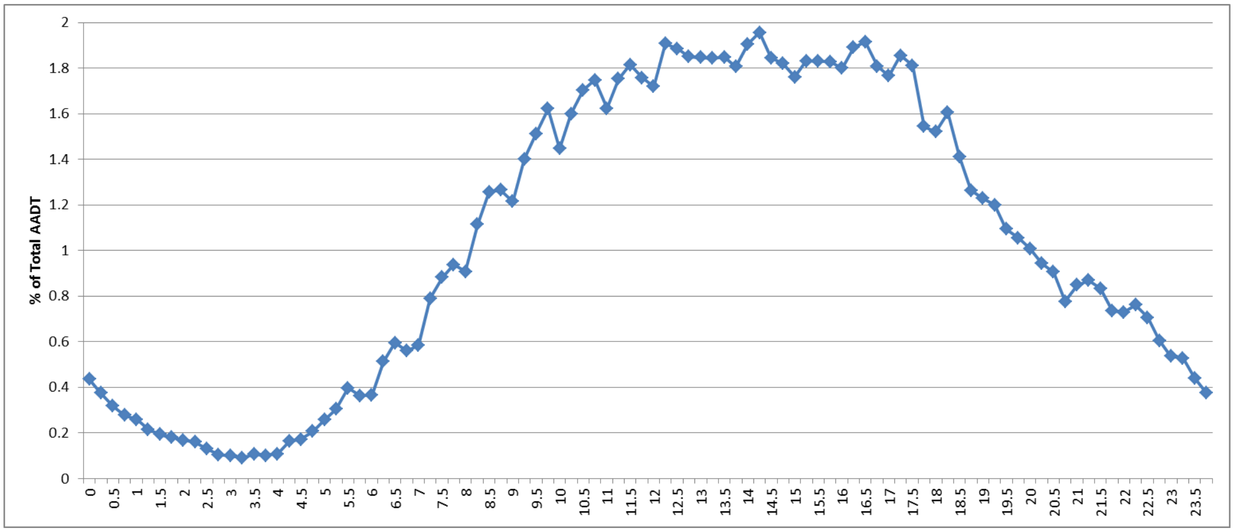


Figure 42. EB Weekend Volume Distribution Profile for Non-Shoulder Segments

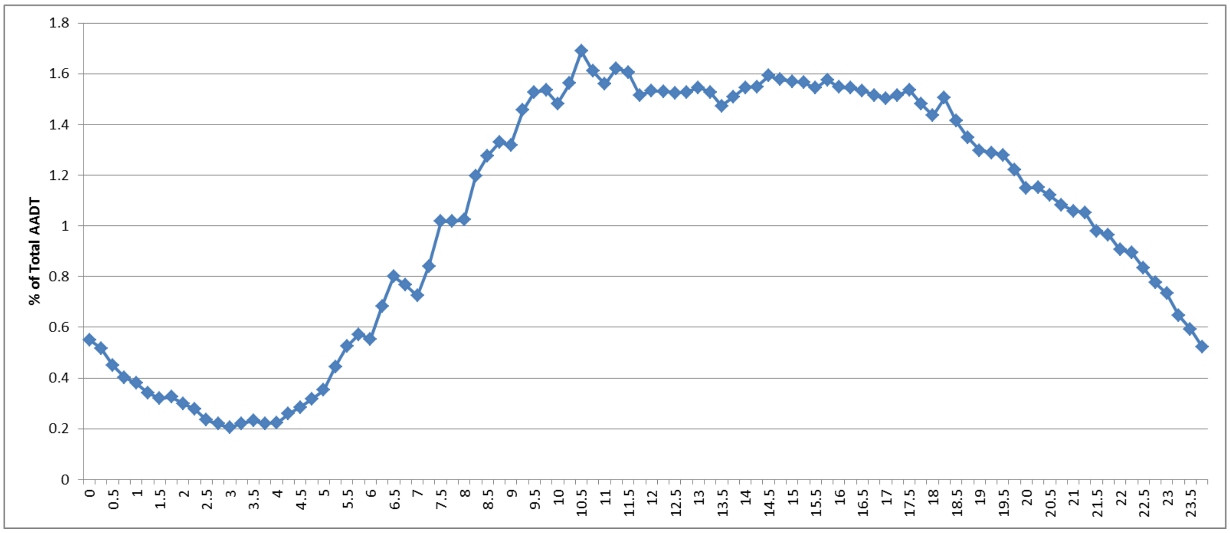


Figure 43. EB Weekend Volume Distribution Profile for Shoulder Segments

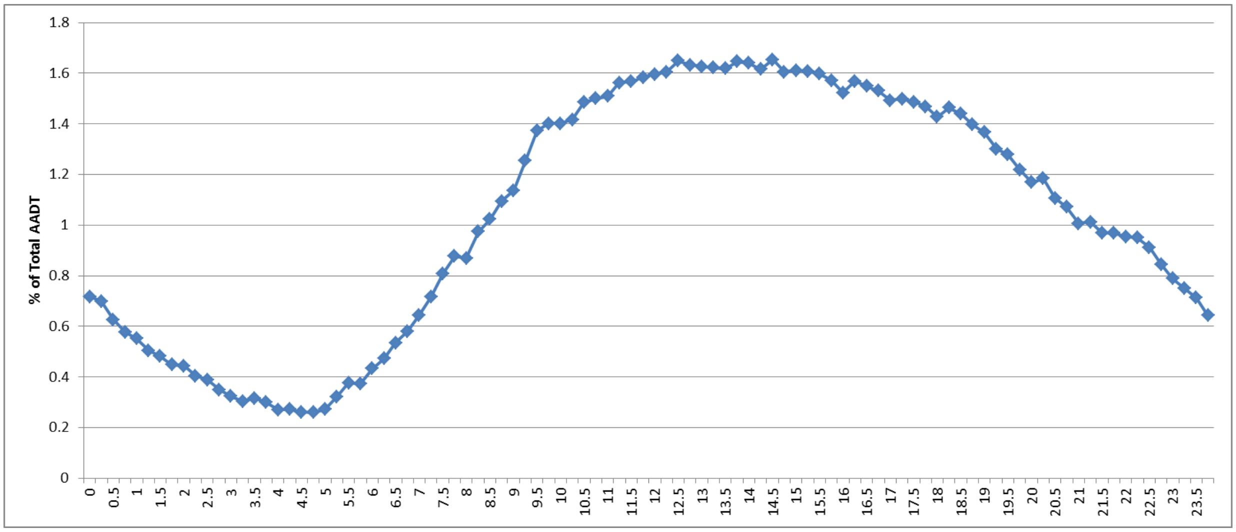


Figure 44. WB Weekend Volume Distribution Profile for Non-Shoulder Segments

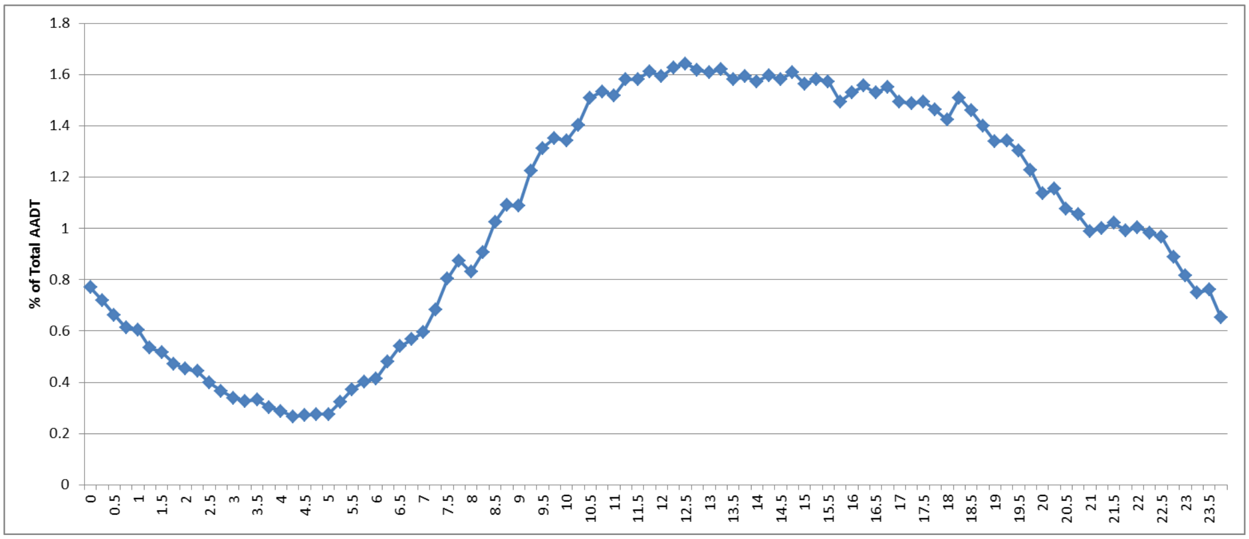


Figure 45. WB Weekend Volume Distribution Profile for Shoulder Segments

With the reference travel time profile, free flow travel time profile, average travel time profiles, and volume distribution profiles developed, the recurrent and non-recurrent congestion levels were analyzed by using equations 3 and 4. Tables 27-28 show the recurrent congestion and non-recurrent congestion levels by direction, time period, and weekday/weekend.

Overall total daily recurrent congestion levels increased on weekdays for both EB and WB directions by 14.32% and 77.58% respectively. This questionably large increase in recurrent congestion levels in the WB direction was driven by the reference travel times. For example, on WB weekdays the reference travel time profile for Oct 2015 – Feb 2016, like the reference travel time profile for Oct 2013 – Feb 2014 that was discussed on page 91, has much higher average travel times in the PM Peak period. Since the calculation of recurrent congestion levels is heavily dependent on the reference travel times, the large increase in the recurrent congestion levels was inevitable. The highest recurrent congestion levels were from peak periods while traveling in the peak directions (AM for EB, PM for WB). The peak periods contained 75-95% of all recurrent congestion levels. Since the average travel times for these peak periods were higher for after-ATM than before-ATM period, it is logical that the reference travel times, which were developed by using these average travel times, would be higher for after-ATM conditions. This higher reference travel times equate to greater recurrent congestion levels, which is supported by equation 3. However, total daily recurrent congestion levels improved significantly on weekends for both EB and WB directions by 76.80% and 73.96% respectively. Most of this improvement occurred during weekend peak period for both EB and WB, and this improvement can be supported by the same logic that lower reference travel times equate to lesser recurrent congestion levels.

Overall non-recurrent congestion levels showed a mixed result, with EB weekday and weekend levels deteriorating by 9.99% and 15.05% respectively, and WB weekday and weekend levels improving by 75.01% and 2.03% respectively. Non-recurrent congestion levels are also influenced heavily by the reference travel time profiles, especially for the WB weekday periods. However, unlike the recurrent congestion levels, unusually high reference travel time profiles lead to non-recurrent congestion levels becoming lower as the unusually high reference travel time profiles already contain much of the non-recurrent congestion levels. Because of this reason, it is more stable to analyze the sum of the recurrent and non-recurrent congestion levels to evaluate total traveler delay improvement or deterioration.

Tables 29 show the total traveler delay calculation results, developed by adding up the recurrent congestion and non-recurrent congestion levels on Table 25 and 26. For an average weekday EB and WB, the total traveler delay levels have deteriorated by 12.96% and 9.01% respectively. For an average weekend on EB and WB, the total traveler delay levels have improved by 58.12% and 67.76% respectively. The increasing trend of average travel times in peak hour periods has created additional recurrent congestion levels on an average weekday. The additional opening of the HSR during weekends, which have dramatically improved average travel times, translated to large improvements in traveler delay levels for the weekend period.

The values from Table 27-29 should be interpreted as daily levels in vehicle-minutes. For example, on Table 27, the total traveler delay level for EB weekday before-ATM period should be interpreted as 178,112 minutes of traveler delay occurring on this corridor per day on a weekday.

Table 27. Day of the Week Recurrent Congestion Levels (Entire-Corridor)

Total Recurrent Congestion (min)				
		Before	After	Change (%)
EB	Weekday	122450	139984	14.32
	Weekend	80416	18654	76.80
WB	Weekday	169213	300489	77.58
	Weekend	70859	18450	73.96

Table 28. Day of the Week Non-Recurrent Congestion Levels (Entire-Corridor)

Total Non-Recurrent Congestion (min)				
		Before	After	Change (%)
EB	Weekday	55662	61220	9.99
	Weekend	20536	23625	15.05
WB	Weekday	138082	34506	75.01
	Weekend	6692	6556	2.03

Table 29. Day of the Week Traveler Delay Levels (Entire-Corridor)

Total Traveler Delay (min)				
Direction	Day of the Week	Before	After	Change (%)
EB	Weekday	178112	201204	12.96
	Weekend	100952	42279	58.12
WB	Weekday	307295	334995	9.01
	Weekend	77551	25006	67.76

5.2 CORRIDOR-LEVEL SAFETY ANALYSIS

5.2.1 Corridor-level Crash Rate Analysis

Although only 3 months of crash data from after ATM implementation were available, it is useful to examine preliminary trends in safety following system activation. These initial trends may not be sustainable, but could provide some indication of initial reactions to the system. According to RNS police crash report data, the total number of crashes has generally been increasing for the before-ATM conditions and this trend is shown on Table 31-32.

Rear-end and sideswipe crashes, which make up of approximately 70-90% of all crashes on the study area, are of the main safety concerns and are most likely to be impacted by ATM. The ATM is known to be effective in mitigating rear-end and sideswipe crashes as speed harmonization and expansion of roadway capacity help to reduce the number of vehicle-to-vehicle interactions (Fontaine and Miller, 2012). Crash rate, which accounts for annual AADT growth in the safety analysis, was analyzed for the corridor and the crash rate results are shown on Table 33. For the average weekdays on both EB and WB, during the before-ATM years of 2012-2014 (Oct-Dec), the crash rates increased annually by 6-34%. For the average weekends on both EB and WB, during the before-ATM years of 2012-2014 (Oct-Dec), the crash rates changes showed slight increased trends.

Given the differing operational impacts between weekdays and weekends, the crash trends were examined separately by those two time periods. This further reduces the amount of after data available, however, so these results should again be viewed with caution. The trends indicate that after the implementation of ATM, crash rates were either decreased or the rate of crash rate increase had been reduced. For all crash severities and rear-end and sideswipe crashes only, crash rates increased every year from 2012-2014, except for the EB weekend period during

2013-2014. As shown on Figure 46 below, for all crash severities and rear-end and sideswipe crashes only, EB weekdays saw a crash rate reduction of 6.90% after ATM implementation while there was a crash rate increase of 6.33% in the before-ATM period. WB weekdays saw crash rate increases of only 2.08% after ATM implementation while there was a crash rate increase of 35.66% immediately prior to ATM activation. The crash rates improvements were much more evident on weekends after the implementation of ATM, as EB and WB weekends saw crash rate improvement of 21.51% and 48.05% versus the before ATM period. The improvement trends were very similar even when the rear-end and sideswipe crashes were divided into Property Damage Only (PDO) and Fatal and Injury crashes, except for WB weekday Fatal and Injury crashes. These results are shown on Figures 47-48.

The crash rates were calculated for using only Oct-Dec crash data for each year, so the full yearly trends may not be represented by this analysis. However, seeing the consistent improvements in crash rates over all conditions especially on weekends show that there may have been noticeable safety improvements along the corridor by implementing ATM. Since VSL was only activated starting on mid-January, most of the safety benefits that may have come out of the ATM in this time period are likely to have been due to HSR operation. As a result, the initial empirical safety evidence shows promising results due to decreased congestion on the corridor, although this is based on limited data.

Table 30. Corridor-level Weekday and Weekend AADTs for 2012-2015

AADT Weighted Averages - Entire Corridor									
Direction	Length (mi.)	2012 Weekday	2013 Weekday	2014 Weekday	2015 Weekday	2012 Weekend	2013 Weekend	2014 Weekend	2015 Weekend
EB	12.41	80206	81376	80879	84071	67145	67295	66610	69434
WB	12.35	80811	83071	82347	84806	66864	68903	67212	68492

Table 31. Corridor-level Crash Frequency Results for All Crashes

Crash Frequency (All Crashes) - Entire Corridor									
Direction	Length (mi.)	Weekday				Weekend			
		Oct '12 - Dec '12	Oct '13 - Dec '13	Oct '14 - Dec '14	Oct '15 - Dec '15	Oct '12 - Dec '12	Oct '13 - Dec '13	Oct '14 - Dec '14	Oct '15 - Dec '15
EB	12.41	87	99	108	106	32	41	45	33
WB	12.35	59	65	91	106	14	28	22	14

Table 32. Corridor-level Crash Frequency Results for Rear-End and Sideswipe Crashes

Crash Frequency (Rear-end and Sideswipe Crashes) - Entire Corridor									
Direction	Type	Weekday				Weekend			
		Oct '12 - Dec '12	Oct '13 - Dec '13	Oct '14 - Dec '14	Oct '15 - Dec '15	Oct '12 - Dec '12	Oct '13 - Dec '13	Oct '14 - Dec '14	Oct '15 - Dec '15
EB	PDO	49	62	65	64	20	21	20	20
	Injury + Fatal	29	26	28	26	10	13	13	7
WB	PDO	40	40	60	54	9	8	11	7
	Injury + Fatal	8	18	18	28	3	6	6	2

Table 33. Corridor-level Crash Rate Results for Rear-end and Sideswipe Crashes

Crash Rates (Rear-end and Sideswipe Crashes) - Entire Corridor									
Direction	Type	Weekday				Weekend			
		Oct '12 - Dec '12	Oct '13 - Dec '13	Oct '14 - Dec '14	Oct '15 - Dec '15	Oct '12 - Dec '12	Oct '13 - Dec '13	Oct '14 - Dec '14	Oct '15 - Dec '15
EB	PDO	33.40	41.65	43.93	41.62	16.28	17.06	16.41	15.75
	Injury + Fatal	19.77	17.47	18.93	16.91	8.14	10.56	10.67	5.51
	Total	53.16	59.12	62.86	58.52	24.42	27.62	27.08	21.26
WB	PDO	27.19	26.45	40.02	34.98	7.39	6.38	8.99	5.61
	Injury + Fatal	5.44	11.90	12.01	18.14	2.46	4.78	4.90	1.60
	Total	32.63	38.35	52.03	53.11	9.86	11.16	13.89	7.22

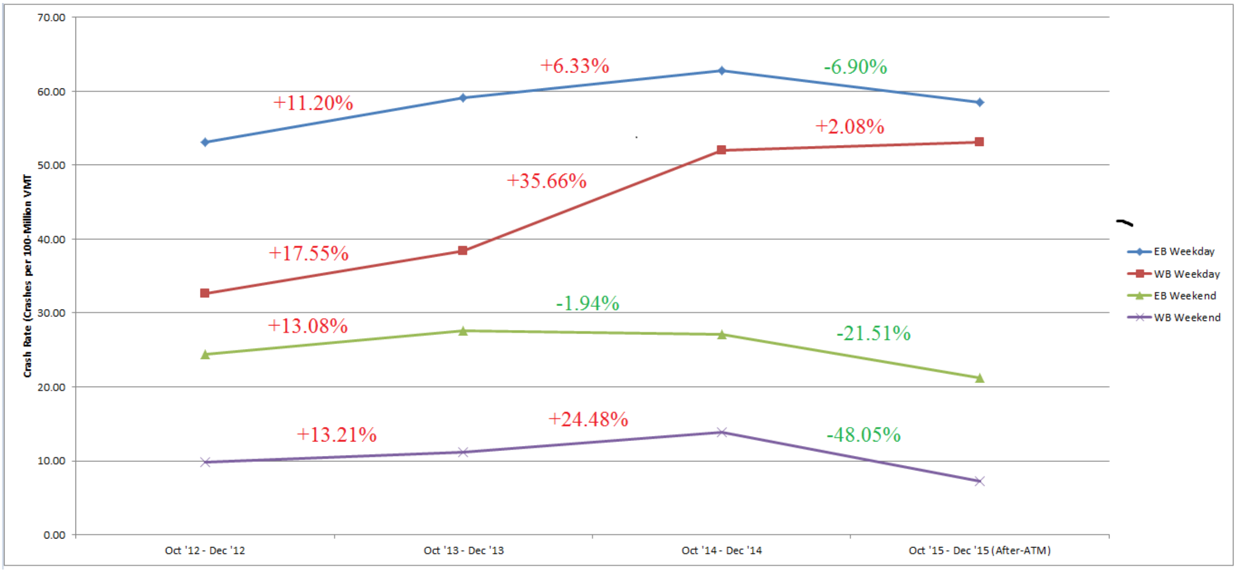


Figure 46. Crash Rate Trends for 2012-2015 on All Severity (Corridor-level, Rear-end and Sideswipe Crashes)

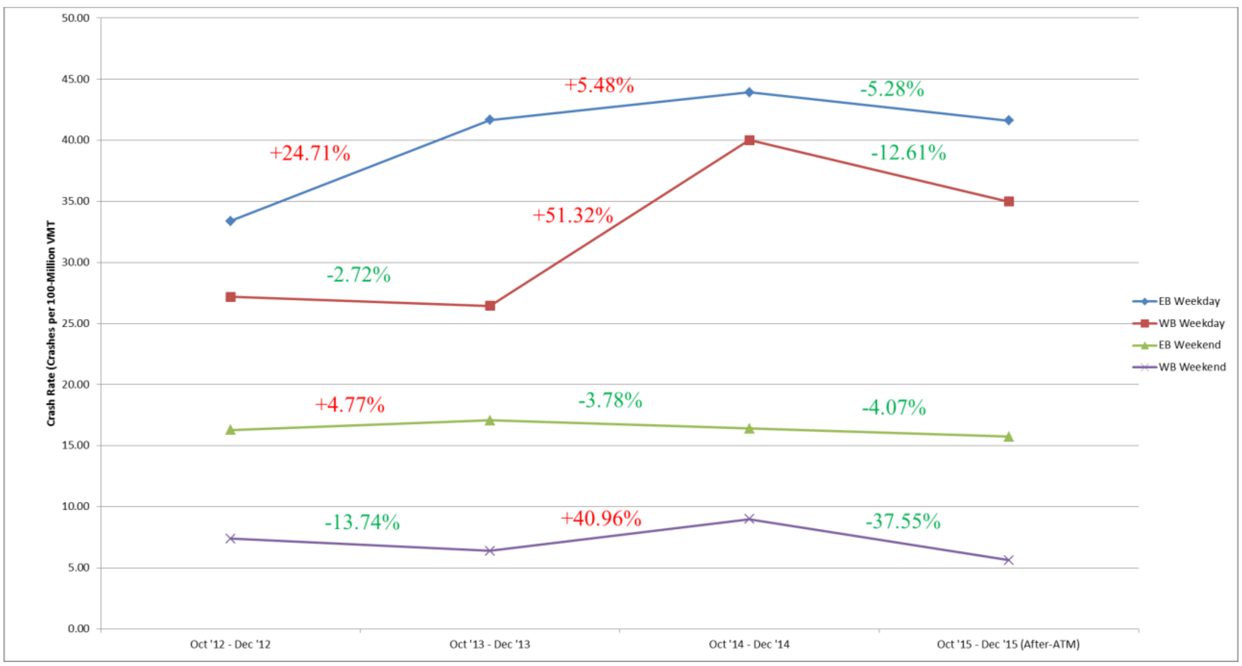


Figure 47. Crash Rate Trends for 2012-2015 on PDO Crashes (Corridor-level, Rear-end and Sideswipe Crashes)

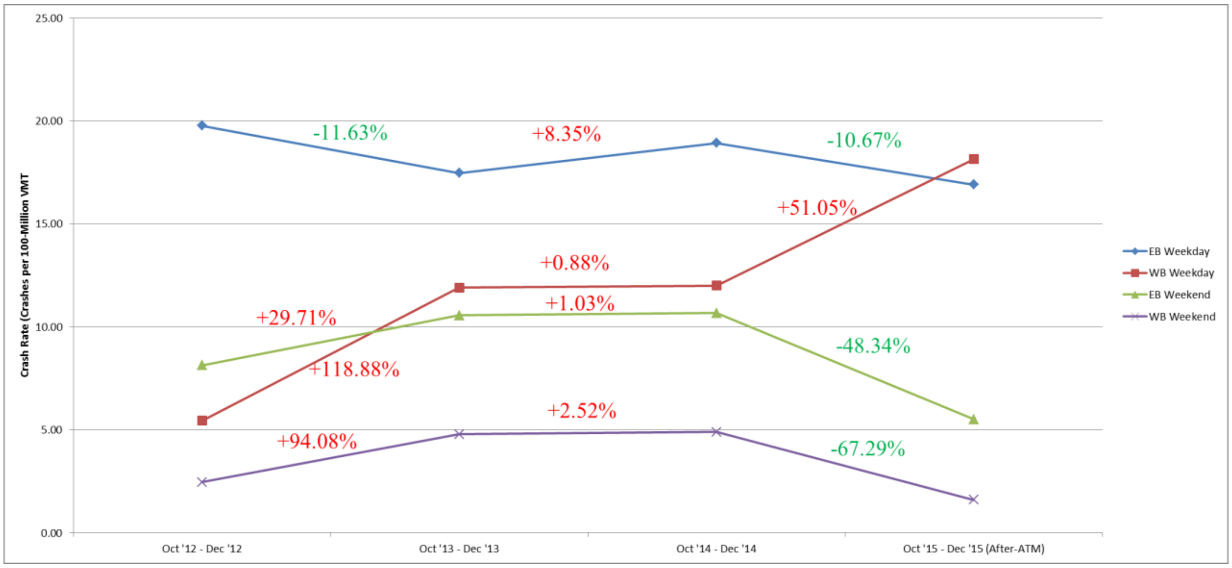


Figure 48. Crash Rate Trends for 2012-2015 on Fatal and Injury Crashes (Corridor-level, Rear-end and Sideswipe Crashes)

5.2.2 Corridor-level Crash Surrogate Analysis

In order to quickly estimate the number of rear-end and sideswipe crashes for the entire study period and for future periods, crash frequency models were developed by correlating speed drop events with crash frequency. Because RNS crash data only contained crash data from Oct – Dec of 2015, and INRIX data can be acquired real-time, it was thought that estimating the number of rear-end and sideswipe crashes using safety surrogate measures that well correlate with past safety at the site would augment the empirical evaluation. Initial investigations revealed that there was a strong linear relationship between speed drop events and crash frequency.

The full results of the weekday and weekend models for both can be found on Table 34-35. The equations for the number of rear end and sideswipe crashes per 15-minute time period across all interchanges and directions as function of the number of 10 mph speed drop events were:

$$\text{Weekday:} \quad \mathrm{C} = 0.045 \times \mathrm{S} - 2.28 \times 10^{-5} \times S^2$$

Equation 6. Weekday Rear-end and Sideswipe Crash Model

$$\text{Weekend:} \quad \mathrm{C} = 0.045 \times \mathrm{S} - 7.32 \times 10^{-5} \times S^2$$

Equation 7. Weekend Rear-end and Sideswipe Crash Model

Where C is the crash frequency and S is the number of speed drop events at the 10 mph threshold. It should be emphasized that only corridor-level correlations were found to be significant, and no models with strong predictive power could be developed for individual interchanges. The adjusted R square values for both models were high at 0.819 for the weekday model and 0.658 for the weekend model, which shows good explanatory power of the models. The constant values for both models were not significantly different from zero at the 95th confidence level. The constants being insignificant makes logical sense as one would expect zero crashes when there are no speed drop events. The models did not violate any of the regression assumptions. The relationship was linear, variables were normally distributed and homoscedasticity was satisfied. Models developed from the randomly selected 70% training data were validated with the appropriate speed drop events from the remaining 30% holdout data. The validation results showed that there was good agreement between the models and the holdout data in terms of both bias and absolute error. The 30% holdout data had an average over-prediction bias for rear end and sideswipe crashes of +6.07% for weekday model and +11.32% for weekend model. The mean absolute percentage errors (MAPE) for the weekday and the weekend models were 11.22% and 19.94% respectively, indicating decent model fit. The MAPE percentage values validate that the models developed from the training data and the developed models could be used to predict sideswipe and rear end crashes as a function of speed drop events on an aggregate basis at interchanges on the corridor.

Table 34. Corridor-level Weekday Regression Model for Speed Drop Events (Rear-end and Sideswipe Crash)

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
Adjusted R Square = 0.819	(Constant)	.265	.414		.641	.523
	spd10	.045	.004	1.319	10.330	.000
	spd10_sq	-2.275E-05	.000	-.442	-3.459	.001

Table 35. Corridor-level Weekend Regression Model for Speed Drop Events (Rear-end and Sideswipe Crash)

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
Adjusted R Square = 0.658	(Constant)	.080	.198		.406	.685
	spd10	.045	.007	1.312	6.351	.000
	spd10_sq	-7.316E-05	.000	-.523	-2.530	.013

The model was further validated with limited available after-ATM RNS crash data that showed the efficiency of the models. The number of rear-end and sideswipe crashes in the ramp influence area during Oct-Dec 2015 was compared to that of the estimated number of rear-end and sideswipe crashes using the developed models. The model accurately predicted both EB and WB weekday crash frequencies with only 3 and 11 percent error respectively. The model was over-predicting EB and WB weekend crash frequencies, mostly likely due to the lower adjusted R square value, with 10 and 25 percent errors respectively. Likewise, the small crash count on weekends (8 and 21 observed crashes in WB and EB, respectively) also made it easier to get higher MAPE values. These validation results can be found on Table 36.

The estimated rear-end and sideswipe crash frequency results showed that after the implementation of ATM, the estimated total rear-end and sideswipe crash frequencies were reduced across both the EB and WB direction for weekdays and weekends. These estimated crash frequency show the estimated crash frequency for before-ATM (Oct ’14 – Feb ’15) and after-ATM (Oct ’15 – Feb ’16) periods. On an average weekday, the crash estimates were improved from 148.00 crashes to 130.85 crashes (11.59% improvement) on EB and 113.33

crashes to 99.30 crashes (12.38% improvement) on WB. On an average weekend, the crash estimates were improved from 87.64 crashes to 38.67 crashes (55.88% improvement) on EB and 53.34 crashes to 16.25 crashes (69.54% improvement) on WB. The time-of-day analysis on Tables 38 to 40 show that most of these crash improvements were from off-peak, midday periods on weekdays and on peak periods on weekends. The estimated crash frequency profiles shown on Figures 49-52 show that before the implementation of ATM, the estimated rear-end and sideswipe crash frequencies were very constant and similar to each other. Only after the implementation of ATM, there was reduction of speed drop events, hence the reduction of estimated crash frequencies for average weekdays and weekends.

Table 36. Model Validation with RNS After-ATM (Oct-Dec 2015) Crash Data (Rear-end and Sideswipe Crashes)

Model Validation with RNS After-ATM Data (Rear-end and Sideswipe Crashes)				
Direction	**Day**	**Estimated Crash Frequency**	**Observed Crash Frequency**	**Percent Error (%)**
EB	Weekday	75	77	3%
	Weekend	23	21	10%
WB	Weekday	57	64	11%
	Weekend	10	8	25%

Table 37. Total Number of Crashes Predicted by the Model for Before-and-after ATM (Oct-Feb 2014-2016, Rear-end and Sideswipe Crashes)

Total Estimated Crash Frequency (Rear-end and Sideswipe Crashes)				
Direction	**Day**	**Before-ATM Oct '14 - Feb '15 (Estimated)**	**After-ATM Oct '15 - Feb '16 (Estimated)**	**Before-After Change (%)**
EB	Weekday	148.00	130.85	11.59
	Weekend	87.64	38.67	55.88
WB	Weekday	113.33	99.30	12.38
	Weekend	53.34	16.25	69.54

Table 38. Weekday Before-and-after ATM Crash Prediction by Time of Day (Oct-Feb 2014-2016, AM and PM Peak, Rear-end and Sideswipe Crashes)

Weekday		Estimated Crash Frequency							
Direction	Day	AM Peak Period (5:30am - 11am) Oct '14 - Feb '15 (# of crashes)	AM Peak Period (5:30am - 11am) Oct '15 - Feb '16 (# of crashes)	Change in AM Peak Period (#)	Change in AM Peak Period (%)	PM Peak Period (2pm - 8pm) Oct '14 - Feb '15 (# of crashes)	PM Peak Period (2pm - 8pm) Oct '15 - Feb '16 (# of crashes)	Change in PM Peak Period (#)	Change in PM Peak Period (%)
EB	Sum	69.89	86.95	17.06	24.41	55.09	22.40	32.69	59.34
WB	Sum	17.14	13.27	3.87	22.58	69.44	70.49	1.05	1.51

Table 39. Weekday Before-and-after ATM Crash Prediction by Time of Day (Oct-Feb 2014-2016, Midday and Overnight, Rear-end and Sideswipe Crashes)

Weekday		Estimated Crash Frequency							
Direction	Day	Midday Period (11am - 2pm) Oct '14 - Feb '15 (# of crashes)	Midday Period (11am - 2pm) Oct '15 - Feb '16 (# of crashes)	Change in Midday Period (#)	Change in Midday Period (%)	Overnight Period (8pm - 5:30am) Oct '14 - Feb '15 (# of crashes)	Overnight Period (8pm - 5:30am) Oct '15 - Feb '16 (# of crashes)	Change in Overnight Period (#)	Change in Overnight Period (%)
EB	Sum	17.59	15.31	2.28	12.96	5.43	6.19	0.76	14.00
WB	Sum	12.62	8.68	3.94	31.22	14.13	6.86	7.27	51.45

Table 40. Weekend Before-and-after ATM Crash Prediction by Time of Day (Oct-Feb 2014-2016, Rear-end and Sideswipe Crashes)

Weekend		Estimated Crash Frequency							
Direction	Day	Peak Period (10am - 8pm) Oct '14 - Feb '15 (# of crashes)	Peak Period (10am - 8pm) Oct '15 - Feb '16 (# of crashes)	Change in Peak Period (#)	Change in Peak Period (%)	Off-Peak Period (8pm - 10am) Oct '14 - Feb '15 (# of crashes)	Off-Peak Period (8pm - 10am) Oct '15 - Feb '16 (# of crashes)	Change in Off-Peak Period (#)	Change in Off-Peak Period (%)
EB	Sum	81.26	32.30	48.96	60.25	6.38	6.37	0.01	0.16
WB	Sum	46.40	10.82	35.58	76.68	6.94	5.43	1.51	21.76

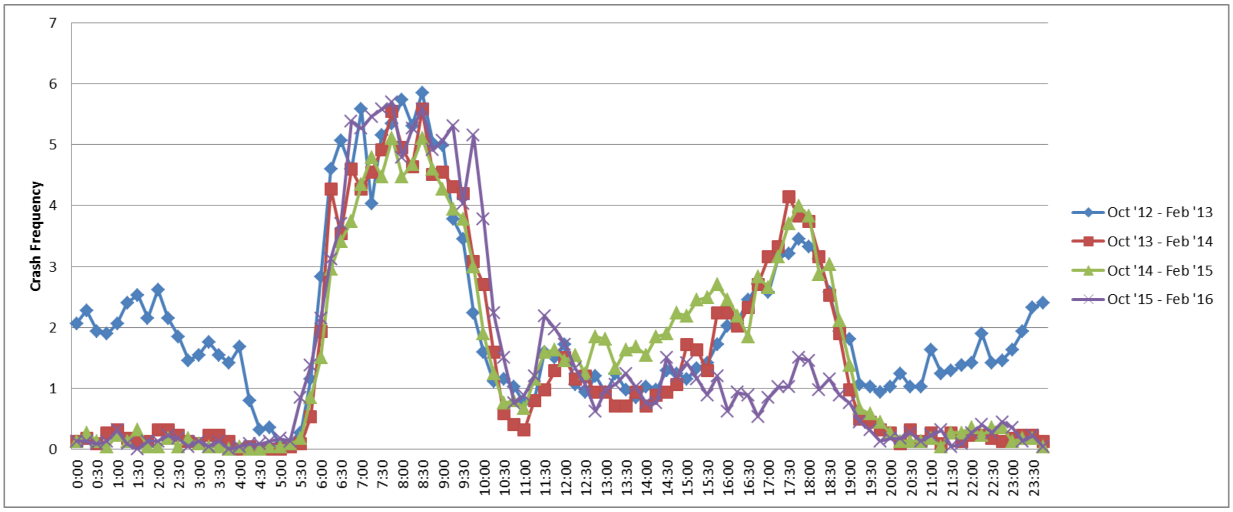


Figure 49. EB Weekday Trends of Estimated Crash Frequency Profile (Oct-Feb 2012-2016, Rear-end and Sideswipe Crashes)

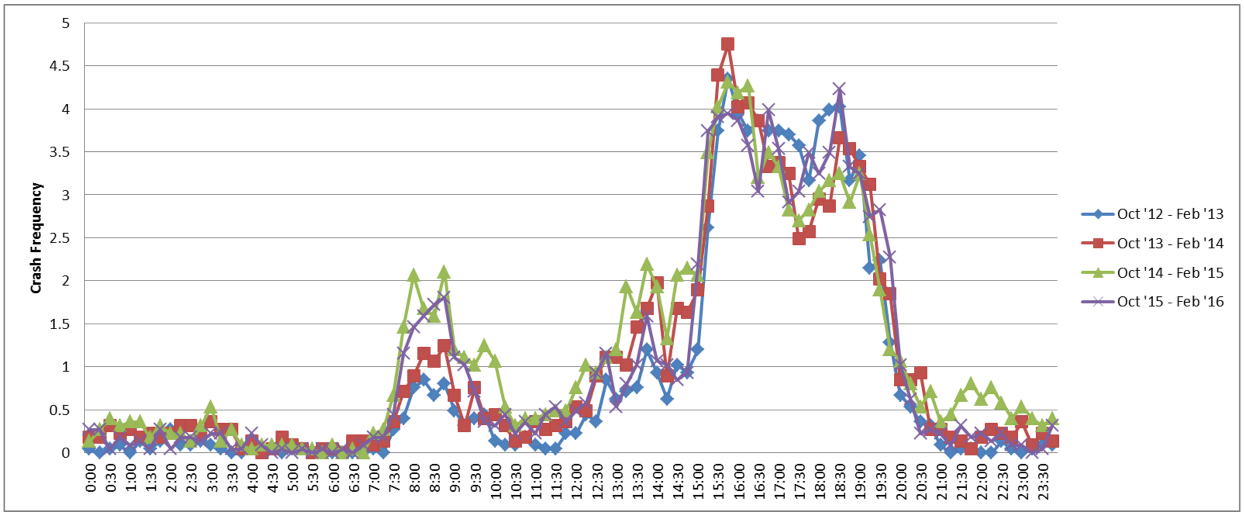


Figure 50. WB Weekday Trends of Estimated Crash Frequency Profile (Oct-Feb 2012-2016, Rear-end and Sideswipe Crashes)

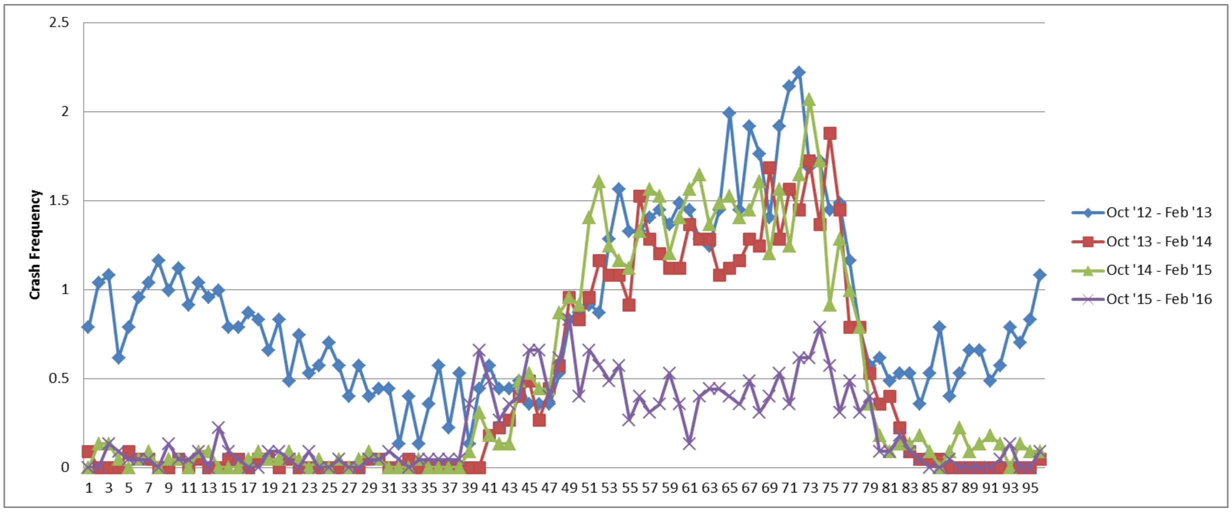


Figure 51. EB Weekend Trends of Estimated Crash Frequency Profile (Oct-Feb 2012-2016, Rear-end and Sideswipe Crashes)

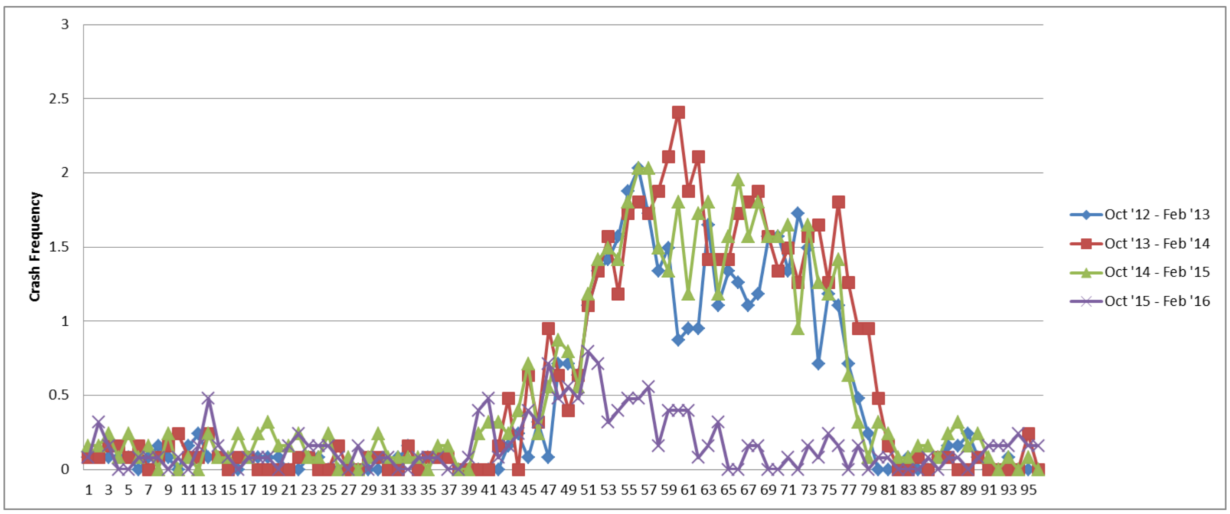


Figure 52. WB Weekend Trends of Estimated Crash Frequency Profile (Oct-Feb 2012-2016, Rear-end and Sideswipe Crashes)

After the implementation of ATM on I-66, there has been a preliminary improvement on both observed and estimated crash frequency. The HSR utilization reduced the likelihood of congestion forming during off peak periods, which seems to produce corresponding safety improvements. The safety improvements were similar to that of the operations improvement as the improvements were most concentrated during weekday midday and off-peak periods and especially during weekend peak periods. While these initial results are very promising, they will need to be confirmed as more data becomes available. VSL utilization may cause additional benefits through advance warning of end of queue, for example.

5.3 SEGMENT-LEVEL OPERATIONS AND SAFETY ANALYSIS

VDOT staff anecdotally indicated that the HSR component of the ATM has been the most active ATM system in operation, especially on weekends. HSR is only present on Segments 4-6, and this analysis will determine if Segments 4-6 had more improvements than other segments in terms of traffic operations. Since each segment has different VMTs, it was most logical to analyze the total recurrent and non-current congestion or total delay levels for each segment to build on the case that HSR was the most influential component of the ATM in improving traffic operations. Segment-level safety analysis will be analyzed by reviewing the trends of estimated crash profiles on each segment.

5.3.1 Segment-level Operations Analysis

The segment-level total delay analysis shows that Segments 4-5 were the segments with the most improvements in mitigating traffic delay. Evaluating the overall delay improvements alone, Segments 4-5 far outperformed other segments and Tables 41-42 shows the full segment-level total delay analysis. The total delay improvements for both directions and all days of the week on Segments 4 and 5 were 2173.9 vehicle-hours per week and 1355.9 vehicle-hours per week respectively. The total delay values were negative (deteriorated) for the other segments which were likely due to increased traffic volume over time. Also, Segments 4-6 showed the greatest improvements in mitigating delay over the weekends for both EB and WB after the implementation of ATM, which is consistent with the fact that average travel times became almost free-flow-like for all weekend hours for both EB and WB.

Table 41. Segment-level Analysis of Total Delay (Segments 1-3, Non-HSR)

Total Delay (min)				
Segment 1		Before	After	Change
EB	Weekday	7756	10508	-2752.48
	Weekend	3102	2629	473.26
WB	Weekday	22362	35675	-13313.02
	Weekend	354	475	-121.14
Total Delay/week				79623.27
Total Delay (min)				
Segment 2		Before	After	Change
EB	Weekday	16124	14668	1455.95
	Weekend	4567	4519	47.68
WB	Weekday	34800	48186	-13386.77
	Weekend	2249	3412	-1162.63
Total Delay/week				61884.01
Total Delay (min)				
Segment 3		Before	After	Change
EB	Weekday	54709	49816	4893.28
	Weekend	7531	2797	4733.25
WB	Weekday	30737	37334	-6596.67
	Weekend	726	2873	-2146.86
Total Delay/week				3344.16

Table 42. Segment-level Analysis of Total Delay (Segments 4-6 HSR)

Total Delay (min)				
Segment 4		Before	After	Change
EB	Weekday	53476	40883	12593.07
	Weekend	28093	7368	20725.06
WB	Weekday	22553	19683	2869.82
	Weekend	7093	1258	5834.91
Total Delay/week				130434.38
Total Delay (min)				
Segment 5		Before	After	Change
EB	Weekday	26846	38577	-11731.03
	Weekend	31625	9478	22146.97
WB	Weekday	112382	104855	7527.45
	Weekend	36923	7885	29038.83
Total Delay/week				81353.71
Total Delay (min)				
Segment 6		Before	After	Change
EB	Weekday	19201	46753	-27551.65
	Weekend	26034	15487	10546.96
WB	Weekday	84462	89263	-4800.54
	Weekend	30205	9104	21101.15
Total Delay/week				98464.76

Most, if not all of traffic operations improvements seemed to occur due to HSR. While LUCs and VSLs may have had some incident management benefits, they did not appear to consistently produce significant improvements in traveler delay.

5.3.2 Segment-level Safety Analysis

It was determined that most of the crash frequency improvements were during the weekend period for EB and WB. The additional HSR utilization has reduced the number of speed drop events on the corridor and the crash frequency that followed. Segments 4-6 showed the greatest estimated improvements and Segments 1-3 did not see much estimated improvement overall in traffic safety. Figures 53-54 show examples of estimated rear-end and sideswipe crash frequency profile at the segment-level for weekends. It is evident that the expected rear-end and sideswipe crash frequency dropped significantly on Segment 4 across all time period after the implementation of ATM. This was not the case on Segment 2 as the before-and-after estimated crash frequency remained approximately the same.

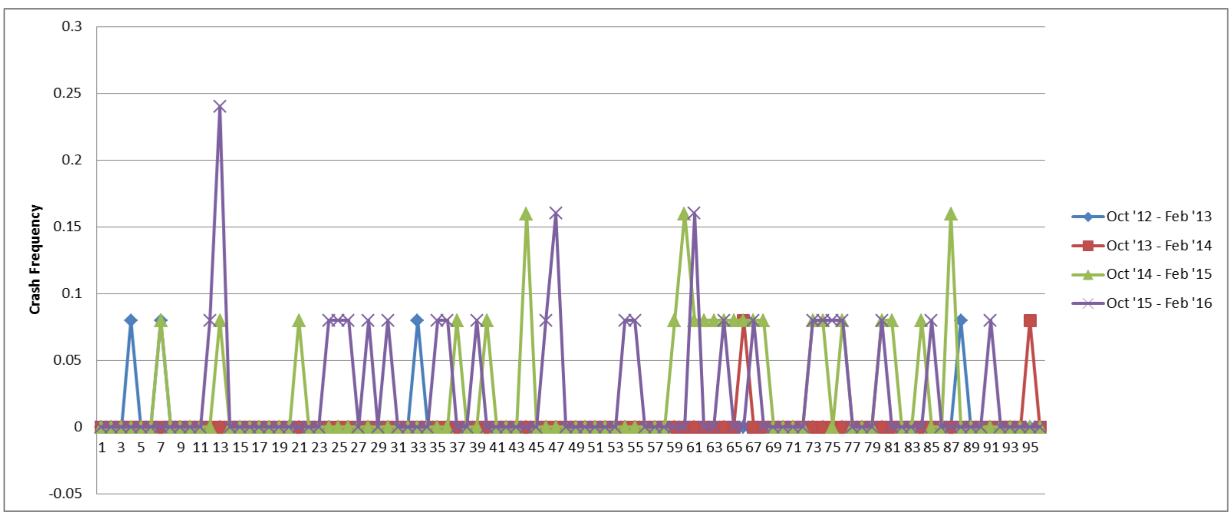


Figure 53. Weekend WB Segment 2 Estimated Crash Frequency Profile

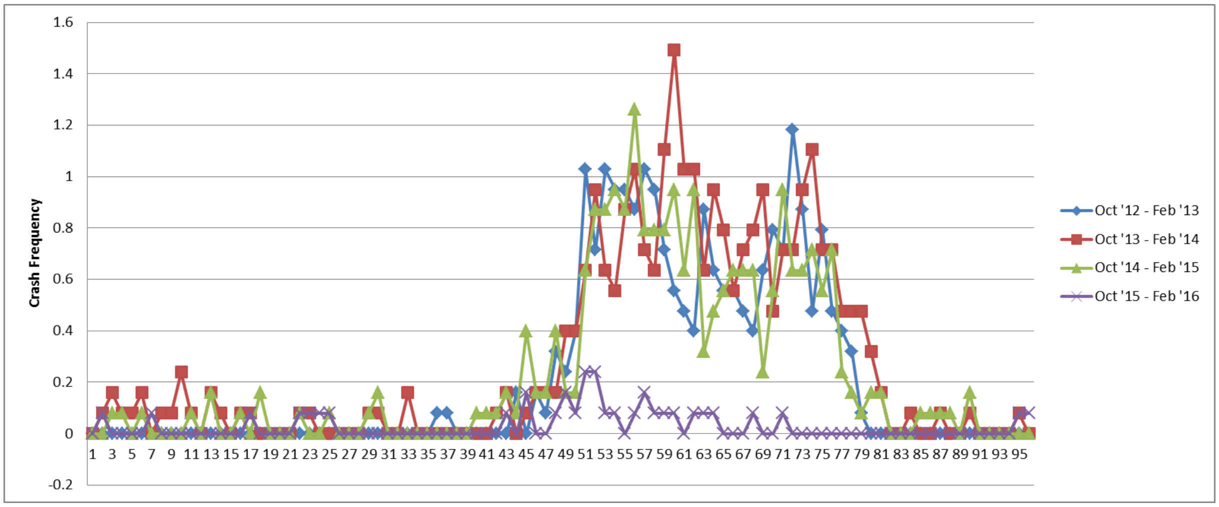


Figure 54. Weekend WB Segment 4 Estimated Crash Frequency Profile

Comparing the before-and-after ATM total rear-end and sideswipe frequency estimated from speed drop events for the before (Oct ’14 – Feb ’15) and after (Oct ’15 – Feb ’16) weekday and weekend periods, there were approximately 119 less rear-end and sideswipe crashes for the after-ATM period than that of before-ATM period. These reductions of speed drop events have caused improvements in rear-end and sideswipe crash estimates for the after-ATM periods. Out of the 119 less rear-end and sideswipe crashes, 98 of the reduction (83%) were from Segments 4-6, where hard shoulders exist. This finding also adds to the claim that most improvements seemed to occur due to HSR. The full segment-level safety analysis on rear-end and sideswipe crash estimates are shown on Tables 43-44.

Table 43. Segment-level Analysis of Estimated Crash Frequency (Segments 1-3, Non-HSR, Rear-end and Sideswipe Crashes)

Total Estimated Crash Frequency (Rear-end and Sideswipe Crashes)				
Segment 1		**Before**	**After**	**Change**
EB	Weekday	12.55	17.21	4.66
	Weekend	2.24	1.60	0.64
WB	Weekday	3.73	2.29	1.44
	Weekend	2.08	0.56	1.52
Total Estimated Crash Frequency (Rear-end and Sideswipe Crashes)				
Segment 2		**Before**	**After**	**Change**
EB	Weekday	11.66	12.33	0.67
	Weekend	3.28	2.80	0.48
WB	Weekday	14.24	14.59	0.35
	Weekend	2.08	2.24	0.16
Total Estimated Crash Frequency (Rear-end and Sideswipe Crashes)				
Segment 3		**Before**	**After**	**Change**
EB	Weekday	35.26	22.90	12.36
	Weekend	15.11	3.92	11.19
WB	Weekday	12.20	13.92	1.72
	Weekend	1.84	1.20	0.64

Table 44. Segment-level Analysis of Estimated Crash Frequency (Segments 4-6, HSR, Rear-end and Sideswipe Crashes)

Total Estimated Crash Frequency (Rear-end and Sideswipe Crashes)				
Segment 4		**Before**	**After**	**Change**
EB	Weekday	35.68	25.92	9.76
	Weekend	15.67	6.64	9.03
WB	Weekday	42.60	33.51	9.09
	Weekend	22.94	2.80	20.14
Total Estimated Crash Frequency (Rear-end and Sideswipe Crashes)				
Segment 5		**Before**	**After**	**Change**
EB	Weekday	27.96	20.82	7.14
	Weekend	16.00	6.48	9.52
WB	Weekday	19.94	14.70	5.24
	Weekend	13.69	4.80	8.89
Total Estimated Crash Frequency (Rear-end and Sideswipe Crashes)				
Segment 6		**Before**	**After**	**Change**
EB	Weekday	29.23	36.24	7.01
	Weekend	37.06	17.60	19.46
WB	Weekday	22.99	22.53	0.46
	Weekend	11.23	4.72	6.51

5.4 BENEFIT-COST ANALYSIS

Benefit-cost analysis was performed at a planning level in order to quantify the benefits of the ATM on I-66. The traveler delay analysis and safety surrogate measures were primarily used to show the ATM benefits. Since there was strong evidence that the ATM improved operations and safety on weekends, only weekend benefits were analyzed for this B/C analysis. There was already a deteriorating operations trend on weekdays, but the rate of deterioration may have been present after the ATM implementation. To have a conservative calculation for this B/C analysis, the improvement in weekday deterioration rate after the implementation of ATM was not considered for this analysis.

Several assumptions were made to develop this benefit-cost analysis. First, it was assumed that the benefits observed during the first 5 months of operation could be extrapolated to the entire year. Second, it was assumed that the benefits observed would remain level over time. Obviously, it is likely that traffic volumes will continue to increase on the corridor, which would in turn impact future year delays and safety. Given the difficulty in forecasting those future year ATM impacts, the assumption for this analysis was to hold benefits level to be conservative. Only user delay and safety benefits were calculated, and no benefits due to decreased emissions or fuel consumption were determined. Likewise, only initial capital costs were considered. VDOT data systems made it difficult to track ongoing maintenance costs for the ATM system, so those were not included.

Using the value of travel time delay used by the Texas Transportation Institute, the operations benefit was quantified. The value of travel time delay is estimated at $17.67 per hour of person travel and $94.04 per hour of truck time (Schrank et al., 2015). For conservative measures, each vehicle is considered to have one passenger on I-66. Overall, there was an improvement of 222,436 minutes of traveler delay combined in both directions of I-66 every weekend. If it is assumed that the trends during the 5 month study period extend over the entire year, this translates to an improvement of 11,566,672 minutes of traveler delay per year. The 2015 VDOT AADT report states that the truck traffic along I-66 were approximately 2%. Using the truck to passenger vehicle distribution rate, the total traveler delay for trucks was determined to be 231,333 minutes and the total traveler delay for passenger vehicles was determined to be 11,335,339 minutes. The total operations benefits were calculated to be approximately $3.7 Million per year based on only weekend improvements.

Crashes can be quantified by injury severity level, and according to the Highway Safety Manual (HSM), the monetary value of reducing a PDO crash and Fatal/Injury crash is $7,400 and $158,200 respectively (AASHTO, 2010). The distribution of Fatal/Injury crashes and PDO crashes on I-66 was approximately 23% and 77% respectively. After the implementation of ATM, the model estimated that the number of both weekend rear-end and sideswipe crashes will be reduced by 86 crashes within 21 weeks. Extrapolating the trend to a full year, the estimated decrease in rear-end and sideswipe crashes was 213 crashes. Using the Fatal/Injury and PDO crash distribution, it was determined that there would be an estimated decrease of 49 Fatal/Injury and 164 PDO rear-end and sideswipe crashes after the implementation of the ATM. The total safety benefits were calculated to be approximately $9 Million per year.

According to VDOT, the cost of implementing the ATM system was $24 Million. The total cost of the project was listed at $39 Million, but the additional $15 Million that was allocated for this project was used to upgrade sensors and cameras that were due for an update anyway. The total operations and safety benefits quantified into monetary values were calculated to be $12.7 Million per year. This means that in less than 2 years, the benefits of the ATM will eclipse the ATM implementation cost. If the project life of the ATM is assumed to be 10 years, the benefit-cost ratio was calculated to be 5.29, which shows that the ATM would be a cost-efficient solution in improving operations and safety on the I-66 corridor. This estimate should be considered a planning level estimate of the benefit-cost ratio of the system given the number of assumptions. Since the B/C ratio exceeded 1 by a large amount, it does appear that the system produced a positive overall net benefit to traffic in the region.

CHAPTER 6: CONLUSIONS AND RECOMMENDATIONS

The main purpose of this thesis was to investigate the effect of the Active Traffic Management System on safety and operations on Virginia's Interstate 66. Interstate 66 has experienced steady growth in traffic volume and average travel times over the years. The Active Traffic Management System was implemented to effectively manage the operations and safety issues associated with the increased demand in roadway capacity without physically expanding the roadway cross section. The key components of ATM that were implemented on I-66 were Variable Speed Limits (VSL), Hard Shoulder Running (HSR), Lane Use Control Signs (LUCs), and Queue Warning Systems (QWS). These ATM components were intended to work jointly to dynamically increase the capacity of the roadway, mitigate recurrent and non-recurrent congestion events, and improve the overall corridor operations and safety. Major conclusions and recommendations from this research are discussed in this chapter.

6.1 CONCLUSIONS

1. Weekday peak periods often saw degraded operations following ATM activation. This deterioration during peak periods was expected as peak period weekday average travel time profiles for both EB and WB have been generally increasing during the last 3 years of before-ATM periods, from 2012-2015. Also, because the shoulders were already open to travel in the peak direction before the ATM system was deployed, the ATM system did not offer any additional capacity beyond what was already in use during pre-ATM conditions.

2. Following ATM activation, weekday off peak periods generally experienced reduced average travel times and improved reliability. Weekday off peak average travel times experienced a statistically significant reduction in the EB direction from 14.66 minutes/vehicle to 13.73 minutes/vehicle (6.35% improvement). In the WB direction, average travel times were reduced from 12.57 minutes/vehicle to 12.29 minutes/vehicle (2.20% improvement). Likewise, there was a statistically significant improvement in PTI of 0.06 (5.45%) in the EB direction and 0.03 (3.33%) in the WB direction on the respective off-peak periods (PM for EB, AM for WB).
3. For average weekend peak periods, the traffic operations benefits were even more evident after the implementation of ATM. The average weekend conditions became almost free-flow-like all throughout the day. For the weekend peak period, there was a statistically significant improvement in average travel times for the EB direction from 14.53 minutes/vehicle to 13.06 minutes/vehicle (10.13% improvement). For the WB direction, average travel times improved from 13.71 minutes/vehicle to 12.25 minutes/vehicle (10.66% improvement). There were also statistically significant improvements in PTI of 0.13 (11.32%) in the EB direction and 0.15 (13.62%) in the WB direction. The total travel delay savings was estimated to be 58,673 minutes per average weekend day EB and 52,545 minutes per average weekend day WB.
4. For rear-end and sideswipe crashes only, weekdays observed either an crash rate reduction or slowed rate of increase in crash rates for the before and after ATM periods analyzed. The crash rates improvements were much more evident on weekends after the implementation of ATM as weekends observed crash rate improvements of 21.51% on

EB and 48.05% on WB compared to that of the before-ATM period. While this is based on limited data, the results seem to be consistent with the operational improvements.

5. Using speed drop events as crash surrogate, frequency of rear-end and sideswipe crashes were predicted for both before and after ATM periods. These estimated crash frequency showed the estimated crash frequency for before-ATM (Oct ’14 – Feb ’15) and after-ATM (Oct ’15 – Feb ’16) periods. On an average weekday, the crash estimates were improved from 148.00 crashes to 130.85 crashes (11.59% improvement) on EB and 113.33 crashes to 99.30 crashes (12.38% improvement) on WB. On an average weekend, the crash estimates were improved from 87.64 crashes to 38.67 crashes (55.88% improvement) on EB and 53.34 crashes to 16.25 crashes (69.54% improvement) on WB.
6. The data showed that HSR, only present in Segments 4-6 of the study corridor, was the primary component of the ATM that contributed to traffic operations and safety improvements during the first 5 months of operation. On average, hard shoulders were open to travel on I-66 an additional 2.5 hours/day on weekdays and 4.5 hours/day on weekends as compared to pre-ATM conditions. This additional lane provided improvements in average travel times, travel time reliability, and reduced the frequency of vehicle-to-vehicle interaction that led to reduction in rear-end and sideswipe crashes.

Based on this analysis, it can be concluded that the I-66 ATM had a positive impact on safety and operations during weekend peak and weekday off-peak periods (PM for EB, AM for WB). The reported operational and safety benefits that I-66 experienced from implementing ATM were very similar to the reported benefits of ATM implementations in Europe and other states in the United States. This is a promising sign as it further supports the effectiveness of ATM on improving operations and safety if implemented on a viable corridor. The system did not create substantial changes during peak periods that were already operating in oversaturated conditions. The planning-level benefit-cost ratio analysis also showed that the monetized operations and safety benefits from implementing the ATM would eclipse the project cost of the ATM in about 2 years. The benefit-cost ratio was 5.29, which showed that the ATM is a cost-effective solution in improving operations and safety on the I-66 corridor.

6.2 RECOMMENDATIONS

1. As this thesis only used five months of after-ATM data for the operations and safety analysis, it is imperative to continue analyzing and monitoring the operations and safety effects of the ATM on I-66. It is important to know if the improvements from implementing the ATM would hold over an extended amount of time. For a more comprehensive operations and safety analysis, 1-year and 3-year after-ATM operations and safety effectiveness evaluations are recommended. This will be required in order to assess the effectiveness of the VSL algorithm on safety.
2. The point sensor database was out of service for the after-ATM period. It is important to analyze the traffic volume changes on I-66 after the implementation of ATM that may have had an impact on operations and safety. Once the point sensor data can be acquired, it is critical that the analysis of traffic volume changes for the after-ATM condition is evaluated to support the findings from this thesis.
3. Given the results of this evaluation, the Virginia Department of Transportation should consider implementing the ATM system on different congested corridors where HSR is feasible. VDOT should examine shoulder depth, lateral clearances, and structure locations

to determine which locations could implement HSR without significant infrastructure changes. HSR usage would appear to offer large potential benefits at locations where it is not presently used during peak periods, as well as during non-recurring congestion events during off peak periods.

4. An addition study to examine travel behavior during VSL and LUCs activation is recommended. The segment-level analysis in this thesis showed HSR to have operational and safety improvements on a macro-level. However, no micro-level analysis was conducted in this thesis. It would be important to understand isolated effects of VSL and LUCs as speed harmonization is a key component of ATM that has been shown to mitigate non-recurrent and recurrent congestions in Europe. Additional analysis of driver behavior on I-66 is needed as greater experience is gained with the VSL system.

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