Analysis of Freeway Traffic Operations using Driver Assistive Truck Platooning Technology

by

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Abstract

Driver assistive truck platooning (DATP) as an application of cooperative adaptive cruise control is expected to have substantial benefits in the freight industry. This FHWA study looks into the benefits of DATP in the freight industry. To investigate the traffic flow impacts of DATP on freeways, traffic microsimulation models are developed for three segments along I-85 near Auburn, AL in the CORSIM software. Parameter variations of time headway, market penetration, and traffic volume were considered.

The results of the simulation were analyzed using the measures of effectiveness of mean speeds and travel time benefits. The statistical significance tests were studied using t- tests, univariate ANOVA, and Tukey’s HSD tests. It was concluded that the traffic stream is most efficient at 100% market penetration, 0.5s headway and at current volumes for the simulation model of a mixed freeway segment. There were no clear trends seen for the basic freeway segment, or for an isolated interchange segment.

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List of Abbreviations

AACC	Automated Adaptive Cruise Control
AADT	Average Annual Daily Traffic
ACC	Adaptive Cruise Control
ALDOT	Alabama Department of Transportation
ANOVA	Analysis of Variance
AWSC	All- Way Stop Control
BFS	Basic Freeway Segment
CACC	Cooperative Adaptive Cruise Control
CORSIM	Corridor Simulation
DATP	Driver Assistive Truck Platooning
DSRC	Dynamic Short Range Communication
FHWA	Federal Highway Administration
HIA	Here-I-Am
HOV	High Occupancy Vehicles
HSD	Honest Significant Difference
I2V	Infrastructure to Vehicle
IIS	Isolated Interchange Segment
ITS	Intelligent Transportation Systems
LCA	Lane Change Assist
LCACC	Leading CACC vehicle platoon
MOE	Measure of Effectiveness
MFS	Mixed Freeway Segment
NHTSA	National Highway Traffic Safety Administration
TSIS	Traffic Software Integrated System
V2V	Vehicle to Vehicle
VMT	Vehicle Miles Traveled
WIM	Weigh-in-Motion

Chapter I

Introduction

Transportation is the movement of goods or people from one place to another. Throughout history, several modes of transportation have evolved, which include transport by roads, rails, water, air etc. Freight trucking is the most common inland transportation mode used for goods. Freight movement along the roadways is anticipated to increase substantially in the future. In the year 2011, 267.2 billion vehicle miles were traveled by freight trucks, which constitutes of 9.1% of the total vehicle miles traveled by all types of vehicles. Assuming that there are no changes in network capacity, it is forecasted that truck and passenger vehicle traffic will increase to expand the areas of recurring peak-period congestion from 10% in 2011 to 34% by 2040. This will slow traffic on 28,000 miles of the national highway system and create stop-and-go conditions on an additional 46,000 miles during the peak periods (FHWA, 2013).

One of the ways to achieve an increase in network capacity of current facilities is through the widening of roads. Widening of roadways is an unreasonably heavy investment venture for state and federal transportation agencies. Even though this measure would increase capacity in the future, it would temporarily require the presence of active work zones along the roadway, which would further decrease the capacity, cause extensive delays in the present, and may also affect safety.

Another means of decreasing congestion and improving roadway capacity without an actual widening is the incorporation of new technologies in the roadway system. The new technologies in the transportation networks that improve the performance of vehicles, roadside infrastructure or better the traffic management are known as Intelligent Transportation Systems (ITS). ITS has a number of different applications such as dynamic message signs, traffic signal control systems, and connected vehicles, among others. Connected vehicles are a part of ITS, which has vehicles sharing important decision making information with one another. They have the ability to improve the entire transportation network with increased efficiency and improved safety performance. By enabling wireless information transfer between vehicles, important information can be transferred like incidents, weather, and emergency services among others. These wireless information transfers are for short range communication, and can be commonly seen these days in the electronic toll collection on selected freeways.

Roadway safety depends greatly on driver attention. The most important task for a driver is paying attention on the road and maintaining the control of the vehicle. Distracted driving is a major cause for vehicle crashes. According to a study by National Highway Traffic Safety Administration (Johnson, 2013), 93% of crashes are attributed to driver error, which may include recognition error, decision error, performance error and non-performance error. Vehicle automation is a notion introduced and implemented to minimize this human error associated with driving. Vehicle automation takes the control away from the driver and assigns it to the vehicle; the vehicle would then be capable of detecting the surroundings with the help of a variety of sensors attached to the vehicle. Then the automated vehicle would make the appropriate judgments for lane maneuvering, braking, accelerating etc. Vehicle automation can be divided into five levels, based on human interference or involvement (NHTSA, 2008). NHTSA defines these five levels as:

- Level 0: No automation, driver is in complete control of the vehicle at all times.

- Level 1: Function specific automation, involving automation of one or more specific control functions. e.g. Cruise Control
- Level 2: Combined function automation, involving automation of at least two primary control functions. e.g. Adaptive Cruise Control (ACC). This is currently seen on the roadway with assistance towards lane centering and blind spot monitoring.
- Level 3: Limiting self-driving automation, which enables the driver to give up full control of all safety-critical functions in certain traffic conditions. It needs the driver to be available for occasional control, with sufficient transition time.
- Level 4: Full self-driving automation, where the vehicle is designed to perform all safety-critical driving functions and monitor roadway conditions for an entire trip. It does not need the driver to be available at all.

The wireless information transfer can be coupled with the current level of automation that is present in the market. Wireless information transfer or vehicle-to-vehicle communication is done between level 2 automated vehicles to obtain Cooperative Adaptive Cruise Control (CACC). When ACC technology is in use, this enables the vehicle to automatically adjust speed to maintain a safe distance from the vehicle ahead. CACC technology allows the vehicles to “talk” to each other by wireless information transfer. This is done by Dedicated Short Range Communication (DSRC), which enables the wireless information transfer of speed, location etc. at high speeds and is immune to extreme weather conditions. With CACC in use, platoons can be formed so that information transfer can take place throughout the platoon. Any incident on the roadway, when detected by the first vehicle, can be transferred to all the platoon vehicles so that they can make decisions regarding speed, braking, and platoon stability without driver being aware.

Using the CACC level of automation would have various impacts on traffic flow, by allowing for reduced headways to increase the capacity, speed etc. These impacts need to be studied thoroughly before they are executed. A traditional field-based before-after study for CACC technology is not feasible because it is associated with a high cost of implementation. Therefore, a tool convenient for use is traffic simulation software. Simulation software has an ability to replicate the real world situations in a computer model. Simulation tools allow researchers to achieve the results at a much faster rate (Katusevski & Hawick, 2009). Traffic microsimulation software can provide more information about the impacts of CACC technology regarding the overall traffic flow. Impacts can be quantified in terms of speeds, delay, and travel time, among others.

1.1 Objectives

This research intends to estimate the traffic impacts of CACC technology in the traffic stream. In this research, the CACC technology is deployed by the heavy duty trucking industry. This particular form of CACC is given the term Driver Assistive Truck Platooning (DATP). In DATP, two or more heavy trucks exchange relevant information, with one or more heavy trucks closely following the leader. This study uses a Level 1 Automation system, in which only longitudinal control is automated; the driver remains fully responsible for steering and has the ability to override system brake or throttle commands at any time. As a part of this project, traffic microscopic simulation models are built to examine the effect of DATP on different types of roadways. The main objectives of this research project are:

- Develop traffic microscopic simulation models for baseline cases of a
    a. A freeway segment with interchanges,
    b. An isolated interchange segment,

c. A basic freeway segment without any interchanges

- Select the parameters to be altered to best capture the effect of DATP in the traffic stream.
- Develop different cases for the parameters selected.
- Assess the impact of DATP on the measure of effectiveness (MOEs) due to these variations in parameters.
- Test the simulations for statistical significance.
- Conclude on optimal levels of DATP technology in roadways for beneficial outputs.

1.2 Scope

Several segments of freeway in the Auburn- Opelika area of Lee County, Alabama were modeled. These roadways served as the base model while using the microscopic simulation software. The sections of roadways in consideration are:

- Interstate 85 between Exit 58 and Exit 62 in Lee County, Alabama.
- Interstate 85 within Exit 60 in Lee County, Alabama.
- Interstate 85 between Exit 42 and Exit 50 in Lee County, Alabama.

These three segments were selected to serve as representative freeway segments, so that the results obtained from this study can be extrapolated to find savings on the actual trucking routes which spread over several thousands of miles. Traffic data for these freeway sections were obtained from Alabama Department of Transportation (ALDOT). Baseline simulation models were created in microscopic traffic simulation software, CORSIM. The three freeway sections under consideration are:

- A 5.3 mile freeway segment, consisting of freeway, interchanges, ramps

- A 0.3 mile isolated interchange segment,
- An 8 mile basic freeway segment, with no interchanges.

An experimental design was developed to study the effect of each parameter on the traffic flow. The three parameters selected for the analysis are time headway between the DATP vehicles, market penetration of the DATP technology into the heavy trucks, and traffic volume. Peak hour of truck traffic is considered for all the freeway analyses. The output parameters that were considered are average speeds of vehicles on the freeway and travel time benefit due to the implementation of DATP technology. The results from all the simulations are then tested for statistical significance using t-tests, multilevel ANOVA and Tukey's honest significant difference test.

1.3 Outline

Chapter Two describes a review of the literature related to CACC technology. This includes the background of CACC technology and associated concepts. The chapter includes a review of research conducted on different methods of simulation of CACC, effects of CACC in traffic flow, autonomous vehicles, platooning of heavy duty vehicles, and effect of CACC on driver behavior. The literature review concludes with a study on management system at intersections to reduce fuel consumption and intersection delay.

Chapter Three describes the methodology accepted for freeway section of the roadway. It begins with justification and description of the sites selected and proceeds to the data collection part. The method used for collection and interpretation of the data obtained from ALDOT is described in this part. The microscopic simulation tool is then explained, along with construction of the baseline model. The chapter then discusses in detail about the parameters that are altered to observe the changes in traffic flow. It then concludes with the statistical tools which are to be used in the results section.

Chapter Four consists of the results which were acquired from the simulation runs conducted for the three freeway sections. The results are shown as average speeds on the freeway segments and travel time benefits. The parameters are then tested for statistical significance using the procedure described in chapter three.

Chapter Five discusses the findings from the above chapters and presents the conclusions from this research. It also provides recommendations and the future work that needs to be done.

Chapter II

Literature Review

Vehicle platooning is a technique where highway traffic is organized into groups of close-following vehicles called a *platoon* or *convoy* (Amoozadeh et al., 2015). Platooning leads the drivers to maintain smaller headways, improve vehicle throughput, safety, homogeneity, and reduces fuel consumption.

The concepts of truck platooning and cooperative adaptive cruise control (CACC) are closely interlinked (Nowakowski et al., 2015). However, there are a few differences present. With CACC, only truck speed is automated, using vehicle to vehicle communication to support forward sensors. The drivers are still responsible for actively steering the vehicle. Another main variation found is that truck platooning mainly takes into account the constant distance gap; however, CACC technology relies on constant time gap control, where the distance between vehicles is related to the speed.

A study conducted by Amoozadeh et al. developed a protocol based on vehicular ad- hoc network and CACC using three basic platoon maneuvers- merge, split and lane change (Amoozadeh et al., 2015). These maneuvers could be used to obtain various platoon operations. The vehicle to vehicle communication is based on single hop beacon messages and event-driven messages for coordination. This protocol ensures traffic flow stability and theoretical vehicle

throughput. The CACC controller and protocol can react to communication loss by degrading to

ACC mode.

The literature review is divided into five parts, which discuss the research that have already been conducted on platooning, traffic flow, driver behavior, traffic flow stability and microsimulation models.

2.1 Platooning

A study was conducted to examine the influence of time headway kept in platoons of vehicles on the other drivers nearby (Gouy et al., 2015). Thirty vehicles were taken in as participants and asked to follow a lead vehicle in three different surrounding traffic conditions: platoons with short headway (time headway of 3 seconds, or THW03), platoons with large headway (time headway of 14 seconds, or THW14) and no platoons. This was a study conducted in live traffic. It was observed that participants maintained an average smaller headway during the first condition. It was also seen that 35-45% of the drivers drove below the safety threshold of 1 second headway. It was also seen that the presence of truck platoon maintaining short time headways in traffic have a significant influence on drivers' performance. Drivers were biased to reduce their time headway towards the lead vehicle while driving in the vicinity of a platoon, which increases the probability of collision due to short headways.

In a study explaining formation of high performance vehicle streams using CACC, it was mentioned that instead of using vehicle to vehicle (V2V) communication CACC, infrastructure to vehicle (I2V) CACC would be more predominant on arterial roads (Shladover et al., 2014). V2V CACC can be implemented in signalized intersections only if there is a coordinated start of vehicles that have been stopped at the signal that changed from red to green. It would also require a string of CACC vehicles to be platooned; else a single unequipped vehicle may break the string, making market penetration an important factor.

Arnaout and Bowling conducted a study that demonstrated the potential of reducing traffic congestion at low CACC penetration levels (Arnaout & Bowling, 2014). This study presented a progressive deployment approach to reduce traffic congestion at low CACC to place these vehicles in high occupancy vehicles (HOV) lanes until the market penetration reaches 40%. As the market penetration goes above 40%, the scattering of all vehicles in the freeway lanes would show a significant improvement in highway capacity due to CACC.

2.2 Traffic Flow

Van Arem et al. simulated the impact of CACC on traffic flow characteristics (van Arem et al., 2006). To study the potential impacts of CACC systems, MIXIC simulation software was used to take into account the stochastic nature of the traffic flow along with the driver performance, traffic safety, exhaust-gas emission and noise emission. MIXIC uses real traffic measurements (time instant, lane, speed, and vehicle length) to generate traffic at the start of the simulation run. The study concluded that CACC shows potential positive effects on traffic throughput by the reduction in the number of shockwaves, and increase in capacity at a lane drop from 4 lanes to 3 lanes. The traffic performance was seen to increase with the market penetration due to CACC platoon formation, improving string stability. A low CACC market penetration (<40%) led to degradation of performance, demonstrated by lower speeds and higher speed variances. A negative impact observed was that formation of platoons prevented vehicles from cutting in, resulting in demand for increased market penetration.

VanderWerf et al. studied the effects of adaptive cruise control on highway traffic flow capacity. Three types of vehicle capabilities were evaluated: manually driven vehicles, vehicles with autonomous ACC (AACC) having a 1.4s constant time headway, and cooperative ACC (CACC) with more tight 0.5s time headways. AACC vehicles were studied to see if they would cause an increase in highway capacity if they are given a priority access to special lanes. It was concluded from this project that AACC increased the highway capacity by only 7% as compared to normal conditions, when the market penetration was 20-60%, and time gap setting of 1.4s. Also, it was mentioned that increasing the market penetration to 60% may cause a decrease in capacity, as people do not tend to drive in such high gaps. It was also established that CACC systems using V2V communication are able to produce much shorter headways of 0.5s, thereby having a potential to produce much higher highway capacities.

In a study conducted to analyze the impacts of CACC on freeway traffic flow by Shladover et al., four types of vehicles were simulated: manually driven, ACC, Here I am (HIA) vehicle (driven manually and has a radio feature to broadcast location and speed, which enables it to behave as CACC if it is followed by a CACC vehicle), and CACC (Shladover et al., 2012). The road geometry adopted for the simulation was a single lane freeway 6.5km long and having a speed limit of 105km/h (65mi/h). The results from this study concluded that use of only ACC vehicles would be able to increase the capacity of the road only from 2000 veh/h to 2100 veh/h. However, use of HIA vehicles and a modest market penetration of CACC vehicles show a significant increase in the capacity. With a 20% market penetration of CACC vehicles, addition of HIA vehicles increase the capacity by 7%, 30% CACC increases it more than 10%.

2.3 Driver Behavior

In a study conducted by Nowakowski et al., impact of CACC vehicles on drivers, highway traffic flow capacity and stability was estimated by testing drivers' choices in the field (Nowakowski et al., 2012). To conduct this test, 16 random drivers were selected and subjective reactions and their adaptability towards ACC or CACC vehicles were noted. The results pointed out that time gap setting of 1.1s was most frequently used for ACC system. A gender bias was observed, with males preferring the shortest time gap setting (<0.5s) while females preferred the middle time gap setting (1.6s). Taking into consideration the CACC system, the shortest time gap setting was of 0.6s. Most males were shown to select 0.6s while females were more inclined towards a 0.7s setting. It was concluded that participants of the study were comfortable with both the ACC and CACC systems. However, they tend to give preference to ACC in light and moderate traffic and switch to CACC in heavy traffic. With shorter gaps safely maintained, CACC provide an opportunity to improve traffic flow density and efficiency without compromising safety or other roadway improvements, however, adaptability of drivers to the technology is also an important factor.

2.4 Traffic Flow Stability

In a study performed by Gu et al., a new leading vehicle model was proposed for CACC vehicle platoons (LCACC) (Gu et al., 2015). It consisted of a bi- directional framework to integrate CACC following behavior and following CACC vehicle comfort. Four different car following models were used to describe the manually driven behavior of the vehicles and tested for 3, 5 and 10 vehicle platoons. It was found that both the CACC and the LCACC vehicles are found to have a stabilizing effect on manual driving vehicles, as well as provide stability in the traffic flow.

Schakel et al. studied the effects of CACC on the traffic flow stability. This was done by evaluating shockwave characteristics in the traffic flow (Schakel et al., 2010). Shockwaves are a result of congestion and queueing. With the use of CACC technology, the response time for the vehicles decreases, thereby reducing the length of shockwaves. In this paper, the stability of traffic flow is noted at different levels of penetration of CACC. The results showed that CACC would increase the initial deceleration of a shockwave; which would shorten the duration and lengthen its range. It was also noticed that lane changes were the main reason for creation of shock waves. Another conclusion from this paper is that human drivers may face a challenge to anticipate the shockwave speeds and behave differently, which may cause serious implication on traffic safety.

Bareket et al. presented a methodology to assess the behavior of adaptive cruise control systems (Bareket et al., 2003). This was accomplished by measuring the ACC system performance, which was followed by modeling and simulation of measured ACC performance. Three representative ACC systems were created to study the parametric variations. 'Quick' and 'Slow' responses indicate the use of higher and lower acceleration rates respectively. It was found that with the present characteristics of ACC vehicles, substantial overshoots in velocity and range clearance was seen in response to change in the velocity of the preceding vehicle. The authors asserted that research on ACC vehicles would be able to substantially increase the traffic flow during high traffic density.

2.5 Microsimulation Models & Traffic System

Elefteriadou et al. published a report titled "Using microsimulation to evaluate the effects of advanced vehicle technology in congestion", in which CORSIM was used as a microsimulation tool for the implementation of advanced vehicle technology into the simulation model (Elefteriadou et al., 2009). Two types of advanced technology used were Adaptive Cruise Control (ACC) and Lane Change Assist (LCA). A test network was developed in CORSIM to assess the relative impacts of these technologies on congestion. The drivers were assumed to keep the technology on at all times. With the simulation results, it was noted that with the implementation of ACC, congestion is eliminated even for a market penetration of 20%

(decrease in volumes from 1531 to 1255 vehicles per hour per lane). In presence on LCA technology alone, there was an increase in lane change maneuvers and VMT, although the travel time did not change significantly. With implementation of both the technologies together, the initial congested conditions significantly improved.

In a paper titled Simulation Framework for Vehicle platooning and Car following behaviors under Connected Vehicle Environment, a 4km stretch of a 2 lane freeway is used for the simulation (Zhao & Sun, 2013). Three types of vehicles, namely, ACC, CACC and manually driven vehicles were used for analysis, which consisted of three different modes of operation were chosen of analysis: manual vehicle driving, single ACC or CACC driving, and CACC platoon driving. This paper analyzes the effect of different percentages of market penetration of CACC vehicles on traffic flow stability. The time headway was varied between 0.5s and 1.4s in all the simulations. The penetration rates were changed from 10-100% in multiples of 10 and platoon sizes from 1-10. The analysis demonstrated that traffic capacity increased significantly with the increase of market penetration of CACC technology. However, it was also noticed that traffic capacity scarcely increased with the increase in the number of vehicles by platoon size. This implied that higher platoons may reduce the traffic flow if lateral movements were accounted for.

Zohdy et al. presented an intersection management system for autonomous vehicles using CACC technology, to reduce the fuel consumption and intersection delay (Zohdy et al., 2013). The aim of this project was to use the vehicle communication and automation technology to replace the traffic control signals at intersections. The new technology was named iCACC and compared with all-way-stop-control (AWSC), roundabouts, and conventional traffic signal. Volume to capacity ratio was varied between 0.27 and 0.91. Simulation models were developed to measure the average vehicle delay and their fuel consumption. The results show that the iCACC average delay and fuel consumption values are comparable to the ordinary traffic in roundabouts, but are much lower to the AWSC and traffic signals. iCACC observed savings in average vehicle delay and fuel consumption in the order of 90% and 45% respectively than the traffic signal or AWSC. This technology, although effective, would work only with complete connected vehicle technology on the roads, with V2V and V2I communications in place.

2.6 Summary

This chapter describes a review of the literature related to CACC technology and associated concepts. This chapter includes prior research related to how the platooned vehicles behave in a traffic stream and provisions on the roadway for safe movement of platoons. Effects of platoons on the traffic flow and responses of drivers driving in platoons have been discussed in this chapter. Additional literature regarding the stability and reliability achieved by CACC vehicles was explored. Different methods of using CACC for microsimulation were also studied.

With most of the research focused on passenger vehicles only, minimal research is conducted on the freight trucks with new technology. The previous literature provides information about the research conducted either with the technology or freight trucks or related simulation studies. This research aims to find how the CACC technology would work in freight truck platooning, by the use of CORSIM for simulation.

Chapter III

Methodology

3.1 Introduction

This chapter begins with describing the selection of suitable sites for carrying out the DATP simulation study. The chapter then provides necessary guidance on the data to be collected and obtained from different sources. The traffic microsimulation modeling softwares used for this project are described; followed by showing the simulation of baseline model in CORSIM. The parameters that need to be varied for modeling purposes are introduced. The chapter then concludes with explanation of the statistical testing methods.

3.2 Site Selection & Data Collection

There were three study segments taken into consideration as representative segments. These sections were: a 5.3 mile freeway segment consisting of freeways and interchanges, a 0.3 mile segment of an isolated interchange, and an 8 mile segment of basic freeway segment, without any interchanges. These segments were chosen as they were present in small urban or rural area, and in close proximity, and had some data availability.

The first freeway section that was taken into consideration is a segment with several interchanges in a small urban area. This is the section which is in close proximity to Auburn, thereby making the process of data collection more convenient. A 5.3-mile stretch on Interstate 85 was considered as the study segment for the mixed freeway segment, which started just south of Exit 58 and continued to just north of Exit 62 in Opelika, Alabama, as shown in Figure 3.1.

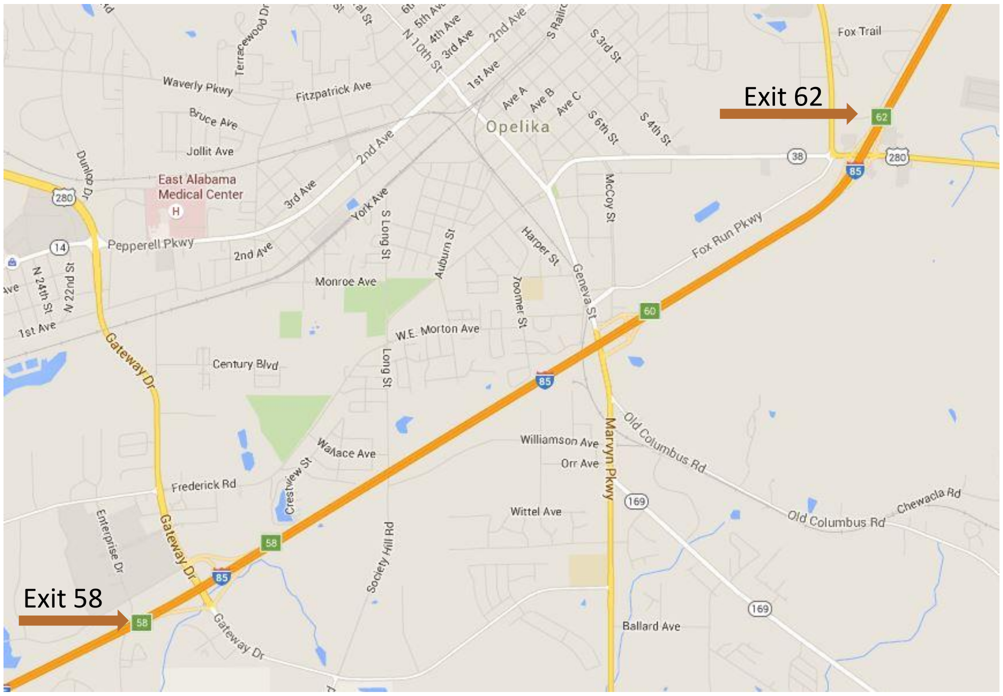

Figure 3.1: Site selection for Simulation of Mixed Freeway Segment

This study simulated the peak hour truck traffic for the 5.3-mile segment. 24- hour traffic data for this segment was not recorded by ALDOT. Thus it was not possible to identify and utilize the peak hour of truck traffic data information. Therefore, an assumption was made to apply the distribution of truck traffic by the time of day from one of the weigh-in-motion (WIM) stations located close to 25 miles to the south of the study segment. The weigh-in-motion data that was used was collected for the entire month of March in 2015. The same percentage distribution was used for the different types of vehicles from the WIM station and applied to the traffic volume on the study segment. The vehicle class distribution data by time of day was assumed to not have much variation from the actual traffic in the study section as the area between the section and the weigh station does not have a particular destination for freight trucks. The peak hour truck traffic was calculated from the data obtained from the weigh-in-motion stations. After the peak hour of truck traffic was calculated, the traffic volume of the segment of I-85 between 58 and 62 in that hour was sorted, and the percentage distribution of vehicle types that was obtained from the weigh-in-motion data was applied to this data. The segment under consideration consisted of three interchanges, which were diamond interchanges (Exit 58 and 62) and partial cloverleaf interchanges (Exit 60). The traffic volume on these on and off ramps helped to determine the division of traffic in the beginning of each interchange and the percentage distribution in the on ramps.

In this study, peak hour volume of all the traffic (obtained from AADT) is assumed to be equal to the peak hour volume of truck traffic. This assumption is made to accommodate for the least favorable condition of having the maximum number of freight trucks in the peak hour. This is done by applying the percentage distribution by vehicle class in the peak hour of truck traffic to the peak hour volume.

Table 3.1 shows the traffic count definitions which were used to find the peak hour volume of truck traffic. The peak hour traffic can calculated as a multiplication of AADT, K factor and D factor, and vehicle type distribution of maximum heavy trucks applied, as obtained from the weigh-in-motion data. Thus,

Peak Hour Volume= AADT x K x D

Table 3.1: Traffic Count Definitions for Peak Hour Volume calculations

AADT	The annual average daily traffic count for the segment represented (Total of all vehicles counted in a year divided by 365 days). AADT is calculated annually for all highway segments.
K	Design Hour Volume defined as the 30th highest annual hourly traffic volume expressed as a percentage of AADT. In this study, it is used as the proportion of daily traffic occurring in the peak hour.
D	The percentage of the design hour value flowing in the peak direction.

The data obtained from ALDOT for the peak hour truck traffic on the ramps are provided in Table 3.2.

Table 3.2: Ramp data for Exits 58-62 on I-85

Ramp no.	Off Ramp Volume	On Ramp Volume	On Ramp Truck percentage
58	274	418	33.2
60	152	109	23.8
62	547	125	11.2

The K factor used for the data obtained from ALDOT was 0.1 and the directional distribution factor D used was 0.53. With an AADT value of 40,660 vehicles/day, the peak hour truck traffic was calculated as 2155 vehicles/ hour, with 23% occupancy by freight trucks, as per FHWA classification. FHWA classification consists of 14 different vehicle types, as shown in Table 3.3 (FHWA, 2014).

Table 3.3: Vehicle Classification by FHWA standards

Class Group	Class Definition
1	Motorcycles
2	Passenger Cars
3	Other Two-Axle Four-Tire Single-Unit Vehicles
4	Buses
5	Two-Axle, Six-Tire, Single-Unit Trucks
6	Three-Axle Single-Unit Trucks
7	Four or More Axle Single-Unit Trucks
8	Four or Fewer Axle Single-Trailer Trucks
9	Five-Axle Single-Trailer Trucks
10	Single-Trailer Trucks
11	Five or Fewer Axle Multi-Trailer Trucks
12	Six-Axle Multi-Trailer Trucks
13	Seven or More Axle Multi-Trailer Trucks
14	Unclassified Vehicle

It can be noted from the table above that vehicle class group 5-13 are freight trucks. The data that was made available by ALDOT consisted of vehicles divided in these categories. Input required by CORSIM however has a different type of vehicle classification, as shown in Table 3.4 (CORSIM, 2011).

Table 3.4 Vehicle classification by CORSIM

Fleet Component	Vehicle Type	Length (default, ft)	Occupants per 100 vehicles (default)	% Fleet Component (default)
Passenger Car	1= Low Performance	14	130	25
	2 = High performance	16	130	75
Truck	3 = Single unit	35	120	31
	4 = Semi-trailer with medium load	53	120	36
	5 = Semi-trailer with full load	53	120	24
	6 = Double-bottom trailer	64	120	9
Bus	7 = Conventional	40	2500	100
Carpool	8 = Low performance	14	250	25
	9 = High performance	16	250	75
Advanced Technology	10 = Low performance	14	130	25
	11 = High performance	16	130	75

Thus, vehicles types 3-6 in CORSIM consist of heavy trucks. For overlapping of FHWA vehicle classification on CORSIM vehicle type, the following assumption was considered, as shown in Table 3.5. The vehicle type 1 & 2 consisted of 77% of the vehicles and vehicle types 3-6 consisted of 23% of all the vehicles, i.e. heavy trucks. With this calculation, the percentage distributions within the heavy trucks are also shown in the table.

Table 3.5: Overlapping of vehicle classification and vehicle types

FHWA Vehicle Classification	CORSIM Vehicle Type	Percentage distribution within heavy trucks
5-7	3	30
8-10	4 & 5	69
11-13	6	1

The second simulation model was that of an isolated interchange. The isolated interchange used for this simulation was Exit 60 of the previous study segment, devoid of any basic freeway segment as shown in Figure 3.2. Exit 60 has partial cloverleaf interchange geometry. The data used for this model is the same as the previous model with the mixed freeway segment. The factors used for this model is the same as the previous model, with K being 0.1 and the directional distribution factor, D, being 0.53. With an AADT value of 40,660 vehicles/ day, the peak hour truck traffic was calculated as 2155 vehicles/hour, with 23% heavy trucks.

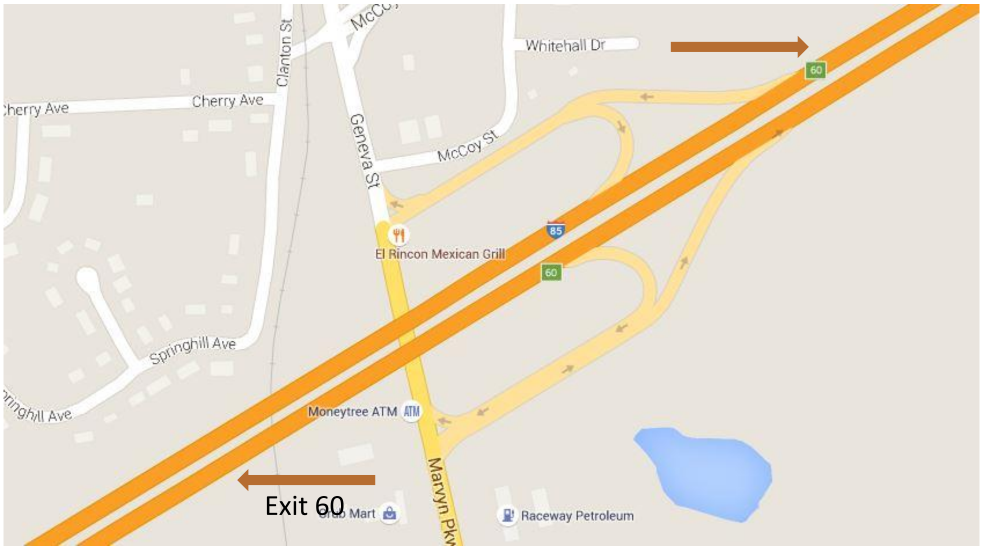

Figure 3.2: Site selection for Simulation of Isolated Interchange Segment

The third simulation model is an eight mile segment of roadway, from Exit 42 to Exit 50 on I-85. This model is a basic freeway segment, without any interchanges or other access points in the considered segment. The data used for this study is taken from the ALDOT website, using the same procedure as the previous models with K and D factor of 0.10 and 0.54 respectively. Figure 3.3 presents the study segment for the basic freeway segment.

Figure 3.3: Site selection for Simulation of Basic Freeway Segment

3.3 Baseline Model

The baseline model was built to simulate the existing conditions on I-85 into the traffic simulation software, CORSIM. CORSIM has two interfaces where the simulation work can be performed, TSIS 6.3 and TSIS Next. TSIS 6.3 has a graphic user interface to accommodate all values required to be input for creating the simulation model. Thus the basic network structure was created in TSIS 6.3. I-85 segment of the model is a freeway; however the ramp segments are arterial segments. Dummy nodes are created to join the freeway segment to the arterial segment of ramps. This could be seen in Figure 3.4. Grey links/ nodes represent freeway part of the network, hollow nodes represent dummy nodes to connect freeways to arterials and black links/ nodes represent the arterial part of the network.

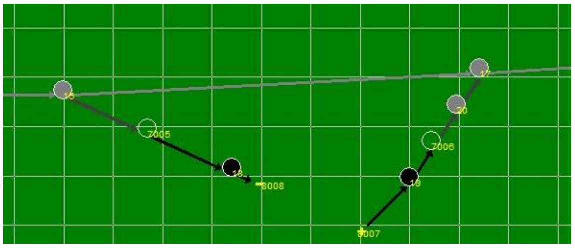

Figure 3.4: CORSIM network for Exit 62 on and off ramp on I-85

However, TSIS 6.3 does not allow for the inclusion of advanced technology (in this case, CACC) vehicles. Thus, TSIS Next is used after the basic model is created, to input the advanced technology characteristics into the model. TSIS Next consists of the entire model framework in a text format, with a capability of altering the CACC parameters using the text editor.

Two other networks are modeled alongside this network, to study the effects of CACC enabled vehicles in an isolated interchange and a freeway segment without any interchanges. These models are altered from the baseline model to accommodate for the effects of an isolated interchange and a basic freeway segment. The isolated interchange under consideration is the partial cloverleaf at the exit 60 of the freeway segment in I-85, as shown in Figure 3.5.

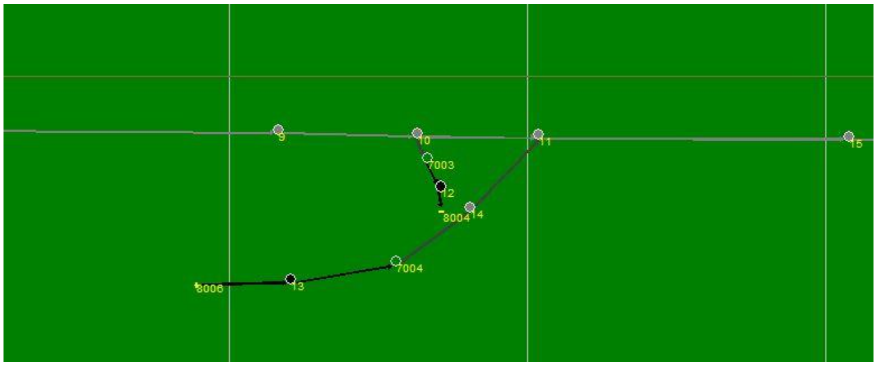

Figure 3.5 Isolated Interchange CORSIM model

For the basic freeway segment modeling, a segment of the freeway on Interstate 85 is considered between Exit 42 and Exit 50, which is an 8-mile long segment, free of any interchanges, as shown in Figure 3.6.

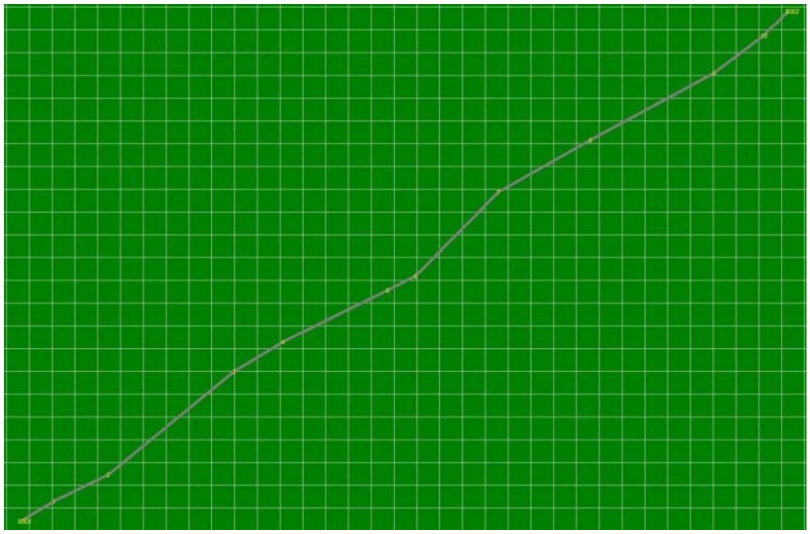

Figure 3.6 Basic Freeway Segment CORSIM model

3.4 Model Parameters

There are various parameters taken into consideration while altering the model inputs to obtain the desired outputs. The parameters that are varied from the baseline case would be difference in time headways, difference in the percentage of advanced technology usage in the vehicles, known as market penetration, and the traffic volumes in the network. Time headway is an important factor to consider in the cooperative adaptive cruise control (CACC) technology implementation. With this technology, the vehicles communicate with each other, therefore being more aware of what is in front of the first vehicle and appropriately speeding or braking when required. Thus, the communication makes it easier to maintain close headways. Lesser headways result in higher capacity of the roadways. Market penetration would be an important consideration for this as well. With more vehicles which are CACC enabled in the system, there is more potential for decreasing headways for more people, thereby increasing the capacity. As the capacity is increased, it is only fair that the model is tested for future increase in traffic volumes too. Thus, the traffic volumes are also altered to accommodate for future demands.

Driver characteristic is the most important factor in determining the time headway parameter in the models. In CORSIM, there are 10 basic driver types, and they are assigned at random to the drivers while the runs are conducted. These driver types are associated with a car following sensitivity factor, which is found in Record Type 68, shown in Table 3.6 and Table 3.7.

Table 3.6: Default distribution of car-following sensitive factors without Adaptive Cruise Control

Driver Type	1	2	3	4	5	6	7	8	9	10
Sensitivity Factor	1.25	1.15	1.05	0.95	0.85	0.75	0.65	0.55	0.45	0.35

Table 3.7: Default distribution of car-following sensitive factors with Adaptive Cruise Control

Driver Type	1	2	3	4	5	6	7	8	9	10
Time Headway	2.00	1.84	1.68	1.53	1.38	1.22	1.06	0.91	0.75	0.60

For this model, it is assumed that all the drivers that do not have the facility of advanced technology, specifically ACC implemented in their vehicle, have a default variable time headway/ sensitivity factor, ranging from 1.25 seconds to 0.35 seconds. Vehicles which have the advanced technology feature have the same time headway for all ten driver types. These advanced technology vehicles (freight trucks, in this study) are forced to come into the system as two truck platoons. The parameters that are altered for better efficiency of the roadway network and their values can be summarized in Table 3.8. The values of market penetration are varied from 0% to 100% to recognize the effect of maximum use of DATP technology in the traffic stream. Traffic volumes are varied to account for the future increase in traffic volumes. Time headways considered for this study are taken to be less than 1.5 seconds, which is the average headway of the traffic stream at maximum capacity. Thus, 1.25 seconds is the upper limit. Literature supports a value of 0.6 seconds comfortable to drive for ACC equipped vehicles (Nowakowski et al., 2012) Thus, a slightly lesser value of 0.5 seconds is adopted in this study.

Table 3.8: Altered parameters and their values

Parameter	Values
Market Penetration	0% (Baseline), 20%, 40%, 60%, 80%, 100%
Time Headway	Default (Baseline), 1.25 s, 1.00 s, 0.75 s, 0.50 s
Traffic Volume	100% (Present Conditions), 115%, 130%

The traffic volumes for this segment were 1080 veh/hour/lane to represent the current traffic conditions, 1240 veh/hour/lane for a 15% increase in traffic volume, and 1400 veh/hour/lane for a 30% increase in traffic volume. The other two parameters, market penetration and time headway can be altered in Record Types 25, 50, 68, 70 and 71, as can be seen in the figures below. Figure 3.7 shows the percentage of vehicles that follow the main stream and exit ramps. In the first box, 87% of the vehicles travel on the freeway, and 13% of the vehicles take the exit ramp. Figure 3.8 depicts the traffic volume on entry ramps, and the percentage of heavy trucks in those entry ramps. Record type 68 in Figure 3.9 shows the car following sensitivity factor for the vehicles with and without the CACC capability. Entry 18 in Record type 70 gives the percentage of vehicles that are a part of CACC technology. Record type 71defines the type of CACC vehicles and the percentage distribution in those vehicle types.

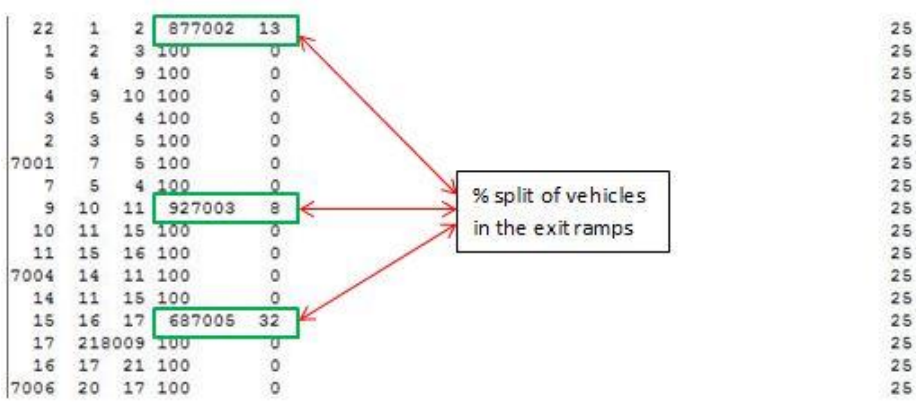

Figure 3.7: Record Type 25 alterations

Figure 3.8: Record Type 50 alterations

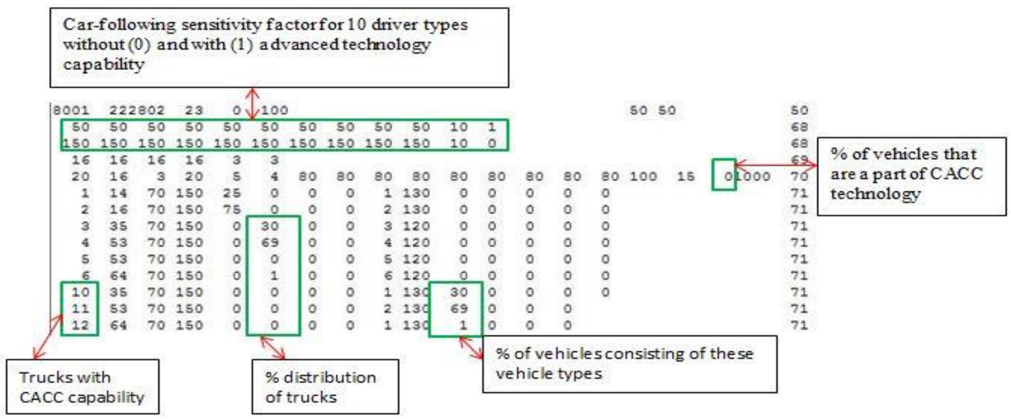

Figure 3.9: Record Types 68, 70, 71 alterations

All the vehicles that are CACC enabled in the CORSIM code are assumed to form a two-truck platoon, as shown in the Figure 3.10. However, this assumption may not be true throughout the simulation.

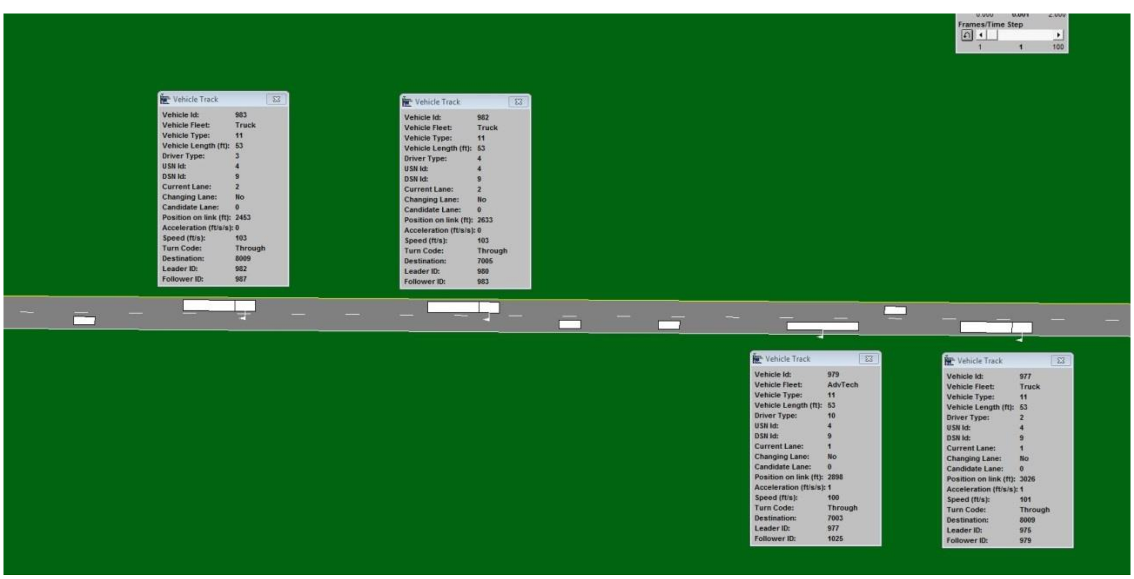

Figure 3.10: CORSIM screen capture showing platooning of advanced technology vehicles

3.5 Statistical Analysis

After all the simulations runs are conducted, the results are checked for their statistical significance. The two important questions that the statistical analysis addresses are-

- Is there a statistically significant difference between the average speed of the traffic stream with a portion of heavy trucks being DATP capable and the average speed of the traffic stream in the baseline case, which is at no market penetration, and at default time headways? Is there a statistically significant effect of any of the parameter levels (time headway, market penetration, or traffic volume) on the traffic stream?

These two research questions are answered by the statistical tests conducted. The methods of statistical analysis conducted on the results are testing for significant differences between the two means described above (t-test) and univariate ANOVA. For the mixed freeway segment model, 10 runs per combination of headway, market penetration, and traffic volume were conducted. With 63 combinations including the base case at each traffic volume level, 630 models were run to capture the stochastic nature of the traffic flow. A hypothesis test is conducted to observe if there is a statistically significant difference between any combination of time headway and market penetrations and the baseline model, called the t-test. A t-test first calculates the t-statistic and then the p-value, as shown below. A p-value is the probability of finding the observed, or more extreme, results when the null hypothesis is true. A one tailed t-test was used in this study, because it was expected that the values of speed would increase with the implementation of CACC, implying closer headways or higher market penetration. As an example, consider a model with CACC market penetration of 20%, headway of 1.25 seconds between vehicles and traffic volume of 100% as the first case. The null hypothesis while conducting the t-test is, there is no significant difference between the baseline model with no CACC implementation and the

CACC model with 20% market penetration, 1.25 seconds headway and at a traffic volume of 100%. The alternate hypothesis is that there is a significant difference between a baseline model with no CACC implementation and the model with 1.25 seconds headway, 20% market penetration and at 100% traffic volume. Ten observations were recorded for each set of parameters. A two sample one tailed t-test was conducted for the entire set. The analyses of t-tests are done using Microsoft Excel. T-statistic was calculated using the following formula:

$$t = \frac{\overline{x_1} - \overline{x_2}}{\frac{s_1^2}{n} + \frac{s_2^2}{n}}$$

Where, t: t statistic

$\overline{x_1}$, $\overline{x_2}$,: mean of the samples

s: standard deviation of the sample

n: sample size

The t- statistic value is then compared to a critical value obtained from the t-chart statistics distribution, and the p values are calculated and checked for statistical significance. The statistical significance is checked at α= 0.05 and α=0.01 respectively. If the p value of the model is less than 0.05, the model rejects the null hypothesis and is statistically significant at α= 0.05. With a p value greater than 0.05, the model fails to reject the null hypothesis and is not statistically significant. Similar conclusions are drawn when α=0.01.

After the significance tests are conducted, univariate analysis of variance (Univariate ANOVA) tests are carried out to determine if the models have a statistically significant effect on the traffic flow, individually, or with an interaction with other parameters. Univariate ANOVA conducts multiple comparisons between the parameters (headway, market penetration and traffic volume), to see if there is a statistically significant difference between the levels of a parameter, keeping the other parameter constant. The null hypothesis in this case is that there is no difference between averages of travel time benefits between the different levels of a parameter, i.e.

$H_0 = \mu_1 = \mu_2 = \mu_3 = \mu_4 = \mu_5$.

As an example, keeping traffic volume and time headway constant, a change in the market penetration would not have a statistically significant effect on the traffic flow. The alternate hypothesis would then be that there is a difference between at least one change in market penetration level will cause a statistically significant effect on the traffic flow. IBM SPSS is used for the doing the statistical testing using univariate ANOVA. A univariate general linear model (GLM) is analyzed, with all possible pairwise interactions. The dependent variable is travel time benefit, and fixed factors or independent variables are headway, market penetration and traffic volume. If the results show that the alternative hypothesis is true, then a post hoc test is conducted with only the significant variables and pairs of variables considered in the model. The post hoc test used for the study is Tukey's honest significant difference (HSD) test (NIST, 2012). Tukey's test analyzes if there is a statistically significant difference present within the different levels of the parameters. It compares the mean of every level of parameter with the mean of each of the other parameter level. It includes all the pairwise comparisons as well, and identifies the differences between any two comparisons which are greater than the standard error, i.e.

$Q = (\mu_1 - \mu_2) / SE$

Where, μ_1 & μ_2: averages of any two parameter levels
SE: standard error

Since the sample size is equal for this set, Tukey's test is chosen as it gives satisfactory results. IBM SPSS is used for the doing the post hoc Tukey's HSD tests. The tests are done at 95% confidence interval.

3.6 Summary

This chapter dealt with the site selection aspect of the study, along with the data collection and data interpretation. The formation of baseline model in CORSIM was explained for all the three models considered in this research. The parameters that were altered for observing their effects on the traffic flow were explained. Finally the method used for conducting the statistical analysis and the tests that were conducted was described. The next chapters covers the results obtained from the simulations and from the statistical tests conducted.

Chapter IV

Results and Discussion

This chapter compiles the results obtained from the microsimulation models that were performed in CORSIM. There were three models created; a mixed freeway segment with interchanges, an isolated interchange segment, and a basic freeway segment. Chapter IV is divided into three sections, based on the microsimulation models. These three sections describe the results obtained from microsimulation models and the respective graphic representation. Finally the results obtained from statistical tests are presented and interpreted.

4.1 Mixed Freeway Segment (MFS): Simulation Analysis

This segment is a 5.3 mile stretch, consisting of freeway segments and interchanges. This study segment is representative of a typical freeway which receives some traffic from on ramps. The model was first evaluated for baseline case, with no market penetration of DATP equipped vehicles at current traffic volume, and the default distribution of headway adopted in CORSIM, also known as the Pitt-car following model. The models are then altered by each parameter one at a time, while keeping the other factors constant. There are three traffic volumes, four time headways and five market penetrations taken into account in the models. For each combination of traffic volume, time headway, and market penetration, ten simulations are performed with different random number seeds, to replicate the situation on the field. Thus 630 different simulations were executed, and average speeds obtained from each of the 10 runs were recorded in each cell. Tables 4.1, 4.2, and 4.3 show the speeds (in mph) obtained by variation of market penetrations with varying headways for 100%, 115% and 130% of the current traffic volumes respectively. The speeds of the baseline cases are also accounted for in the table. The trends that can be seen in these results are shown in the graphs following the tables.

Table 4.1: Average Speed Results (mph) for Mixed Freeway Section at Current Volume

Baseline: 68.96	Market Penetration				
Headway	20%	40%	60%	80%	100%
1.25s	68.92	69.03	69.10	69.14	69.20
1s	68.95	69.01	69.11	69.19	69.26
0.75s	68.99	69.03	69.13	69.17	69.24
0.5s	68.95	69.03	69.17	69.21	69.26

Table 4.2: Average Speed Results (mph) for Mixed Freeway Section at 115% current volume

Baseline: 68.34	Market Penetration				
Headway	20%	40%	60%	80%	100%
1.25s	68.43	68.40	68.53	68.55	68.72
1s	68.44	68.35	68.53	68.57	68.76
0.75s	68.42	68.52	68.66	68.79	68.88
0.5s	68.55	68.55	68.64	68.73	68.92

Table 4.3: Average Speed Results (mph) for Mixed Freeway Section at 130% current volume

Baseline: 67.16	Market Penetration				
Headway	20%	40%	60%	80%	100%
1.25s	66.89	67.11	67.34	67.37	67.69
1s	67.19	67.23	67.33	67.76	67.96
0.75s	67.19	67.32	67.62	67.78	67.92
0.5s	67.23	67.38	67.54	67.93	68.01

Plots to study the variation of market penetration and speeds at different headways are shown in Figures 4.1, 4.2, and 4.3. Within each market penetration value, as the headways decrease from 1.25 seconds to 0.5 seconds, the speeds tend to increase. This implies that as the vehicles (or heavy trucks, in this case) move at shorter time headway, there is a tendency for the speeds to increase. This trend is seen at all the three different traffic volumes, although it may not be consistently increasing in each combination of market penetration and traffic volume.

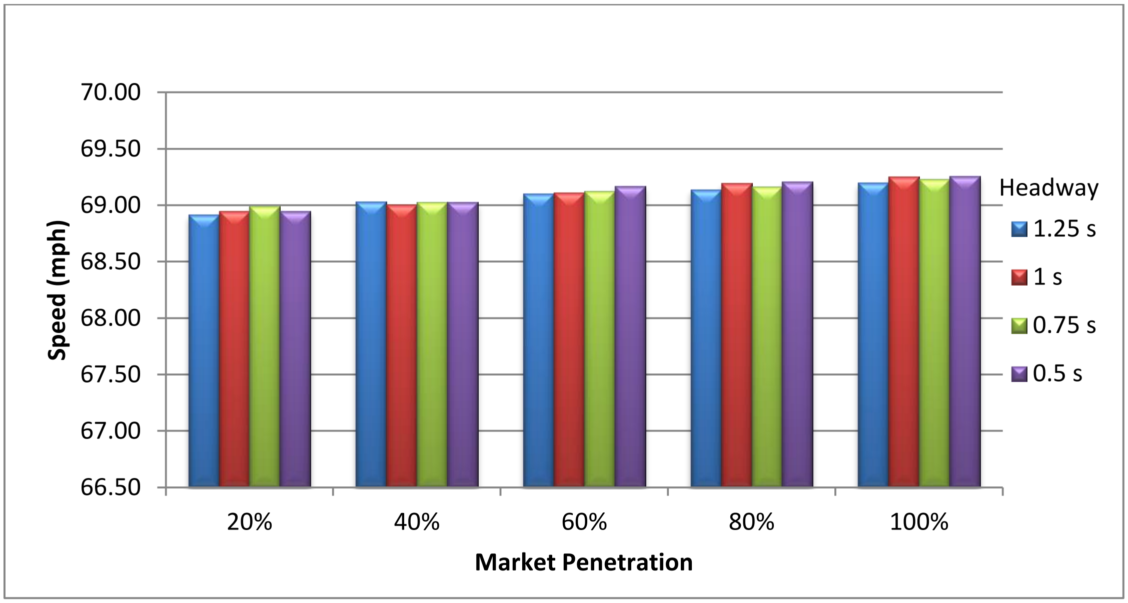

Figure 4.1: Average speeds of vehicles at different market penetrations and different headways at 100% traffic volume for MFS

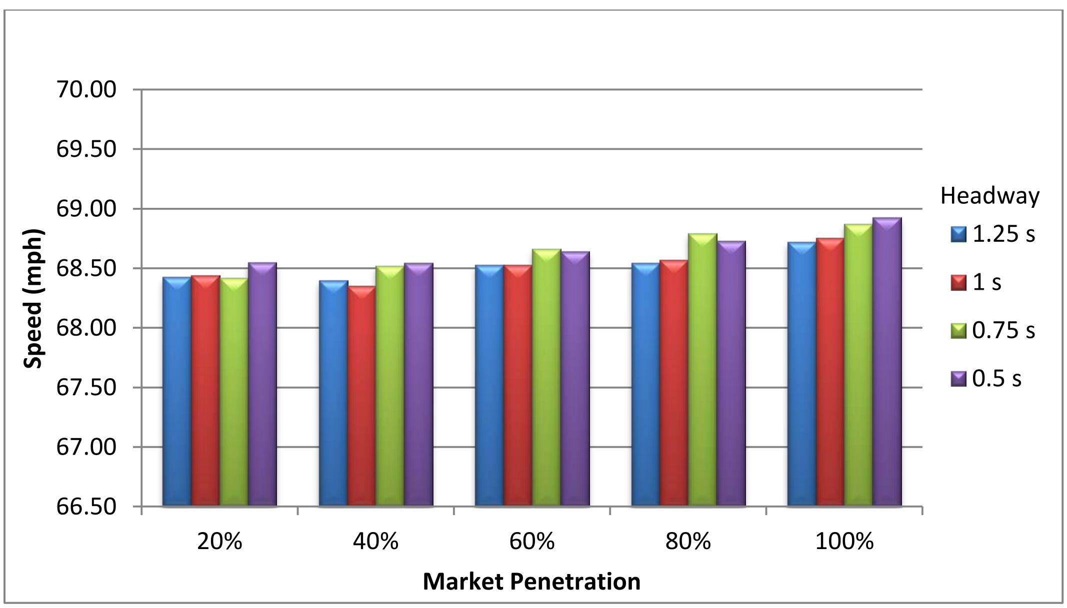

Figure 4.2: Average speeds of vehicles at different market penetrations and different headways at 115% traffic volume for MFS

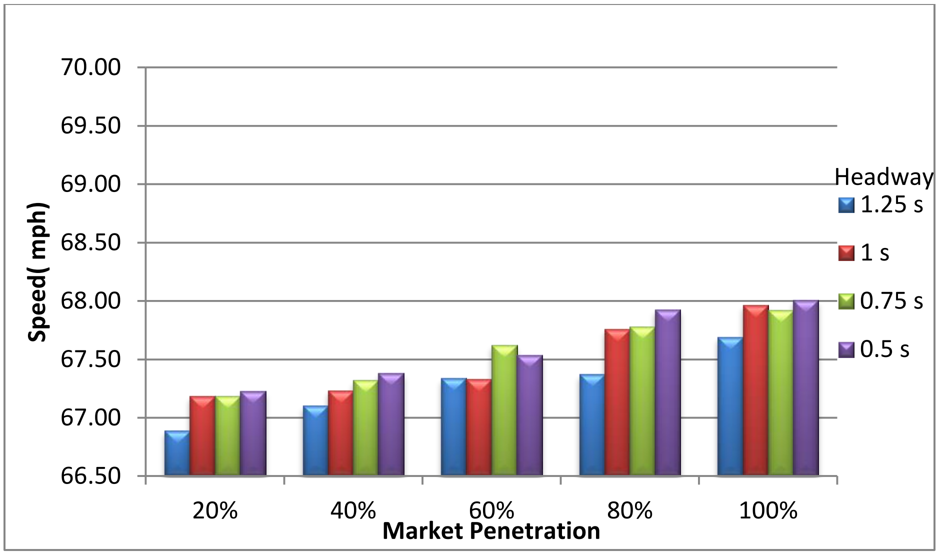

Figure 4.3: Average speeds of vehicles at different market penetrations and different headways at 130% traffic volume for MFS

While looking at a particular headway for different market penetrations, it can be seen that all the different combinations of headways and traffic volumes showed an increase in mean speed. As an example, the graph showing mean speeds at different market penetrations for 0.75 seconds headway at 115% of the current traffic volume, with values obtained from Tables 4.1, 4.2, and 4.3, is shown in Figure 4.4.

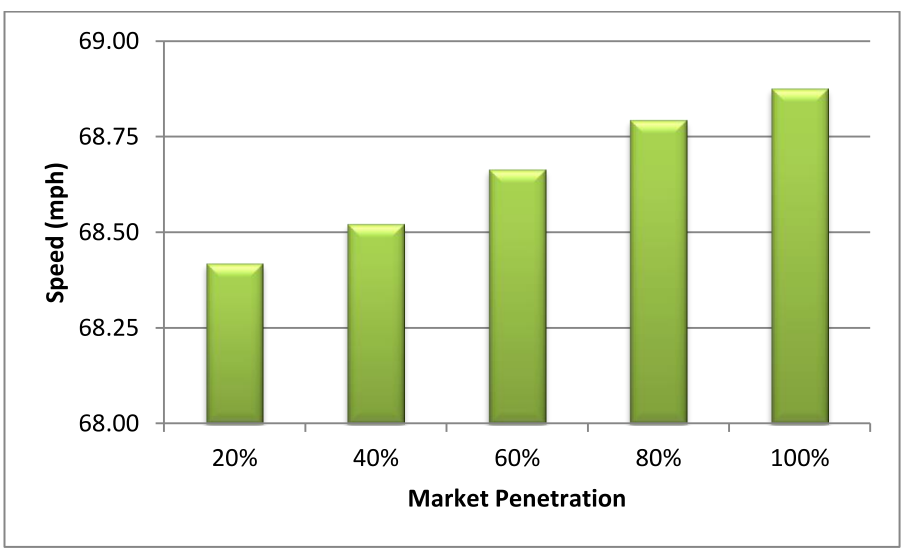

Figure 4.4: Average speeds at different market penetration for a headway of 0.75 seconds at 115% traffic volume for MFS

The effect of varying headway over a particular market penetration and traffic volume (in this example, 20% market penetration and 115% traffic volume) shown in Figure 4.5, it can be seen that there is hardly any variation in the speeds, and all the variation is within a range of 0.25 mph. This is true for all the cases. Thus it can be said that for any particular market penetration and traffic volume, a change in time headway does not cause a substantial change in the speeds of vehicles.

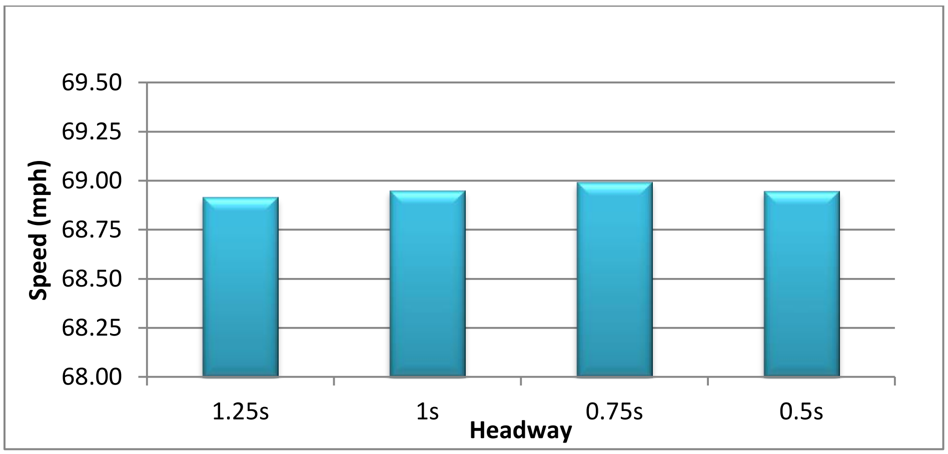

Figure 4.5: Average speeds at different headway distribution for 20% market penetration for 115% traffic volume for MFS

However, an interesting observation that could be seen in Figures 4.1, 4.2, and 4.3 is that, although the trends are increasing for each headway and market penetration, the overall speeds pertaining to increasing traffic volumes are decreasing. As an example, Figure 4.6 shows a plot of average speed for 60% market penetration and 1 second headway; as traffic volumes increase, average speed showed a declining trend. The same is true for all the cases. Thus it can be said that at any given market penetration and headway, the speeds show a declining trend with the increase in traffic volume. This is consistent with the speed flow curves for freeways (HCM, 2010), where the speeds are constant as the flow rate increases up to a certain volume and then shows a declining trend. The increase in flow rate can be implied as increase in volume, thus in accordance with the graph presented below.

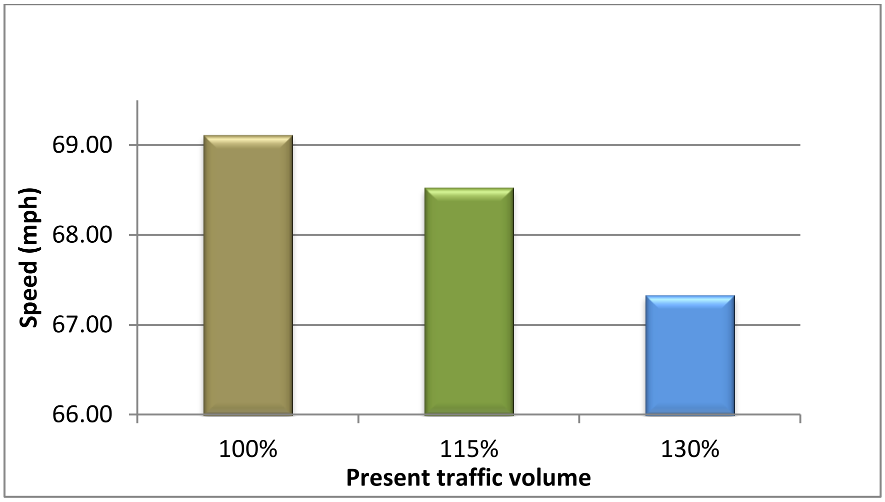

Figure 4.6: Average speeds of vehicles at different traffic volumes for 60% market penetration and 1 second headway for MFS

Travel time benefits were calculated as another way of expressing how the increase in speeds would affect the average amount of time spent by vehicles on the road. The travel time required by each combination of time headway, market penetration and traffic volume is calculated, and compared against the baseline model with no CACC heavy trucks at all. Thus, a relative gain or loss of travel time (in s/veh/mile) is recorded from the baseline case. This can be seen in Tables 4.4, 4.5, and 4.6 for 100%, 115%, and 130% traffic volume respectively. The tables below show a positive sign if there is a travel time gain and a negative sign when there is a travel time loss by the use of CACC parameters as compared to the baseline case.

Table 4.4: Travel time savings in seconds per mile for 100% traffic volume for MFS

Baseline: 0	Market Penetration				
Headway	20%	40%	60%	80%	100%
1.25s	0.11	0.20	0.25	0.28	0.33
1s	0.14	0.18	0.26	0.32	0.37
0.75s	0.17	0.20	0.27	0.30	0.36
0.5s	0.14	0.20	0.30	0.33	0.37

Table 4.5: Travel time savings in seconds per mile for 115% traffic volume for MFS

Baseline: 0	Market Penetration				
Headway	20%	40%	60%	80%	100%
1.25s	0.06	0.04	0.14	0.15	0.28
1s	0.07	0.00	0.14	0.17	0.31
0.75s	0.05	0.13	0.24	0.34	0.41
0.5s	0.15	0.15	0.22	0.29	0.44

Table 4.6: Travel time savings in seconds per mile for 130% traffic volume for MFS

Baseline: 0	Market Penetration				
Headway	20%	40%	60%	80%	100%
1.25s	-0.22	-0.04	0.14	0.17	0.42
1s	0.02	0.06	0.14	0.47	0.63
0.75s	0.02	0.13	0.36	0.49	0.60
0.5s	0.06	0.18	0.30	0.61	0.67

From the tables above and in Figures 4.7, 4.8, and 4.9, it is observed that at any given headway, the travel time savings increase with the increase in market penetration. Similarly, at any given market penetration, with a decrease in headway, it can be seen that there is an increase in the travel time savings; however the increase is not as distinct as the variation in market penetration. It can also be seen that travel time savings increase with the increase of traffic volume on the freeway.

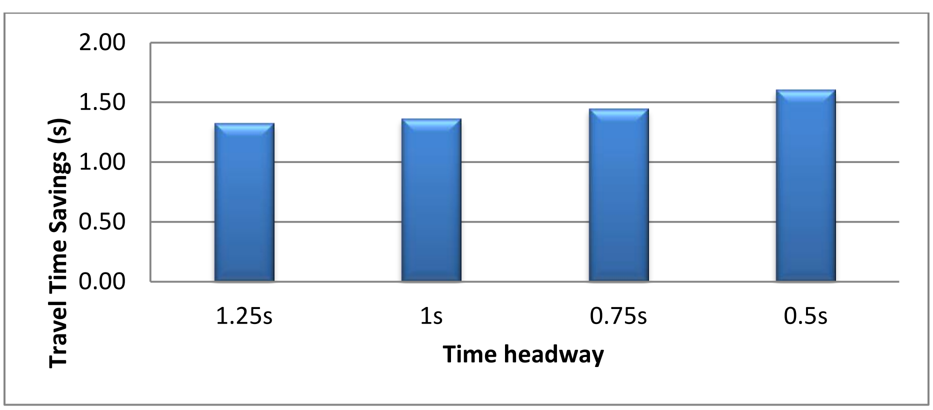

Figure 4.7: Travel time savings for different time headways at 60% market penetration and at present traffic volume for MFS

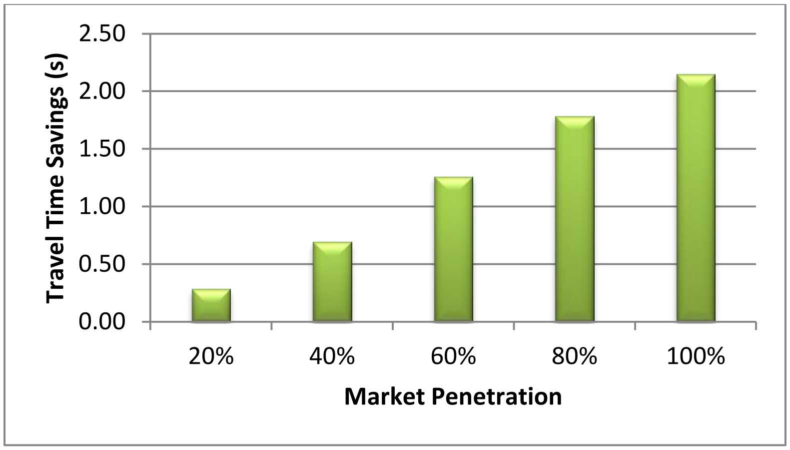

Figure 4.8: Travel time savings for different market penetration at 0.75 s time headway and at 115% traffic volume for MFS

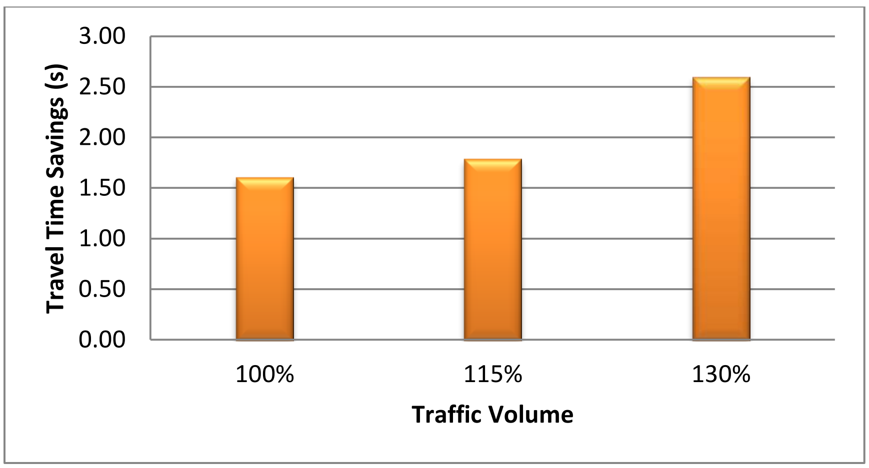

Figure 4.9: Travel time savings for different traffic volumes at 80% market penetration and headway of 0.75s headway for MFS

4.2 Mixed Freeway Segment: Statistical Analysis

After gaining useful information about the trends occurring in the speed and travel time data, the model is then tested for statistical significance. The statistical analysis procedure begins with conducting a t-test to determine if the speed values are statistically significant at $\alpha=0.05$ level. For the variability consideration of the t-tests, the values from each of the 10 runs for every combination of parameters serve as the inputs. Tables 4.7, 4.8, and 4.9 show the p values obtained from the t-tests. Full results are presented in the appendices. Looking at the statistical significance levels shown in the tables, it can be seen that the statistical significance occurs only at higher market penetration values, and is generally independent of decreasing headway values. The significance levels show a similar trend at 130% traffic volume, and have more statistically significant values at higher market penetration levels.

Table 4.7: P-values at current traffic volume for MFS

Headway	Market Penetration				
	20%	40%	60%	80%	100%
1.25 s	0.0740	0.0484*	0.0001**	0.0000**	0.0000**
1.00 s	0.2046	0.2272	0.0008**	0.0000**	0.0000**
0.75 s	0.3698	0.0644	0.0030**	0.0000**	0.0000**
0.5 s	0.1812	0.0710	0.0000**	0.0000**	0.0000**

(* denotes statistical significance at α=0.05 level. ** denotes statistical significance at α=0.01 level)

Table 4.8: P-values at 115% current traffic volume results for MFS

Headway	Market Penetration				
	20%	40%	60%	80%	100%
1.25 s	0.1028	0.2039	0.0028**	0.0011**	0.0000**
1.00 s	0.0564	0.4292	0.0029**	0.0010**	0.0000**
0.75 s	0.0710	0.0022**	0.0000**	0.0000**	0.0000**
0.5 s	0.0009**	0.0019**	0.0000**	0.0000**	0.0000**

(* denotes statistical significance at α=0.05 level. ** denotes statistical significance at α=0.01 level)

Table 4.9: P-values at 130% current traffic volume results for MFS

Headway	Market Penetration				
	20%	40%	60%	80%	100%
1.25 s	0.0931	0.3664	0.1283	0.0520	0.0014**
1.00 s	0.4357	0.3323	0.1245	0.0002**	0.0000**
0.75 s	0.4338	0.1302	0.0013**	0.0002**	0.0000**
0.5 s	0.3358	0.0690	0.0091**	0.0000**	0.0000**

(* denotes statistical significance at α=0.05 level. ** denotes statistical significance at α=0.01 level)

A univariate ANOVA test is then conducted to determine if there are any statistically significant effects of any one parameter on the travel time benefit, and if there are any statistically significant effects on the travel time benefit due to interactions of one parameter with another parameter. The results of these tests are shown in Table 4.10, which shows the univariate ANOVA results, with traffic volume, market penetration and headways as the three parameters, whose levels are tested and are compared in between them. From the table, it can be seen that a change in traffic volume, time headway, and market penetration, will cause a statistically significant effect on the traffic flow, which is calculated by travel time savings in ANOVA. Interaction terms of traffic volume with time headway and traffic volume with market penetration also show statistical significance, implying that a combination of these parameters causes an additional effect on the traffic flow. It can be seen from the table that the pairwise combination of headway with market penetration, and the combination of all three parameters are statistically insignificant.

Table 4.10: Univariate ANOVA test for the parameters of time headway, market penetration, and traffic volume

Source	Type III Sum of Squares	df	Mean Square	F	Sig.
Corrected Model	551.565[a]	59	9.349	15.702	0.000
Intercept	559.137	1	559.137	939.146	0.000
TV	79.103	2	39.551	66.432	0.000
HW	48.722	3	16.241	27.278	0.000
MP	304.404	4	76.101	127.822	0.000
TV * HW	26.881	6	4.480	7.525	0.000
TV * MP	73.732	8	9.217	15.480	0.000
HW * MP	7.799	12	0.650	1.092	0.365
TV * HW * MP	10.925	24	0.455	0.765	0.783
Error	321.499	540	0.595		
Total	1432.201	600			
Corrected Total	873.064	599			

(TV: traffic volume; MP: market penetration; HW: time headway)

After the significance is checked for in the univariate ANOVA method, the analysis of these differences that are observed need to be done. Thus another test is done, known as Tukey's HSD Test. Tukey's test compares the mean of every level of parameter with the mean of each of the other parameter level. In Tukey's test, HSD number is first calculated, using the following formula:

$$\text{HSD} = q\left(\sqrt{\frac{MS\ within}{n}}\right)$$

Where, q= studentized range statistic,

MS within= mean square value, obtained from ANOVA

n= number of values within each parameter level

With HSD value calculated, mean difference of each level of parameter (I-J) is calculated, and compared with HSD. If the difference between two parameter levels exceeds HSD, implies there is a statistically significant difference between the two levels of parameter. Tables 4.11, 4.12 and 4.13 show the first mean difference value (I-J) in each of the three tables at a statistical significance of α=0.05.

Table 4.11: Tukey's HSD Results for traffic volume variations

TV	100%	115%	130%
100%	-	-0.559*	-0.879*
115%	0.559*	-	-0.320*
130%	0.879*	0.320*	-

(* denotes statistical significance at α=0.05 level; TV: traffic volume)

Table 4.12: Tukey's HSD Results for time headway variations

HW	1.25	1	0.75	0.5
1.25	-	-0.342*	-0.621*	-0.740*
1	0.342*	-	-0.279*	-0.398*
0.75	0.621*	.279*	-	-0.119
0.5	0.740*	0.398*	0.119	-

(* denotes statistical significance at α=0.05 level; HW: time headway)

Table 4.13: Tukey's HSD Results for market penetration variations

MP	20%	40%	60%	80%	100%
20%	-	-0.284*	-0.884*	-1.396*	-1.950*
40%	0.284*	-	-0.600*	-1.111*	-1.665*
60%	0.884*	0.600*	-	-0.512*	-1.065*
80%	1.396*	1.111*	0.512*	-	-0.554*
100%	1.950*	1.665*	1.065*	0.554*	-

(* denotes statistical significance at α=0.05 level; MP: market penetration)

It can be observed that differences in travel time benefits at different traffic volumes always have a statistically significant difference at every level of the parameter. The same is true for market penetration as well. The difference in time headway variations are significant at all levels, except at 0.75s and 0.5s.

4.3 Isolated Interchange Section (IIS): Simulation Analysis

The second model which was considered was of an interchange was isolated from the larger segment used in the mixed freeway segment model. This model was developed to see the changes in traffic parameters like speed and delay associated with platoons in an interchange alone. This model has 0.3 mile of freeway segment along with on- and off- ramps. The same parameters were applied and the results were noted, as can be seen in Tables 4.14, 4.15, and 4.16. The baseline cases are also noted for comparison in the tables.

Table 4.14: Average Speed Results (mph) for Isolated Interchange Section at 100% traffic volume for IIS

Baseline: 69.24	Market Penetration				
Headway	20%	40%	60%	80%	100%
1.25s	69.19	69.22	69.17	69.15	69.22
1s	69.26	69.15	69.20	69.14	69.23
0.75s	69.26	69.07	69.22	69.34	69.15
0.5s	69.25	69.21	69.30	69.26	69.26

Table 4.15: Average Speed Results (mph) for Isolated Interchange Section at 115% traffic volume for IIS

Baseline: 68.34	Market Penetration				
Headway	20%	40%	60%	80%	100%
1.25s	68.39	68.48	68.92	68.20	68.62
1s	68.81	68.75	68.78	68.61	68.25
0.75s	68.40	68.50	68.70	68.72	68.80
0.5s	68.31	68.81	68.53	68.82	68.97

Table 4.16: Average Speed Results (mph) for Isolated Interchange Section at 130% traffic volume for IIS

Baseline: 66.94	Market Penetration				
Headway	20%	40%	60%	80%	100%
1.25s	66.19	66.83	67.18	66.27	67.60
1s	67.37	66.75	67.63	67.25	67.34
0.75s	66.40	67.13	66.84	67.72	67.42
0.5s	65.44	66.37	66.34	66.62	66.91

Figures 4.10, 4.11, and 4.12 show the speed variations for time headway and market penetration distributions for 100%, 115%, and 130% of the current traffic volumes, respectively. It can be seen from the graphs that the distribution does not show a particular trend. The speeds for 100% of the current traffic volume hardly show any variations, with all the speeds above 69 mph. It can be seen that 115% traffic volume graph may be showing a few trends, but they are occurring within each time headway. The 130% traffic volume graph shows random variations which do

not exhibit any trend. It is suspected that the very short segment being modeled was not long enough for trends to emerge and overcome any random noise in the models. These relatively low variations in no specific trend may also be a result of the random number seeds chosen.

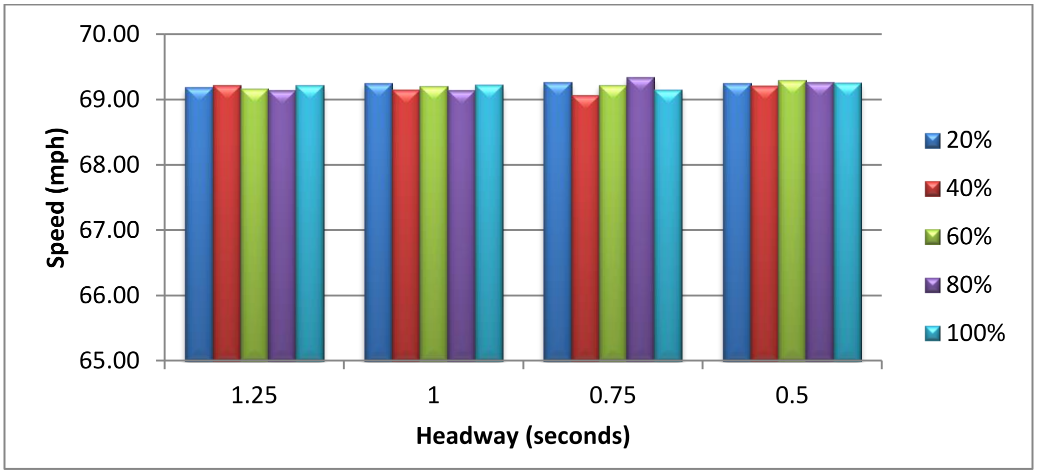

Figure 4.10: Average speeds at different headway distribution and market penetration for 100% traffic volume for IIS

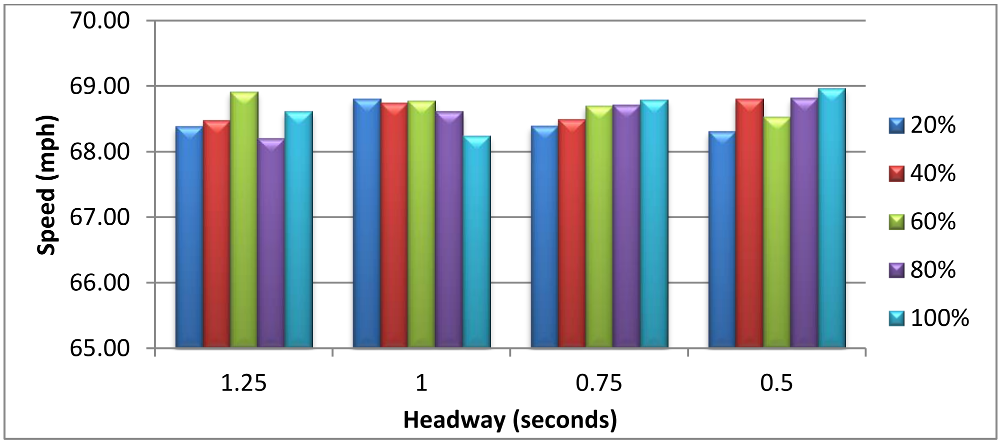

Figure 4.11: Average speeds at different headway distribution and market penetration for 115% traffic volume for IIS

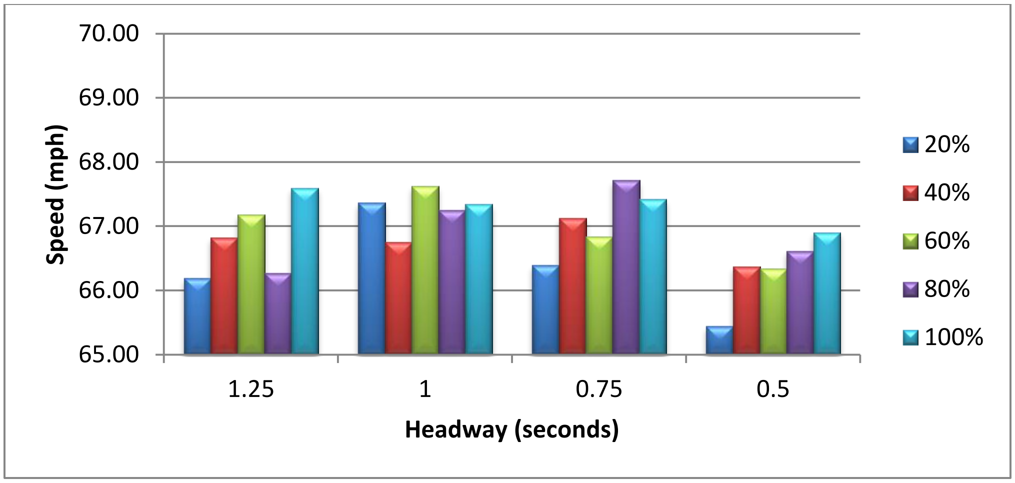

Figure 4.12: Average speeds at different headway distribution and market penetration for 130% traffic volume for IIS

Travel time benefits were calculated in the same manner as the previous model. While observing the Table 4.17, 4.18, and 4.19, it could be seen that there are more cells with negative entries than positive entries. This implies that there are only a few market penetration, time headway, and traffic volume combinations which actually have travel time savings with respect to the baseline case.

Table 4.17: Travel time savings in seconds per mile for 100% traffic volume for IIS

Baseline: 0	Market Penetration				
Headway	20%	40%	60%	80%	100%
1.25s	-0.05	-0.02	-0.07	-0.09	-0.02
1s	0.02	-0.09	-0.04	-0.10	-0.01
0.75s	0.02	-0.16	-0.02	0.10	-0.09
0.5s	0.01	-0.03	0.06	0.02	0.02

Table 4.18: Travel time savings in seconds per mile for 115% traffic volume for IIS

Baseline: 0	Market Penetration				
Headway	20%	40%	60%	80%	100%
1.25s	0.05	0.14	0.57	-0.14	0.27
1s	0.46	0.40	0.43	0.26	-0.09
0.75s	0.06	0.16	0.35	0.37	0.45
0.5s	-0.03	0.46	0.19	0.47	0.61

Table 4.19: Travel time savings in seconds per mile for 130% traffic volume for IIS

Baseline: 0	Market Penetration				
Headway	20%	40%	60%	80%	100%
1.25s	-0.78	-0.11	0.25	-0.69	0.67
1s	0.44	-0.20	0.70	0.32	0.41
0.75s	-0.56	0.19	-0.10	0.79	0.49
0.5s	-1.57	-0.59	-0.62	-0.33	-0.03

There are no visible trends in the travel time savings for either decreasing headway or increasing market penetration, keeping other parameters constant, as can be seen in Figures 4.13 and 4.14. The same is observed for a difference in traffic volumes as well. A representative graph is shown in Figure 4.15.

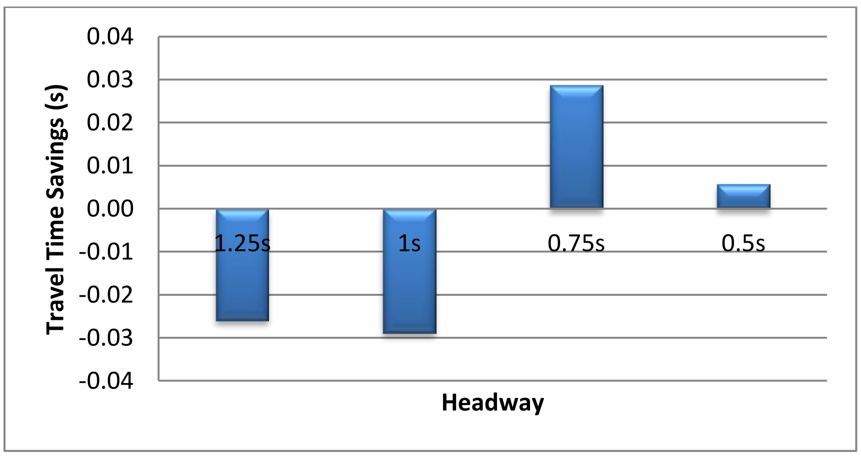

Figure 4.13: Change in travel time with time headways at 80% market penetration and 100% traffic volume for IIS

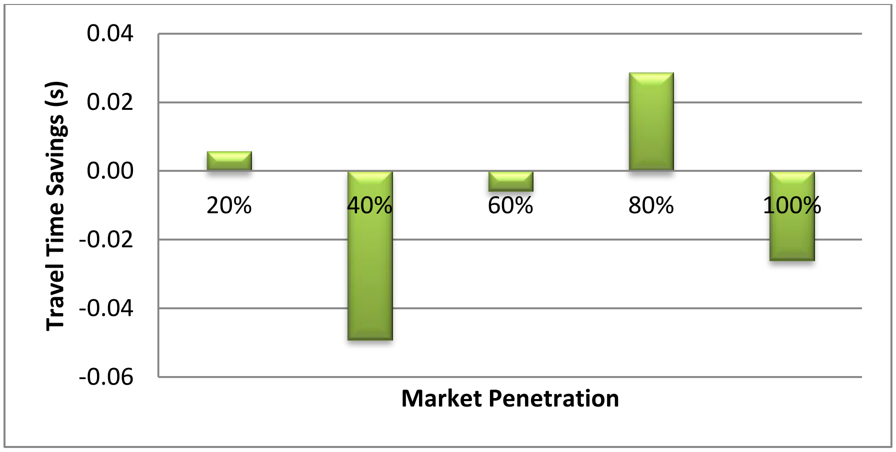

Figure 4.14: Change in travel time with market penetration at 0.75 s headway and 100% traffic volume for IIS

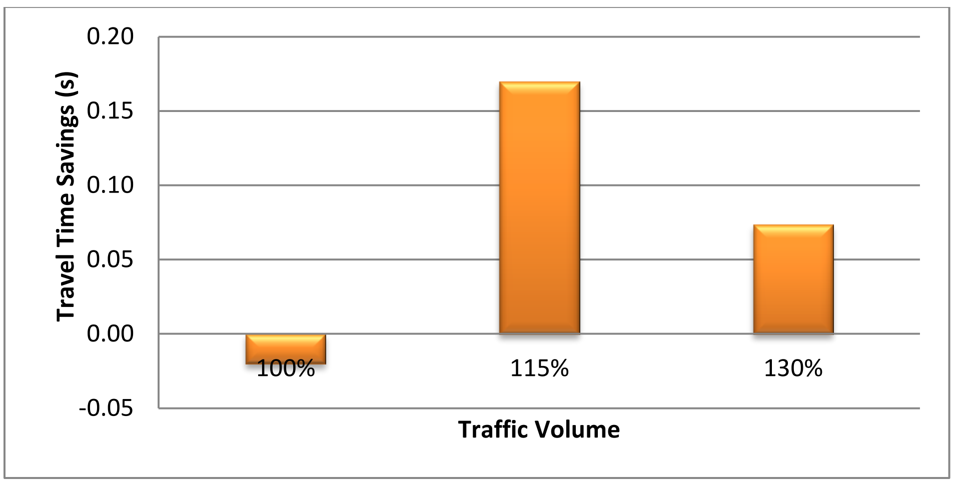

Figure 4.15: Change in travel time with traffic volume at 60% market penetration and 1.25 s headway for IIS

4.4 Isolated Interchange Segment: Statistical Analysis

Statistical analysis for the isolated interchange segment was carried out in the same manner as that of the mixed freeway. For the statistical analysis, a t-test is conducted to check for statistically significance of speed values at $\alpha=0.05$ level. For the variability consideration of the t-tests, the values from each of the 3 runs for every combination of parameters serve as the inputs. Tables 4.20, 4.21, and 4.22 show the results obtained from the t-tests. Complete results are presented in the appendices section of the report. It can be seen from the significance levels that at present traffic conditions, no parameter combination shows a statistical significance at α=0.05. A similar case is found at 115% and 130% traffic volume levels. Overall, the isolated interchange segment has only a single combination of traffic volume, time headway and market penetration which shows a statistically significant difference from its respective baseline case.

Table 4.20: P-values at current traffic volume for IIS

Headway	Market Penetration				
	20%	40%	60%	80%	100%
1.25 s	0.2904	0.4248	0.2047	0.1231	0.3795
1 s	0.4606	0.2353	0.3057	0.1999	0.4422
0.75 s	0.4077	0.1590	0.3940	0.1235	0.1405
0.5 s	0.4671	0.3570	0.2752	0.4003	0.4271

(* denotes statistical significance at α=0.05 level. ** denotes statistical significance at α=0.01 level)

Table 4.21: P-values at 115% current traffic volume for IIS

Headway	Market Penetration				
	20%	40%	60%	80%	100%
1.25 s	0.4468	0.3546	0.0857	0.4298	0.2186
1 s	0.1522	0.1698	0.1710	0.2322	0.4056
0.75 s	0.4488	0.4323	0.1954	0.1667	0.1239
0.5 s	0.4662	0.1244	0.3222	0.1130	0.0716

(* denotes statistical significance at α=0.05 level. ** denotes statistical significance at α=0.01 level)

Table 4.22: P-values at 130% current traffic volume for IIS

Headway	Market Penetration				
	20%	40%	60%	80%	100%
1.25 s	0.1547	0.4247	0.3692	0.0557	0.2492
1 s	0.2252	0.3910	0.0568	0.1843	0.2552
0.75 s	0.0867	0.3919	0.4523	0.0392*	0.1039
0.5 s	0.0785	0.2205	0.0876	0.2890	0.4787

(* denotes statistical significance at α=0.05 level. ** denotes statistical significance at α=0.01 level)

To look at the significant effects of each parameter over the others and the effects of pairwise combinations, univariate ANOVA test is performed. The results of the univariate ANOVA tests are shown in Table 4.23, with traffic volume, market penetration and headways as the three parameters, whose levels are tested and are compared in between them. Traffic volume and market penetration have statistically significant effects on the traffic volume. However, it can be seen from the table that time headway is not a statistically significant parameter. Also, there is an additional significant effect on the traffic volume by the interaction of time headway and traffic volume.

Table 4.23: Univariate ANOVA test for the parameters of time headway, market penetration, and traffic volume

Source	Type III Sum of Squares	df	Mean Square	F	Sig.
Corrected Model	2.558[a]	59	0.043	1.534	0.025
Intercept	.040	1	0.040	1.428	0.234
TV	.355	2	0.178	6.287	0.003
HW	.198	3	0.066	2.331	0.078
MP	.283	4	0.071	2.500	0.046
HW * MP	.453	12	0.038	1.335	0.208
TV * HW	.551	6	0.092	3.249	0.005
TV * MP	.344	8	0.043	1.522	0.157
TV * HW * MP	.374	24	0.016	0.552	0.954
Error	3.392	120	0.028		
Total	5.990	180			
Corrected Total	5.949	179			

(TV: traffic volume; MP: market penetration; HW: time headway)

Tukey's HSD test is then conducted to analyze the statistically significant differences found in univariate ANOVA. The statistically insignificant parameters, such as time headway, interaction terms of time headway and market penetration etc., are removed from the model and then

Tukey's HSD test is implemented. The values are calculated in the same manner as explained in statistical analysis section of mixed freeway segment. Tables 4.24 and 4.25 show the first mean difference value (I-J) in each of the three tables at a statistical significance of α=0.05.

Table 4.24: Tukey's HSD Results for traffic volume variations

TV	100%	115%	130%
100%		-0.086*	0.015
115%	0.086*		0.101*
130%	-0.015	-0.101*	

(* denotes statistical significance at α=0.05 level; TV: traffic volume)

Table 4.25: Tukey's HSD Results for market penetration variations

MP	20%	40%	60%	80%	100%
20%	-	-0.047	-0.091	-.073320	-0.116*
40%	0.047	-	-0.044	-0.027	-0.069
60%	0.091	0.044	-	0.018	-.0246
80%	0.073	0.027	-0.018	-	-0.0422
100%	0.116*	0.069	0.025	0.0422	-

(* denotes statistical significance at α=0.05 level; MP: market penetration)

It can be observed that traffic volumes do not have a statistically significant difference between 100% and 115% level. For market penetrations, a statistically significant difference between the means has been found only in 20% and 100% levels.

4.5 Basic Freeway Segment (BFS): Simulation Analysis

The third model considered in this study was that of an extended basic freeway segment in a rural area. This model was simulated to observe the changes in traffic parameters like speed and delay associated with platoons in a freeway segment devoid of any interchanges. The parameters that were altered to study the measure of effectiveness of the models were the same as the previous two models, and the values are noted in Tables 4.26, 4.27, and 4.28 for 100%, 115%, and 130% of the traffic volumes, respectively. The baseline case results are noted in the top right corner of every table.

Table 4.26: Average Speed Results for Basic Freeway Segment at 100% for BFS

Baseline: 66.49	Market Penetration				
Headway (s)	20%	40%	60%	80%	100%
1.25	66.57	66.35	66.51	66.32	66.37
1	66.56	66.38	66.55	66.48	66.35
0.75	66.60	66.50	66.60	66.56	66.43
0.5	66.59	66.49	66.50	66.64	66.64

Table 4.27: Average Speed Results for Basic Freeway Segment at 115% for BFS

Baseline: 65.95	Market Penetration				
Headway (s)	20%	40%	60%	80%	100%
1.25	66.08	66.07	65.92	65.91	65.77
1	65.88	66.01	66.03	65.96	65.95
0.75	66.06	65.99	66.13	66.11	66.11
0.5	66.12	66.10	66.22	66.23	66.25

Table 4.28: Average Speed Results for Basic Freeway Segment at 130% for BFS

Baseline: 65.66	Market Penetration				
Headway (s)	20%	40%	60%	80%	100%
1.25	65.49	65.52	65.45	65.41	65.36
1	65.65	65.51	65.48	65.40	65.39
0.75	65.46	65.57	65.59	65.62	65.45
0.5	65.64	65.62	65.79	65.62	65.74

Figures 4.16, 4.17, and 4.18 show the speed variations for time headway and market penetration distributions for 100%, 115%, and 130% of the current traffic volumes, respectively. It can be seen from the graphs that the distribution does not show a particular trend. The speeds for 100%, 115% and 130% of the current traffic volume hardly show any variations for all types of headways, with speeds close to 66.5 mph, 66 mph, and 65.5 mph respectively.

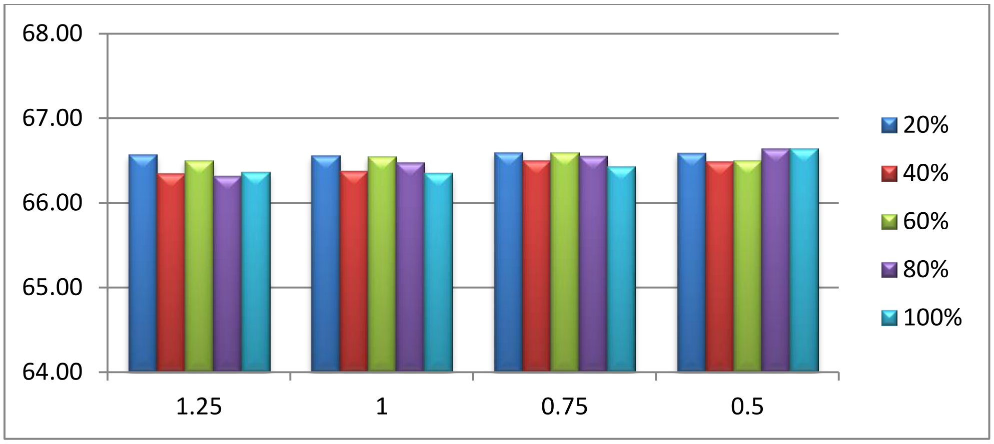

Figure 4.16: Average speeds at different headway distribution and market penetration for 100% traffic volume for BFS

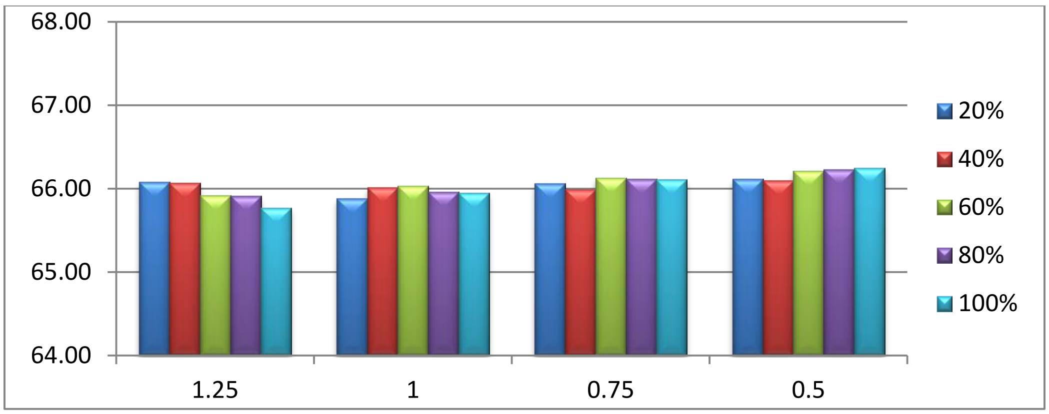

Figure 4.17: Average speeds at different headway distribution and market penetration for 115% traffic volume for BFS

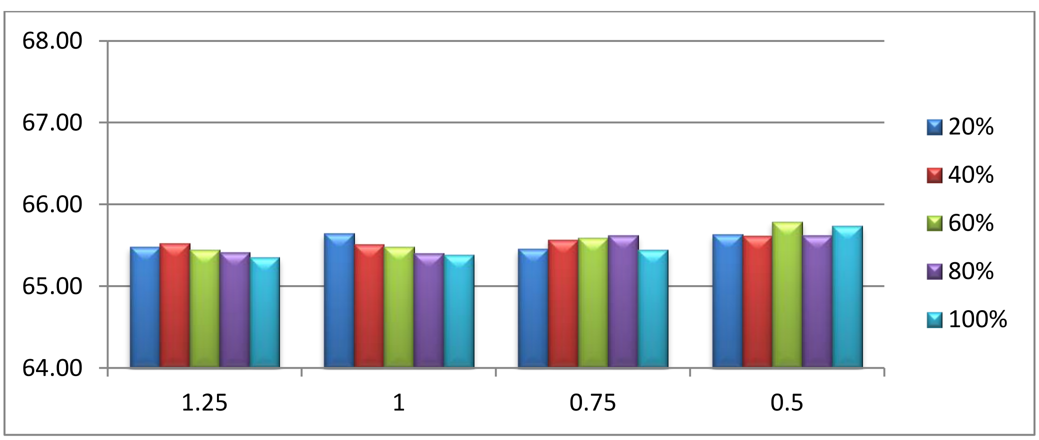

Figure 4.18: Average speeds at different headway distribution and market penetration for 130% traffic volume for BFS

Travel time savings are calculated in the same manner as before. The delays resulting from each segment for each combination is calculated and compared against the base model to find the travel time savings, as shown in Tables 4.29, 4.30, and 4.31. It can be seen that maximum travel time savings are obtained in the 115% traffic volume, and the travel time is almost always more than the base line case in 130% traffic volume, implying losses in travel time.

Table 4.29: Travel time savings in seconds per mile for 100% traffic volume for BFS

Baseline: 0	Market Penetration				
Headway	20%	40%	60%	80%	100%
1.25s	0.07	-0.11	0.02	-0.14	-0.10
1s	0.06	-0.09	0.05	-0.01	-0.11
0.75s	0.09	0.01	0.09	0.06	-0.05
0.5s	0.08	0.00	0.01	0.12	0.12

Table 4.30: Travel time savings in seconds per mile for 115% traffic volume for BFS

Baseline: 0	Market Penetration				
Headway	20%	40%	60%	80%	100%
1.25s	0.11	0.10	-0.02	-0.03	-0.15
1s	-0.06	0.05	0.07	0.01	0.00
0.75s	0.09	0.03	0.15	0.13	0.13
0.5s	0.14	0.12	0.22	0.23	0.25

Table 4.31: Travel time savings in seconds per mile for 130% traffic volume for BFS

Baseline: 0	Market Penetration				
Headway	20%	40%	60%	80%	100%
1.25s	-0.14	-0.12	-0.18	-0.21	-0.25
1s	-0.01	-0.13	-0.15	-0.22	-0.23
0.75s	-0.17	-0.08	-0.06	-0.03	-0.18
0.5s	-0.02	-0.03	0.11	-0.03	0.07

There are no trends seen at either variable market penetrations, traffic volumes or time headways, which can be seen in Figures 4.19, 4.20, and 4.21. This may be attributed to the monotonous nature of this freeway segment, with no interruptions or variations in the traffic flow, except the random number seeds. The randomness in the values may also be credited to the differences in random number seeds used for the analysis without any other substantial sources of variability to impact the traffic stream.

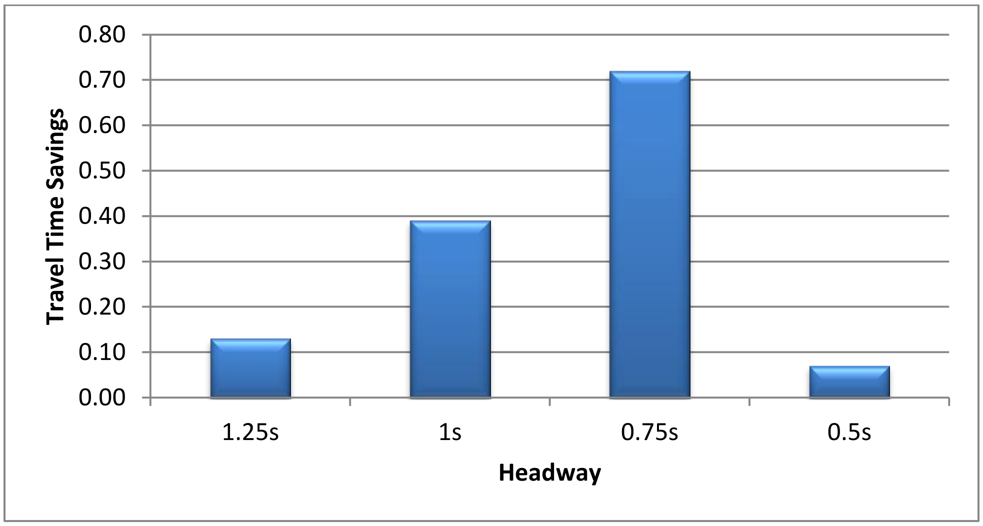

Figure 4.19: Time headway alterations for 60% market penetration at 100% volume for BFS

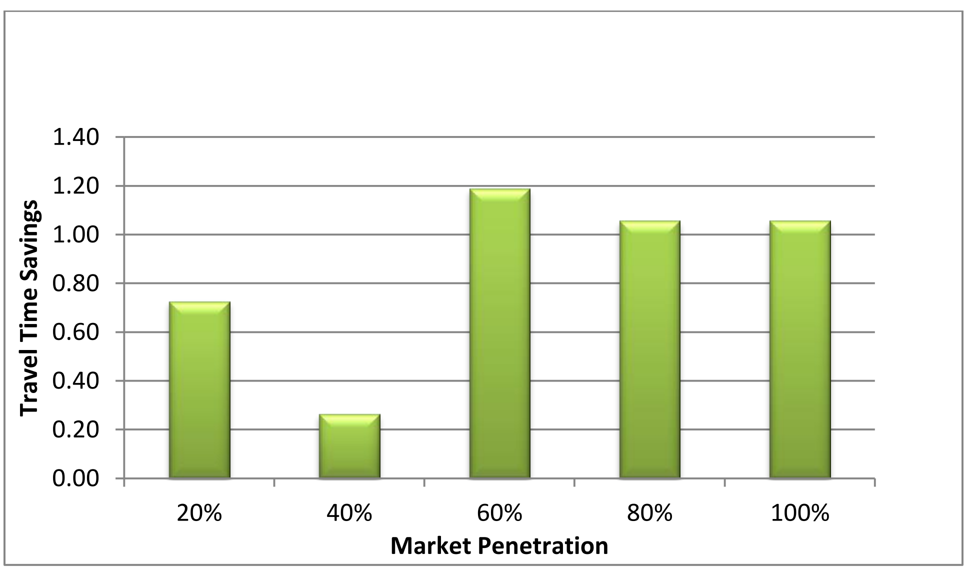

Figure 4.20: Market penetration alteration for 0.75 s headway and 115% traffic volume for BFS

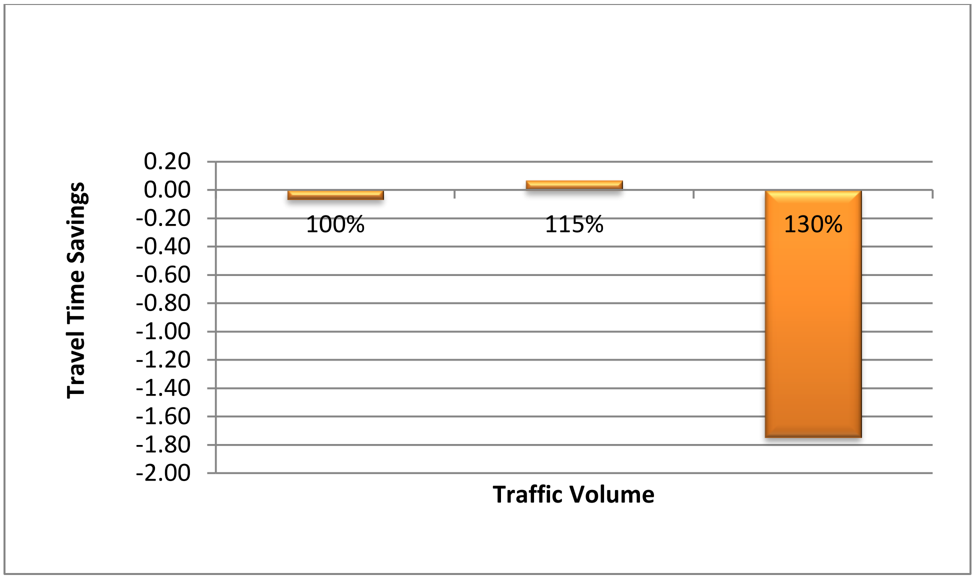

Figure 4.21: Alterations to traffic volume at 80% market penetration and 1s headway for BFS

4.6 Basic Freeway Segment: Statistical Analysis

Statistical analysis for the basic freeway segment was carried out in the same manner as that of the previous two segments. The statistical analysis procedure begins with conducting a t-test for figuring out if the speed values are statistically significant at α=0.05 level. For the variability consideration of the t-tests, the values from each of the 3 runs for every combination of parameters serve as the inputs. Tables 4.32, 4.33, and 4.34 show the results obtained from the t-tests. Full results are presented in the appendices. As seen from the significance levels, it can be seen that at present traffic conditions, most of the headway and market penetration combinations are statistically insignificant at any α level. Overall, the basic freeway segment has only a few combinations of traffic volume, time headway and market penetration which show a statistically significant difference from their respective baseline cases, and all of them occur at high market penetration levels.

Table 4.32: P-values at current traffic volume and t-test results for BFS

Headway	Market Penetration				
	20%	40%	60%	80%	100%
1.25 s	0.2811	0.1132	0.4549	0.0874	0.1342
1.00 s	0.2589	0.2284	0.3396	0.4421	0.2393
0.75 s	0.1886	0.4722	0.2092	0.3211	0.3337
0.50 s	0.2058	0.4847	0.4718	0.1246	0.1079

(* denotes statistical significance at α=0.05 level. ** denotes statistical significance at α=0.01 level)

Table 4.33: P-values at 115% current traffic volume and t-test results for BFS

Headway	Market Penetration				
	20%	40%	60%	80%	100%
1.25 s	0.1157	0.1731	0.3911	0.3607	0.0722
1.00 s	0.3242	0.2959	0.2246	0.4446	0.4880
0.75 s	0.2449	0.3828	0.0769	0.1058	0.0861
0.5 s	0.0833	0.0988	0.0558	0.0353*	0.0611

(* denotes statistical significance at α=0.05 level. ** denotes statistical significance at α=0.01 level)

Table 4.34: P-values at 130% current traffic volume and t-test results for BFS

Headway	Market Penetration				
	20%	40%	60%	80%	100%
1.25 s	0.0901	0.0973	0.1129	0.1233	0.0158*
1.00 s	0.4502	0.0948	0.0565	0.0314*	0.0192*
0.75 s	0.0593	0.1686	0.3266	0.3522	0.0678
0.5 s	0.3917	0.3780	0.0898	0.3755	0.1728

(* denotes statistical significance at α=0.05 level. ** denotes statistical significance at α=0.01 level)

Univariate ANOVA test is then conducted to observe the effects of each parameter on the other, and if there are any significant effects of any of the pairwise combinations on others. The results of the univariate ANOVA tests are shown in Table 4.35, with traffic volume, market penetration and headways as the three parameters, whose levels are tested and are compared in between them. It can be seen from the table that time headway, traffic volume, market penetration, and the interaction term of time headway and market penetration have significant effects on the traffic flow.

Table 4.35: Univariate ANOVA test for the parameters of time headway, market penetration, and traffic volume

Source	Type III Sum of Squares	df	Mean Square	F	Sig.
Corrected Model	161.578[a]	59	2.739	3.728	0.000
Intercept	.280	1	.280	0.381	0.538
TV	63.080	2	31.540	42.932	0.000
HW	49.485	3	16.495	22.453	0.000
MP	7.189	4	1.797	2.446	0.050
TV * HW	3.348	6	0.558	0.759	0.603
TV * MP	4.860	8	0.607	0.827	0.581
HW * MP	17.335	12	1.445	1.966	0.033
TV * HW * MP	16.283	24	0.678	0.923	0.571
Error	88.157	120	0.735		
Total	250.016	180			
Corrected Total	249.736	179			

(TV: traffic volume; MP: market penetration; HW: time headway)

After the significance is checked for in the univariate ANOVA method, the analysis of these differences that are observed need to be done. Thus, Tukey's HSD Test is conducted. Tukey's test compares the mean of every level of parameter with the mean of each of the other parameter level. It includes all the pairwise comparisons as well, and identifies the differences between any two comparisons which are greater than the standard error. The values are calculated in the same manner as explained in the statistical analysis section for the mixed freeway segment model. Tables 4.36, 4.37, and 4.38 show the first mean difference (I-J) value in each of the three tables at a statistical significance of $\alpha=0.05$.

Table 4.36: Tukey's HSD Results for traffic volume variations

TV	100%	115%	130%
100%	-	-0.610*	0.834*
115%	0.610*	-	1.444*
130%	-0.834*	-1.444*	-

(* denotes statistical significance at α=0.05 level; TV: traffic volume)

Table 4.37: Tukey's HSD Results for time headway variations

HW	1.25	1	0.75	0.5
1.25	-	-0.219	-0.744*	-1.358*
1	0.219	-	-0.524*	-1.138*
0.75	0.744*	0.524*	-	-0.614*
0.5	1.358*	1.138*	0.614*	-

(* denotes statistical significance at α=0.05 level; HW: time headway)

Table 4.38: Tukey's HSD Results for market penetration variations

MP	20%	40%	60%	80%	100%
20%	-	0.319	-0.040	0.234	0.493
40%	-0.319	-	-0.360	-0.084	0.174
60%	0.040	0.360	-	0.275	0.534
80%	-0.234	0.084	-0.275	-	0.259
100%	-0.493	-0.174	-0.534	-0.259	-

(* denotes statistical significance at α=0.05 level; MP: market penetration)

It can be observed that traffic volumes always have a statistically significant difference at every level of the parameter. The same is true for time headway as well, except between 1 second and 1.25 seconds. The differences in market penetration variation are statistically insignificant at all levels from one another.

4.7 Summary

This chapter provided the results obtained from the simulation of the mixed freeway segment, isolated interchange segment, and basic freeway segment. It also showed the effects of the DATP technology on different segments of freeways at a statistically significant level by t-test and univariate ANOVA method, and analyzed the differences between each parameter from Tukey's HSD test. Conclusions and recommendations from this chapter are summarized in the next chapter.

Chapter V

Conclusions and Recommendations

5.1 Conclusions

The application of Cooperative Adaptive Cruise Control (CACC) leads to improved traffic flow, along with drivers being more comfortable to drive at shorter gaps. The task of accelerating and braking in the vehicle is enabled by sensors and performed automatically, which makes the reaction time much lesser than that of the driver. CACC combines vehicle-to-vehicle communication with automated vehicles, impacting highways. Through this research, possible conclusions can be achieved about optimum time headways and market penetrations, along with looking at how much increase in traffic volume the road can support with technology, before the queueing occurs.

This research focused on evaluating traffic impacts of CACC technology on heavy trucks. The main objective of this research was to develop representative sections of a freeway in traffic microsimulation. These models are then altered to best capture the effects of driver assistive truck platooning (DATP) in the traffic stream. The results obtained from the simulations are tested for their statistical significance, and optimal levels of DATP technology on roadways are concluded.

5.2 Model Results

This research mainly focused on the use of CACC technology in heavy trucks, called DATP. In the CORSIM simulation model, CACC enabled heavy trucks consisted of single unit trucks, semi-trailer trucks, and double bottom trailer trucks. This technology was employed in three different freeway scenarios. The average speeds and travel time benefits were obtained from its implementation. The parameters which were altered to check for the sensitivity of the model were market penetration, time headway, and traffic volume. These output parameters varied across each freeway segment.

The first section in consideration was a 5.3 mile freeway section with interchanges and basic freeway segments in a small urban area. It was observed that as the time headway decreased, the speeds on the segment increased. This increase in speed leads to faster movement of vehicles, which ultimately leads to increase in the capacity. With heavy trucks moving closer due to the fixed headway constraint, there is also a possibility of increase in the capacity of the roadway. The results also show that with an increase of this technology into the system, the speeds show a small increase, implying that market penetration also serves as an important factor in the increase in capacity of the roadways. However, as the volume of vehicles increased on the roadway, the speeds declined. A statistical analysis presented that there were significant differences in the mean speeds of modified parameters as compared to the baseline model, and also that the significance became more predominant with the increase in market penetration and time headway. There was a statistically significant relationship found between the interaction terms of traffic volume and market penetration, and traffic volume and time headway.

The second section in consideration was an isolated interchange segment, 0.3 mile long. At the present traffic flow conditions, there are hardly any variations seen in the trends, for either increase in market penetration or decrease in time headway. As the traffic volumes increase, the variations turned out to be more chaotic, with the speeds showing variations up to 3 mph. These chaotic results may be due to the very short length of the segment considered. The platoon effects may not be evident over such a small segment. Statistically, the significance of the altered results was also similar. There are very few combinations of time headway and market penetration which show a statistical significant difference in mean speed and travel time when compared to the baseline case, although these results also do not follow any particular order. There was a statistically significant relationship found between the interaction terms of traffic volume and time headway.

The third section of this study was a basic freeway segment which is 8 miles in length, devoid of any interchanges or interruptions. From the graphs, it can be noted that there are hardly any differences in the speeds for any altered set of parameters. The differences are less than 0.25 mph at maximum, and the travel time benefits are seen only in the present traffic conditions, and not much as the traffic volume increases. The statistical analysis yielded conclusions similar to graphical results, with hardly any alteration being significantly different at a statistical level from their baseline case. Though there is no statistically significant difference seen between the parameters of market penetrations, there is a statistically significant relationship between the interaction term of time headway and market penetration.

Looking at the trends and statistical significance of all the parameters in the given traffic conditions for the mixed freeway segment, it can be concluded that the traffic stream would be the most efficient at the maximum market penetration (100%), minimum time headway (0.5 seconds) and at current traffic volumes. Although the results obtained from these simulations are statistically significant, their practical significance would be best judged by the users of this technology, such as trucking companies, as well as system operators (transportation agencies). On the other hand, the basic freeway segment and the isolated interchange segment do not show any clear trends or statistical significance with the change in the levels of any of the parameters. Thus, an optimum value cannot be concluded.

By observing the speed variations in the results and understanding the statistical significance of each of the parameters, another important consideration is a qualitative perspective of practical significance. The calculation of travel time savings is an important consideration in assessing the results from a practical perspective. Though the savings in travel times are not very high, these values are for very short segments. Freight drivers cover an average of 500 miles a day (American Trucking Association, 2015), and the time savings may be useful to them.

5.3 Recommendations

Using microsimulation as a tool to study the effects of a new technology is a very feasible option while studying the technology in the initial stage. However, trying to accommodate every real life condition is not possible. As the complexity of the road network increases, it is required to test the technology in live traffic as well. These results may vary from the simulation results because the mathematical models for driver behavior may not entirely account for the behavior of drivers in live traffic. Incidents or temporary traffic controls may also affect the traffic flow.

The use of DATP in the mixed traffic is also essential. There are runs being conducted in controlled environments, but the signal frequency, weather conditions, roadway geometry etc. may be subject to change in the performance of the platooning technology. The current study is a simulation of Interstate 85 in Alabama. This makes the results very site specific, and field study is important to decide if generalization of these results is possible to all the freeways.

In addition to the technology implementations, it is also important for the road owning agencies, such as state and federal departments of transportation (DOTs) contribute their effort towards the adaptation of the new technology into the live traffic. Currently, since the technology is still in the testing stage, accommodations must be provided for the pilot projects to be tested in the real traffic conditions. Several of these projects have already been initiated in states like Florida, California, and Nevada etc. At a federal level, NHTSA is working towards providing policy guidance and recommendations to the states which have allowed the safe testing of automated vehicles. There is a need for the government to allow pilot projects in more number of states to understand the difference in sites present.

With DATP technology being implemented on the heavy trucks, substantial speed, fuel savings, travel time savings etc. can be achieved. Therefore there is a need for the government to implement the technology sooner, such that the benefits can be observed to the road users, road owning agencies, and trucking industries.

5.4 Future Work

There are potential aspects of this technology in the traffic stream, which have not been explored yet. This study focused on 2-truck platoons only, due to the software limitations. Adding another truck to the platoon may cause substantial improvements in terms of traffic conditions and fuel savings, but may also increase opportunities for passenger vehicles to disrupt or break the platoons. Therefore, platoons of two or more heavy trucks and their effects on traffic stream need to be studied.

DATP technology is being observed on test tracks in the current stage, and would soon be a part of traffic flow. While the study focused on how this technology would behave in the mixed traffic conditions only, there may be a possibility of roadways being modified to have truck-only lanes, or platoon vehicles only lanes. These would allow the platooned heavy trucks to move together without any interruptions by the passenger vehicles cutting through the platoons. Thus, the effects of truck-only lanes on the platooned truck traffic need to be considered.

With the implementation of DATP technology and its steady proliferation into the market, the truck-only lanes can be gradually assigned as DATP- only lanes. The effect of this dedicated DATP lane on traffic volume and flow needs to be examined.

This study tested the DATP technology on freeways only. It was beyond the scope of this study to examine the effects of DATP implementation on arterials that have at- grade intersections with traffic signals. With signals being encountered on the roadway, the platoons may be subjected to breaking in an urban arterial with a higher signal density, or if the signal timing is not adequate for the entire platoon to cross the intersection. This interaction between arterials and the DATP technology also need to be investigated.

References

American Trucking Association (2015). Trucking Moves America Forward: Professional Truck Driver. *http://www.trucking.org/ata%20docs/what%20we%20do/image%20and%20outreach%20programs/misc%20documents/pro%20Truck%20Drivers_final.pdf*

Amoozadeh, M., Deng, H., Chuah, C. N., Zhang, H. M. & Ghosal D. (2015). Platoon Management with Cooperative Adaptive Cruise Control Enabled by VANET. *Vehicular Communications Journal. Vol. 2(2), April 2015, pp.110–123.*

Arnaout, G. M., & Bowling, S. (2014). A progressive deployment strategy for cooperative adaptive cruise control to improve traffic dynamics. *International Journal of Automation and Computing*. *Vol. 11(1), pp. 10–18.*

Bareket, Z., Fancher, P., Huei Peng, Lee, K., & Assaf, C (2003). Methodology for assessing adaptive cruise control behavior." *IEEE Transactions on Intelligent Transportation Systems. Vol*. *4(3), pp. 123-131*.

Deng, Q. & Ma, X. (2015). A Simulation Platform for Autonomous Heavy-duty Vehicle Platooning in Mixed Traffic. *Paper presented at the 94th Transportation Research Board Annual Meeting.*

Elefteriadou, L., Martin, B., Simmerman, T. & Hale, D. (2009). Using Microsimulation to Evaluate the Effects of Advanced Vehicle Technologies on Congestion. *Center of Mitigation Studies Report. CMS Project Number: 2009-006.*

Elefteriadou, L. An Introduction to Traffic Flow Theory, Springer Optimization and Its Applications, Vol. 84 (Springer, 2014).

Gouy, M., Wiedemann, K., Stevens, A., Brunett, G. & Reed, N. (2014). Driving next to automated vehicle platoons: How do Short Time Headways Influence Non-Platoon Drivers' Longitudinal Control? *Transportation Research Part F, 27, 264-273.*

Gu, H., Jin, P, J., Wan, X., Shah, J., & Ran, B. (2015). A Leading Vehicle Model for Comfortable Acceleration among Cooperative Adaptive Cruise Control (CACC) Vehicle Platoons. *Presented at the 94th Annual Meeting of Transportation Research Board.*

Hallenbeck, M. E., Selezneva, O. I., & Quinly, R. (2014). Verification, Refinement, and Applicability of Long Term Pavement Performance Vehicle Classification Rules. Federal Highway Administration. Report No. FHWA-HRT-13-091.

Johnson, T. (2013). Enhancing Safety through Automation. *Society of Automotive Engineers Gov't Industry Meeting, Automation and Connected Vehicle Safety, NHTSA.*

Katusevski, G., & Hawick, K. A. (2009). A Review of Traffic Simulation Software. *Computational Science Technical Note. CSTN 095.*

McTrans Center. (2011). CORSIM User's Manual.

National Highway Traffic Safety Administration. National Motor Vehicle Crash Causation Survey 2008.

NIST (2012). NIST/SEMATECH e-Handbook of Statistical Methods. *http://www.itl.nist.gov/div898/handbook*. Accessed May 26 2016.

National Highway Traffic Safety Administration (2013). U.S. Department of Transportation Releases Policy on Automated Vehicle Development. *http://www.nhtsa.gov/About+NHTSA/Press+Releases/U.S.+Department+of+Transportation+Releases+Policy+on+Automated+Vehicle+Development.* Accessed May 21, 2016.

Nowakowski, C., Shladover, S. E., Lu, X. Y., Thompson, D., & Kailas, A. (2015). Cooperative Adaptive Cruise Control (CACC) for Truck Platooning: Operational Concept Alternatives. *California PATH Research Report.*

Nowakowski, C., Shladover, S. E., Cody, D., Bu, F., O'Connell, J., Spring, J., Dickey, S., & Nelson, D. (2012). Cooperative Adaptive Cruise Control: Testing Drivers' Choices of following distances. *California PATH Research Report UCB-ITS-PRR-2011-01.*

Schakel, W. J., Van Arem, B., Netten, B. (2010). Effects of Cooperative Adaptive Cruise Control on Traffic Flow Stability. *13th International IEEE Conference* on *Intelligent Transportation Systems.*

Shladover, S. E., Nowakowski, C., Lu, X. Y. & Ferlis, R. (2014). Cooperative Adaptive Cruise Control (CACC) Definitions and Operating Concepts. *California PATH Research Report No.: UCB-ITS-PRR-2014-7.*

Shladover, S. E., Nowakowski, C., Lu, X. Y., & Hoogendoorn, R.(2014). Using Cooperative Adaptive Cruise Control (CACC) to form High Performance Vehicle Streams. *California PATH Research Report.*

Shladover, S. E., Su, D. & Lu, X.Y.(2012). Impacts of cooperative adaptive cruise control on freeway traffic flow. *Transportation Research Record: Journal of Transportation Research Board of National Academies. Vol 2324, pp.63-70, 2012.*

Strocko, E., Sprung, M., Nguyen, L., Rick, C., & Sedor, J. (2014). Freight Facts and Figures 2013 FHWA. *Transportation Research Board – 93rd Annual Meeting, Washington, D.C., 2014.*

Transportation Research Board (2010). Highway Capacity Manual. *Exhibit 11-2, pp. 11-3*.

Vahidi, A., & Eskandarian, A. (2003). Research Advances in Intelligent Collision Avoidance and Adaptive Cruise Control. *IEEE Transactions on Intelligent Transportation Systems, Vol. 4 (3), September 2003.*

Van Arem, B., Van Driel, C. J. G. & Visser, R. (2006). Impact of Cooperative Adaptive Cruise Control on Traffic Flow Characteristics. *IEEE Transactions on Intelligent Transportation Systems, Vol. 7(4), pp.429-436.*

VanderWerf, J., Shladover, S. E., Miller, M. A., & Kourjanskaia, N. (2002). Evaluation of the Effects of Adaptive Cruise Control Systems on Highway Traffic Flow Capacity and Implications for Deployment of Future Automated Systems, *Transportation Research Record: Journal of Transportation Research Board of National Academies. Vol. 1800, pp.78-84, 2002.*

Zhao, L. & Sun, J. (2013). Simulation Framework for Vehicle Platooning and Car-following Behaviors Under Connected-vehicle Environment. *Procedia - Social and Behavioral Sciences. Vol. 96. pp. 914–924.*

Zohdy, I., H., Kamalanathsharma, R., K., & Rakha, H. A. (2013). Intersection Management for Autonomous Vehicles using Cooperative Adaptive Cruise Control Systems. *Transportation Research Board - 92nd Annual Meeting, Washington, D.C., 2013.*

Appendix

Table A.1: Full t- results for Mixed freeway section

Traffic Volume	Time Headway	Market Penetration	Mean Speed	Std Dev	t-stat	P value	Hypothesis	Two sample p-value
100	1.25	20	68.91588316	0.1246371	1.1704636	0.1359	fail to reject H0	0.073957
100	1.25	40	69.03369003	0.0748178	3.524429	0.0032	reject H0	0.048359
100	1.25	60	69.10300187	0.0630475	5.2817652	0.0003	reject H0	0.000118
100	1.25	80	69.13874582	0.0501412	7.3541463	0.0000	reject H0	1.41E-06
100	1.25	100	69.2000335	0.0801512	5.36528	0.0002	reject H0	1.3E-06
100	1	20	68.95130011	0.1013772	1.788371	0.0537	reject H0	0.204597
100	1	40	69.00836333	0.0928153	2.5681473	0.0151	reject H0	0.227153
100	1	60	69.11237086	0.0923423	3.7076276	0.0024	reject H0	0.000839
100	1	80	69.19371752	0.0752478	5.6309616	0.0002	reject H0	9.1E-07
100	1	100	69.25506529	0.0716849	6.7666346	0.0000	reject H0	1.6E-08
100	0.75	20	68.99302515	0.0849722	2.6246836	0.0138	reject H0	0.369766
100	0.75	40	69.02720893	0.0707	3.6380313	0.0027	reject H0	0.064411
100	0.75	60	69.12708426	0.1270008	2.8116688	0.0102	reject H0	0.003015
100	0.75	80	69.16742431	0.0478567	8.3044618	0.0000	reject H0	1.13E-07
100	0.75	100	69.23526109	0.0783727	5.9365223	0.0001	reject H0	1.47E-07
100	0.5	20	68.94772709	0.1023653	1.736204	0.0583	reject H0	0.181162
100	0.5	40	69.02772151	0.0758484	3.3978501	0.0040	reject H0	0.070958
100	0.5	60	69.17079732	0.0661766	6.0564837	0.0001	reject H0	1.01E-06
100	0.5	80	69.21047313	0.0710638	6.1982742	0.0001	reject H0	1.72E-07
100	0.5	100	69.26117911	0.0433067	11.341872	0.0000	reject H0	1.79E-10
115	1.25	20	68.42565182	0.1596038	0.4739976	0.3234	fail to reject H0	0.102832
115	1.25	40	68.40027468	0.1833506	0.2741997	0.3951	fail to reject H0	0.20388
115	1.25	60	68.52884154	0.1371521	1.3039647	0.1123	fail to reject H0	0.002792
115	1.25	80	68.54781382	0.1296609	1.5256238	0.0807	fail to reject H0	0.001114
115	1.25	100	68.7222699	0.0824292	4.5162369	0.0007	reject H0	5.7E-07
115	1	20	68.44078033	0.1395169	0.6506763	0.2658	fail to reject H0	0.056398
115	1	40	68.35170018	0.165036	0.0103019	0.4960	fail to reject H0	0.429233
115	1	60	68.52769579	0.1364504	1.3022737	0.1126	fail to reject H0	0.002858
115	1	80	68.57105394	0.1535443	1.4396754	0.0919	fail to reject H0	0.001014
115	1	100	68.75607739	0.128267	3.1658752	0.0057	reject H0	5.72E-07
115	0.75	20	68.41831861	0.0929226	0.7352208	0.2405	fail to reject H0	0.070954

115	0.75	40	68.52143666	0.1173278	1.4611771	0.0890	fail to reject H0	0.002205
115	0.75	60	68.66417129	0.1498363	2.0967639	0.0327	reject H0	3.57E-05
115	0.75	80	68.79332502	0.1407716	3.1641991	0.0057	reject H0	3.34E-07
115	0.75	100	68.87590174	0.1550522	3.3917721	0.0040	reject H0	8.23E-08
115	0.5	20	68.5497604	0.1261326	1.5837334	0.0739	fail to reject H0	0.000931
115	0.5	40	68.54646555	0.1516979	1.2951102	0.1138	fail to reject H0	0.001854
115	0.5	60	68.64238779	0.139787	2.0916659	0.0330	reject H0	4.84E-05
115	0.5	80	68.72905547	0.1759647	2.1541557	0.0298	reject H0	1.7E-05
115	0.5	100	68.92444786	0.1040794	5.5193236	0.0002	reject H0	1.68E-09
130	1.25	20	66.89260219	0.5073411	-0.5270572	#NUM!	fail to reject H0	0.093125
130	1.25	40	67.1056997	0.3837342	-0.141505	#NUM!	fail to reject H0	0.36641
130	1.25	60	67.33737662	0.3258714	0.5443148	0.2997	fail to reject H0	0.128346
130	1.25	80	67.37325734	0.1201194	1.7753783	0.0548	reject H0	0.052042
130	1.25	100	67.69139451	0.3235075	1.6426034	0.0674	fail to reject H0	0.001363
130	1	20	67.18514191	0.2431329	0.1034081	0.4600	fail to reject H0	0.435653
130	1	40	67.23193805	0.3433345	0.2095276	0.4194	fail to reject H0	0.332333
130	1	60	67.33037597	0.2641482	0.6450015	0.2675	fail to reject H0	0.124496
130	1	80	67.76156123	0.1459484	4.12174	0.0013	reject H0	0.000179
130	1	100	67.96449872	0.1734884	4.6371893	0.0006	reject H0	1.14E-05
130	0.75	20	67.18861786	0.3189473	0.089726	0.4652	fail to reject H0	0.433762
130	0.75	40	67.32119466	0.2172908	-6.6675865	#NUM!	fail to reject H0	0.130179
130	0.75	60	67.62461077	0.1154869	4.0230593	0.0015	reject H0	0.001254
130	0.75	80	67.77993216	0.2361888	2.6247317	0.0138	reject H0	0.000166
130	0.75	100	67.92345565	0.213315	3.5790055	0.0030	reject H0	1.85E-05
130	0.5	20	67.22889091	0.3267736	0.2108215	0.4189	fail to reject H0	0.335762
130	0.5	40	67.38197381	0.2667303	0.8322032	0.2134	fail to reject H0	0.069035
130	0.5	60	67.53734403	0.2893037	1.304318	0.1122	fail to reject H0	0.009131
130	0.5	80	67.92666963	0.1784068	4.2973116	0.0010	reject H0	1.82E-05
130	0.5	100	68.01103299	0.2431095	3.5006157	0.0034	reject H0	6.22E-06

Table A.2: Univariate Analysis of Variance for MFS

Between-Subjects Factors

		Value Label	N
Travel Time	1	100%	200
	2	115%	200
	3	130%	200
Headway	1	1.25s	150
	2	1.00s	150
	3	0.75s	150
	4	0.5s	150
Market Penetration	1	20%	120
	2	40%	120
	3	60%	120
	4	80%	120
	5	100%	120

Tests of Between-Subjects Effects

Dependent Variable: Travel Time Gain

Source	Type III Sum of Squares	df	Mean Square	F	Sig.
Corrected Model	551.565[a]	59	9.349	15.702	.000
Intercept	559.137	1	559.137	939.146	.000
TV	79.103	2	39.551	66.432	.000
HW	48.722	3	16.241	27.278	.000
MP	304.404	4	76.101	127.822	.000
TV * HW	26.881	6	4.480	7.525	.000
TV * MP	73.732	8	9.217	15.480	.000
HW * MP	7.799	12	.650	1.092	.365
TV * HW * MP	10.925	24	.455	.765	.783
Error	321.499	540	.595		
Total	1432.201	600			
Corrected Total	873.064	599			

a. R Squared = .632 (Adjusted R Squared = .592)

Table A.3: Post hoc Tests (Tukey's Tests)

Travel Time

Multiple Comparisons

Dependent Variable: Travel Time Gain

Tukey HSD

(I) Travel Time		Mean Difference (I-J)	Std. Error	Sig.	95% Confidence Interval	
					Lower Bound	Upper Bound
100%	115%	-.558182*	.0768546	.000	-.738774	-.377591
	130%	-.878754*	.0768546	.000	-1.059346	-.698162
115%	100%	.558182*	.0768546	.000	.377591	.738774
	130%	-.320571*	.0768546	.000	-.501163	-.139980
130%	100%	.878754*	.0768546	.000	.698162	1.059346
	115%	.320571*	.0768546	.000	.139980	.501163

Based on observed means.
The error term is Mean Square(Error) = .591.

*. The mean difference is significant at the 0.05 level.

Homogenous Subsets

Travel Time Gain

Tukey HSD[a,b]

Travel Time	N	Subset		
		1	2	3
100%	200	.486369		
115%	200		1.044551	
130%	200			1.365122
Sig.		1.000	1.000	1.000

Means for groups in homogeneous subsets are displayed.
Based on observed means.
The error term is Mean Square(Error) = .591.
a. Uses Harmonic Mean Sample Size = 200.000.
b. Alpha = 0.05.

Headway

Multiple Comparisons

Dependent Variable: Travel Time Gain

Tukey HSD

(I) Headway		Mean Difference (I-J)	Std. Error	Sig.	95% Confidence Interval Lower Bound	Upper Bound
1.25s	1.00s	-.341761*	.0887440	.001	-.570414	-.113108
	0.75s	-.620831*	.0887440	.000	-.849484	-.392178
	0.5s	-.739501*	.0887440	.000	-.968154	-.510847
1.00s	1.25s	.341761*	.0887440	.001	.113108	.570414
	0.75s	-.279070*	.0887440	.009	-.507723	-.050417
	0.5s	-.397740*	.0887440	.000	-.626393	-.169086
0.75s	1.25s	.620831*	.0887440	.000	.392178	.849484
	1.00s	.279070*	.0887440	.009	.050417	.507723
	0.5s	-.118670	.0887440	.540	-.347323	.109983
0.5s	1.25s	.739501*	.0887440	.000	.510847	.968154
	1.00s	.397740*	.0887440	.000	.169086	.626393
	0.75s	.118670	.0887440	.540	-.109983	.347323

Based on observed means.
The error term is Mean Square(Error) = .591.

*. The mean difference is significant at the 0.05 level.

Homogenous Subsets

Travel Time Gain

Tukey HSD[a,b]

Headway	N	Subset 1	2	3
1.25s	150	.539824		
1.00s	150		.881585	
0.75s	150			1.160655
0.5s	150			1.279325
Sig.		1.000	1.000	.540

Means for groups in homogeneous subsets are displayed.
Based on observed means.
The error term is Mean Square(Error) = .591.

a. Uses Harmonic Mean Sample Size = 150.000.
b. Alpha = 0.05.

Market Penetration

Multiple Comparisons

Dependent Variable: Travel Time Gain

Tukey HSD

(I) Market Penetration	(J)	Mean Difference (I-J)	Std. Error	Sig.	95% Confidence Interval Lower Bound	Upper Bound
20%	40%	-.284407*	.0992189	.035	-.555913	-.012900
	60%	-.884154*	.0992189	.000	-1.155660	-.612648
	80%	-1.395791*	.0992189	.000	-1.667298	-1.124285
	100%	-1.949629*	.0992189	.000	-2.221135	-1.678122
40%	20%	.284407*	.0992189	.035	.012900	.555913
	60%	-.599747*	.0992189	.000	-.871253	-.328241
	80%	-1.111385*	.0992189	.000	-1.382891	-.839878
	100%	-1.665222*	.0992189	.000	-1.936728	-1.393716
60%	20%	.884154*	.0992189	.000	.612648	1.155660
	40%	.599747*	.0992189	.000	.328241	.871253
	80%	-.511638*	.0992189	.000	-.783144	-.240131
	100%	-1.065475*	.0992189	.000	-1.336981	-.793968
80%	20%	1.395791*	.0992189	.000	1.124285	1.667298
	40%	1.111385*	.0992189	.000	.839878	1.382891
	60%	.511638*	.0992189	.000	.240131	.783144
	100%	-.553837*	.0992189	.000	-.825343	-.282331
100%	20%	1.949629*	.0992189	.000	1.678122	2.221135
	40%	1.665222*	.0992189	.000	1.393716	1.936728
	60%	1.065475*	.0992189	.000	.793968	1.336981
	80%	.553837*	.0992189	.000	.282331	.825343

Based on observed means.
The error term is Mean Square(Error) = .591.

*. The mean difference is significant at the 0.05 level.

Homogenous Subsets

Travel Time Gain

Tukey HSD[a,b]

Market Penetration	N	Subset				
		1	2	3	4	5
20%	120	.062551				
40%	120		.346958			
60%	120			.946705		
80%	120				1.458343	
100%	120					2.012180
Sig.		1.000	1.000	1.000	1.000	1.000

Means for groups in homogeneous subsets are displayed.
Based on observed means.
The error term is Mean Square(Error) = .591.

a. Uses Harmonic Mean Sample Size = 120.000.

b. Alpha = 0.05.

Table A.4: Full t- results for isolated interchange segment

Traffic Volume	Time Headway	Market Penetration	Mean Speed	Std Dev	t-stat	P value	Hypothesis	Two sample P value
100	1.25	20	69.187634	0.1220441	-0.4290763	#NUM!	fail to reject H0	0.290362
100	1.25	40	69.221879	0.1515735	-0.1195509	#NUM!	fail to reject H0	0.424846
100	1.25	60	69.171356	0.0849105	-0.8084235	#NUM!	fail to reject H0	0.204728
100	1.25	80	69.146341	0.0403106	-2.3234412	#NUM!	fail to reject H0	0.123103
100	1.25	100	69.218006	0.0834369	-0.2636035	#NUM!	fail to reject H0	0.379456
100	1	20	69.256166	0.1750648	0.0923423	0.4642	fail to reject H0	0.460601
100	1	40	69.15176	0.1655215	-0.5331028	#NUM!	fail to reject H0	0.235267
100	1	60	69.203606	0.0653125	-0.5572314	#NUM!	fail to reject H0	0.305738
100	1	80	69.141309	0.152636	-0.6465779	#NUM!	fail to reject H0	0.199888
100	1	100	69.22731	0.1469448	-0.0863622	#NUM!	fail to reject H0	0.442199
100	0.75	20	69.263255	0.086762	0.2680279	0.3974	fail to reject H0	0.407714
100	0.75	40	69.069712	0.2258664	-0.7539304	#NUM!	fail to reject H0	0.15904
100	0.75	60	69.222418	0.0696098	-0.2525748	#NUM!	fail to reject H0	0.393989
100	0.75	80	69.3408	0.0271705	3.7099146	0.0024	reject H0	0.123524
100	0.75	100	69.154481	0.0492983	-1.7347226	#NUM!	fail to reject H0	0.140464
100	0.5	20	69.253996	0.1732516	0.0807835	0.4687	fail to reject H0	0.467104
100	0.5	40	69.209992	0.1037598	-0.2892023	#NUM!	fail to reject H0	0.356972
100	0.5	60	69.296171	0.091787	0.6119712	0.2778	fail to reject H0	0.275202
100	0.5	80	69.263238	0.0669402	0.3471386	0.3682	fail to reject H0	0.400328
100	0.5	100	69.258291	0.0747601	0.2446611	0.4061	fail to reject H0	0.427065
115	1.25	20	68.388662	0.2367744	0.2055219	0.4209	fail to reject H0	0.446764
115	1.25	40	68.481039	0.3315181	0.4254343	0.3403	fail to reject H0	0.354591
115	1.25	60	68.919084	0.1410405	4.1057959	0.0013	reject H0	0.085661
115	1.25	80	68.203722	1.1385376	-0.1196954	#NUM!	fail to reject H0	0.429787
115	1.25	100	68.620434	0.1872123	1.4979473	0.0842	fail to reject H0	0.218602
115	1	20	68.806978	0.4751399	0.9828225	0.1757	fail to reject H0	0.152167
115	1	40	68.749111	0.4233287	0.9664134	0.1795	fail to reject H0	0.169805
115	1	60	68.783749	0.5119285	0.8668183	0.2043	fail to reject H0	0.171021
115	1	80	68.614402	0.2868522	0.9565984	0.1819	fail to reject H0	0.232223
115	1	100	68.245553	0.4410568	-0.2141381	#NUM!	fail to reject H0	0.405623
115	0.75	20	68.395644	0.4526585	0.1229276	0.4524	fail to reject H0	0.448752
115	0.75	40	68.280994	0.3133514	-0.1883068	#NUM!	fail to reject H0	0.432294
115	0.75	60	68.703957	0.425232	0.8559029	0.2071	fail to reject H0	0.195372
115	0.75	80	68.717283	0.2962599	1.273487	0.1174	fail to reject H0	0.166659
115	0.75	100	68.797431	0.2406004	1.9012047	0.0449	reject H0	0.123872
115	0.5	20	68.314249	0.1995474	-0.1290468	#NUM!	fail to reject H0	0.466152
115	0.5	40	68.808156	0.3082365	1.5188214	0.0816	fail to reject H0	0.124449
115	0.5	60	68.53162	0.4350928	0.4404122	0.3350	fail to reject H0	0.322179
115	0.5	80	68.824392	0.1389023	3.487289	0.0034	reject H0	0.113046
115	0.5	100	68.972714	0.2697986	2.3451361	0.0218	reject H0	0.071574
130	1.25	20	66.193051	0.9527723	-0.7839742	#NUM!	fail to reject H0	0.154743
130	1.25	40	66.826818	0.8665268	-0.1306154	#NUM!	fail to reject H0	0.424682
130	1.25	60	67.179149	1.0086148	0.2371065	0.4089	fail to reject H0	0.369184
130	1.25	80	66.266723	0.3324355	-2.0252849	#NUM!	fail to reject H0	0.055678
130	1.25	100	67.600234	1.3638681	0.4840896	0.3199	fail to reject H0	0.249207
130	1	20	67.368158	0.7332401	0.5839257	0.2868	fail to reject H0	0.225184
130	1	40	66.749886	1.0030847	-0.1895295	#NUM!	fail to reject H0	0.39095
130	1	60	67.625703	0.3575721	1.9176625	0.0437	reject H0	0.056821
130	1	80	67.250259	0.2046036	1.5163895	0.0819	fail to reject H0	0.184336

130	1	100	67.343654	0.8179663	0.4934852	0.3167	fail to reject H0	0.255175
130	0.75	20	66.397528	0.3307285	-1.6402333	#NUM!	fail to reject H0	0.086725
130	0.75	40	67.129115	0.9650636	0.1959615	0.4245	fail to reject H0	0.391855
130	0.75	60	66.839583	1.2827154	-0.0782845	#NUM!	fail to reject H0	0.452325
130	0.75	80	67.715089	0.2750156	2.8183453	0.0101	reject H0	0.03917
130	0.75	100	67.421115	0.2727897	1.7636851	0.0558	reject H0	0.103855
130	0.5	20	65.442618	1.2243956	-1.2229559	#NUM!	fail to reject H0	0.078504
130	0.5	40	66.374191	1.0034363	-0.5638711	#NUM!	fail to reject H0	0.220518
130	0.5	60	66.344359	0.4391307	-1.3564096	#NUM!	fail to reject H0	0.087631
130	0.5	80	66.615768	0.7985263	-0.4060378	#NUM!	fail to reject H0	0.288968
130	0.5	100	66.907058	0.9834649	-0.0334963	#NUM!	fail to reject H0	0.478686

Table A.5: Univariate Analysis of Variance for IIS

Between-Subjects Factors

		Value Label	N
Travel Time	1	100%	60
	2	115%	60
	3	130%	60
Headway	1	1.25s	45
	2	1.00s	45
	3	0.75s	45
	4	0.5s	45
Market Penetration	1	20%	36
	2	40%	36
	3	60%	36
	4	80%	36
	5	100%	36

Tests of Between-Subjects Effects

Dependent Variable: Travel Time Gain

Source	Type III Sum of Squares	df	Mean Square	F	Sig.
Corrected Model	2.558[a]	59	.043	1.534	.025
Intercept	.040	1	.040	1.428	.234
TV	.355	2	.178	6.287	.003
HW	.198	3	.066	2.331	.078
MP	.283	4	.071	2.500	.046
HW * MP	.453	12	.038	1.335	.208

TV * HW	.551	6	.092	3.249	.005
TV * MP	.344	8	.043	1.522	.157
TV * HW * MP	.374	24	.016	.552	.954
Error	3.392	120	.028		
Total	5.990	180			
Corrected Total	5.949	179			

a. R Squared = .430 (Adjusted R Squared = .150)

Between-Subjects Factors

		Value Label	N
Travel Time	1	100%	60
	2	115%	60
	3	130%	60
Headway	1	1.25s	45
	2	1.00s	45
	3	0.75s	45
	4	0.5s	45
Market Penetration	1	20%	36
	2	40%	36
	3	60%	36
	4	80%	36
	5	100%	36

Tests of Between-Subjects Effects

Dependent Variable: Travel Time Gain

Source	Type III Sum of Squares	df	Mean Square	F	Sig.
Corrected Model	1.387[a]	15	.092	3.323	.000
Intercept	.040	1	.040	1.451	.230
TV	.355	2	.178	6.387	.002
MP	.283	4	.071	2.540	.042
TV * HW	.749	9	.083	2.990	.003
Error	4.563	164	.028		
Total	5.990	180			
Corrected Total	5.949	179			

a. R Squared = .233 (Adjusted R Squared = .163)

Table A.6: Post Hoc Tests (Tukey's Tests)

Travel Time

Multiple Comparisons

Dependent Variable: Travel Time Gain

Tukey HSD

(I) Travel Time		Mean Difference (I-J)	Std. Error	Sig.	95% Confidence Interval	
					Lower Bound	Upper Bound
100%	115%	-.086137*	.0304523	.014	-.158163	-.014112
	130%	.014553	.0304523	.882	-.057472	.086578
115%	100%	.086137*	.0304523	.014	.014112	.158163
	130%	.100690*	.0304523	.003	.028665	.172716
130%	100%	-.014553	.0304523	.882	-.086578	.057472
	115%	-.100690*	.0304523	.003	-.172716	-.028665

Based on observed means.
The error term is Mean Square(Error) = .028.

*. The mean difference is significant at the 0.05 level.

Homogenous Subsets

Travel Time Gain

Tukey HSD[a,b]

Travel Time	N	Subset	
		1	2
130%	60	-.023438	
100%	60	-.008885	
115%	60		.077252
Sig.		.882	1.000

Means for groups in homogeneous subsets are displayed.
Based on observed means.
The error term is Mean Square(Error) = .028.
a. Uses Harmonic Mean Sample Size = 60.000.

b. Alpha = 0.05.

Market Penetration

Multiple Comparisons

Dependent Variable: Travel Time Gain

Tukey HSD

(I) Market Penetration		Mean Difference (I-J)	Std. Error	Sig.	95% Confidence Interval	
					Lower Bound	Upper Bound
20%	40%	-.046768	.0393137	.757	-.155210	.061673
	60%	-.090942	.0393137	.146	-.199384	.017499
	80%	-.073320	.0393137	.340	-.181761	.035122
	100%	-.115518*	.0393137	.031	-.223959	-.007076
40%	20%	.046768	.0393137	.757	-.061673	.155210
	60%	-.044174	.0393137	.794	-.152616	.064268
	80%	-.026551	.0393137	.961	-.134993	.081890
	100%	-.068750	.0393137	.407	-.177191	.039692
60%	20%	.090942	.0393137	.146	-.017499	.199384
	40%	.044174	.0393137	.794	-.064268	.152616
	80%	.017623	.0393137	.992	-.090819	.126064
	100%	-.024576	.0393137	.971	-.133017	.083866
80%	20%	.073320	.0393137	.340	-.035122	.181761
	40%	.026551	.0393137	.961	-.081890	.134993
	60%	-.017623	.0393137	.992	-.126064	.090819
	100%	-.042198	.0393137	.820	-.150640	.066243
100%	20%	.115518*	.0393137	.031	.007076	.223959
	40%	.068750	.0393137	.407	-.039692	.177191
	60%	.024576	.0393137	.971	-.083866	.133017
	80%	.042198	.0393137	.820	-.066243	.150640

Based on observed means.
The error term is Mean Square(Error) = .028.
*. The mean difference is significant at the 0.05 level.

Homogenous Subsets

Travel Time Gain

Tukey HSD[a,b]

Market Penetration	N	Subset	
		1	2
20%	36	-.050333	
40%	36	-.003565	-.003565
80%	36	.022986	.022986
60%	36	.040609	.040609
100%	36		.065184
Sig.		.146	.407

Means for groups in homogeneous subsets are displayed.
Based on observed means.
The error term is Mean Square(Error) = .028.
a. Uses Harmonic Mean Sample Size = 36.000.
b. Alpha = 0.05.

Table A.7: Full t- results for basic freeway section

Traffic Volume	Time Headway	Market Penetration	Mean Speed	Std Dev	t-stat	P value	Hypothesis	2 sample p value
100	1.25	20	66.570847	0.1598625	0.5057275	0.3126	fail to reject H0	0.281142644
100	1.25	40	66.350695	0.0566652	-2.4583821	#NUM!	fail to reject H0	0.113192906
100	1.25	60	66.505547	0.1248813	0.1244931	0.4518	fail to reject H0	0.454923264
100	1.25	80	66.317986	0.1067683	-1.6110977	#NUM!	fail to reject H0	0.087386259
100	1.25	100	66.36574	0.0664666	-1.8695159	#NUM!	fail to reject H0	0.134162721
100	1	20	66.560654	0.0724515	0.9751868	0.1775	fail to reject H0	0.258932751
100	1	40	66.376813	0.1927431	-0.5872425	#NUM!	fail to reject H0	0.22843855
100	1	60	66.545638	0.1491192	0.3731133	0.3589	fail to reject H0	0.339594693
100	1	80	66.476217	0.103032	-0.1337746	#NUM!	fail to reject H0	0.442143858
100	1	100	66.353858	0.2610874	-0.5214419	#NUM!	fail to reject H0	0.239327186
100	0.75	20	66.598871	0.11525	0.9446528	0.1848	fail to reject H0	0.188568635
100	0.75	40	66.499212	0.0651794	0.141332	0.4454	fail to reject H0	0.472181819
100	0.75	60	66.596811	0.1394629	0.7658711	0.2317	fail to reject H0	0.209203776
100	0.75	80	66.55522	0.161966	0.4026776	0.3483	fail to reject H0	0.321052775
100	0.75	100	66.427862	0.1906968	-0.3258465	#NUM!	fail to reject H0	0.333655847
100	0.5	20	66.586053	0.097878	0.9813567	0.1760	fail to reject H0	0.205798942
100	0.5	40	66.488094	0.0949875	-0.020061	#NUM!	fail to reject H0	0.484654678
100	0.5	60	66.502519	0.1863151	0.067191	0.4739	fail to reject H0	0.47179576
100	0.5	80	66.639495	0.1205552	1.2400563	0.1232	fail to reject H0	0.124608741
100	0.5	100	66.640996	0.0160486	9.4087208	0.0000	reject H0	0.107859531
115	1.25	20	66.081618	0.0724054	1.9559026	0.0411	reject H0	0.115725386
115	1.25	40	66.06975	0.142808	0.9085644	0.1936	fail to reject H0	0.173102265
115	1.25	60	65.920014	0.0796345	-0.2509656	#NUM!	fail to reject H0	0.391142606
115	1.25	80	65.911143	0.0871553	-0.3311011	#NUM!	fail to reject H0	0.360720988
115	1.25	100	65.771463	0.0768727	-2.1924225	#NUM!	fail to reject H0	0.072165565
115	1	20	65.883933	0.1755563	-0.3193658	#NUM!	fail to reject H0	0.324197801
115	1	40	66.012325	0.1348965	0.5361541	0.3024	fail to reject H0	0.29594668
115	1	60	66.034242	0.1135324	0.830089	0.2140	fail to reject H0	0.224565682
115	1	80	65.960613	0.051093	0.4034484	0.3480	fail to reject H0	0.444626891
115	1	100	65.951502	0.15933	0.0721899	0.4720	fail to reject H0	0.487956051
115	0.75	20	66.064122	0.2213673	0.5607077	0.2943	fail to reject H0	0.244888979
115	0.75	40	65.989769	0.1815615	0.2741189	0.3951	fail to reject H0	0.382812675

115	0.75	60	66.129135	0.1117823	1.6919909	0.0625	reject H0	0.076899515
115	0.75	80	66.114106	0.136925	1.2715398	0.1177	fail to reject H0	0.105771132
115	0.75	100	66.109844	0.0324731	5.230295	0.0003	reject H0	0.086147057
115	0.5	20	66.118944	0.1051251	1.7021997	0.0615	reject H0	0.083311794
115	0.5	40	66.095005	0.0439167	3.5295165	0.0032	reject H0	0.098785245
115	0.5	60	66.215609	0.1772651	1.5547846	0.0772	fail to reject H0	0.055837175
115	0.5	80	66.229349	0.0077665	37.256128	0.0000	reject H0	0.035335893
115	0.5	100	66.249273	0.2149469	1.4388329	0.0920	fail to reject H0	0.061110682
130	1.25	20	65.485095	0.1407931	-1.1712579	#NUM!	fail to reject H0	0.090129516
130	1.25	40	65.523584	0.089401	-1.4140361	#NUM!	fail to reject H0	0.09730823
130	1.25	60	65.450478	0.2080756	-0.9588922	#NUM!	fail to reject H0	0.112852519
130	1.25	80	65.414726	0.2623078	-0.8969383	#NUM!	fail to reject H0	0.123304726
130	1.25	100	65.355438	0.049956	-5.8964249	#NUM!	fail to reject H0	0.015758967
130	1	20	65.64757	0.0462894	-0.0524916	#NUM!	fail to reject H0	0.450176929
130	1	40	65.512079	0.108554	-1.2705287	#NUM!	fail to reject H0	0.094806649
130	1	60	65.484575	0.0845392	-1.9567844	#NUM!	fail to reject H0	0.056539518
130	1	80	65.403207	0.0091465	-26.982237	#NUM!	fail to reject H0	0.031414001
130	1	100	65.388626	0.0602373	-4.3390776	#NUM!	fail to reject H0	0.019197581
130	0.75	20	65.463302	0.1230129	-1.5177124	#NUM!	fail to reject H0	0.059267305
130	0.75	40	65.570282	0.0648352	-1.2295516	#NUM!	fail to reject H0	0.168571884
130	0.75	60	65.594377	0.1891767	-0.2940288	#NUM!	fail to reject H0	0.326594327
130	0.75	80	65.624008	0.0792805	-0.3278501	#NUM!	fail to reject H0	0.352200933
130	0.75	100	65.445729	0.1539664	-1.3267252	#NUM!	fail to reject H0	0.067815354
130	0.5	20	65.635619	0.0428948	-0.3352508	#NUM!	fail to reject H0	0.391707167
130	0.5	40	65.619448	0.1587478	-0.1924543	#NUM!	fail to reject H0	0.377950781
130	0.5	60	65.78955	0.0528124	2.6423661	0.0134	reject H0	0.089819117
130	0.5	80	65.623402	0.1287924	-0.20652	#NUM!	fail to reject H0	0.375457021
130	0.5	100	65.740085	0.030311	2.9720091	0.0078	reject H0	0.172766574

Tabl A.7: Univariate Analysis of Variance for BFS

Between-Subjects Factors

		Value Label	N
Travel Time	1	100%	60
	2	115%	60
	3	130%	60
Headway	1	1.25s	45
	2	1.00s	45
	3	0.75s	45
	4	0.5s	45
Market Penetration	1	20%	36
	2	40%	36
	3	60%	36
	4	80%	36
	5	100%	36

Tests of Between-Subjects Effects

Dependent Variable: Travel Time Gain

Source	Type III Sum of Squares	df	Mean Square	F	Sig.
Corrected Model	161.578[a]	59	2.739	3.728	.000
Intercept	.280	1	.280	.381	.538
TV	63.080	2	31.540	42.932	.000
HW	49.485	3	16.495	22.453	.000
MP	7.189	4	1.797	2.446	.050
TV * HW	3.348	6	.558	.759	.603
TV * MP	4.860	8	.607	.827	.581
HW * MP	17.335	12	1.445	1.966	.033
TV * HW * MP	16.283	24	.678	.923	.571
Error	88.157	120	.735		
Total	250.016	180			
Corrected Total	249.736	179			

a. R Squared = .647 (Adjusted R Squared = .473)

Table A.9: Post Hoc Tests (Tukey's Tests)

Travel Time

Multiple Comparisons

Dependent Variable: Travel Time Gain

Tukey HSD

(I) Travel Time		Mean Difference (I-J)	Std. Error	Sig.	95% Confidence Interval	
					Lower Bound	Upper Bound
100%	115%	-.610247*	.1541598	.000	-.974991	-.245504
	130%	.834041*	.1541598	.000	.469298	1.198785
115%	100%	.610247*	.1541598	.000	.245504	.974991
	130%	1.444288*	.1541598	.000	1.079545	1.809032
130%	100%	-.834041*	.1541598	.000	-1.198785	-.469298
	115%	-1.444288*	.1541598	.000	-1.809032	-1.079545

Based on observed means.
The error term is Mean Square(Error) = .713.

*. The mean difference is significant at the 0.05 level.

Homogenous Subsets

Travel Time Gain

Tukey HSD[a,b]

Travel Time	N	Subset		
		1	2	3
130%	60	-.798878		
100%	60		.035163	
115%	60			.645411
Sig.		1.000	1.000	1.000

Means for groups in homogeneous subsets are displayed.
Based on observed means.
The error term is Mean Square(Error) = .713.
a. Uses Harmonic Mean Sample Size = 60.000.

b. Alpha = 0.05.

Multiple Comparisons

Dependent Variable: Travel Time Gain

Tukey HSD

(I) Headway	Mean Difference	Std. Error	Sig.	95% Confidence Interval

		(I-J)			Lower Bound	Upper Bound
1.25s	1.00s	-.219967	.1780084	.605	-.682180	.242247
	0.75s	-.743998*	.1780084	.000	-1.206211	-.281784
	0.5s	-1.358944*	.1780084	.000	-1.821157	-.896730
1.00s	1.25s	.219967	.1780084	.605	-.242247	.682180
	0.75s	-.524031*	.1780084	.019	-.986245	-.061818
	0.5s	-1.138977*	.1780084	.000	-1.601191	-.676764
0.75s	1.25s	.743998*	.1780084	.000	.281784	1.206211
	1.00s	.524031*	.1780084	.019	.061818	.986245
	0.5s	-.614946*	.1780084	.004	-1.077160	-.152733
0.5s	1.25s	1.358944*	.1780084	.000	.896730	1.821157
	1.00s	1.138977*	.1780084	.000	.676764	1.601191
	0.75s	.614946*	.1780084	.004	.152733	1.077160

Based on observed means.
The error term is Mean Square(Error) = .713.

*. The mean difference is significant at the 0.05 level.

Homogenous Subsets

Travel Time Gain

Tukey HSD[a,b]

Headway	N	Subset 1	2	3
1.25s	45	-.620162		
1.00s	45	-.400195		
0.75s	45		.123836	
0.5s	45			.738782
Sig.		.605	1.000	1.000

Means for groups in homogeneous subsets are displayed.
Based on observed means.
The error term is Mean Square(Error) = .713.
a. Uses Harmonic Mean Sample Size = 45.000.
b. Alpha = 0.05.

Market Penetration

Multiple Comparisons

Dependent Variable: Travel Time Gain

Tukey HSD

(I) Market Penetration		Mean Difference (I-J)	Std. Error	Sig.	95% Confidence Interval Lower Bound	Upper Bound
20%	40%	.319561	.1990194	.496	-.229640	.868762
	60%	-.040787	.1990194	1.000	-.589988	.508414
	80%	.234628	.1990194	.763	-.314573	.783829
	100%	.493646	.1990194	.100	-.055555	1.042847
40%	20%	-.319561	.1990194	.496	-.868762	.229640
	60%	-.360347	.1990194	.371	-.909548	.188854
	80%	-.084933	.1990194	.993	-.634134	.464268
	100%	.174085	.1990194	.906	-.375116	.723286
60%	20%	.040787	.1990194	1.000	-.508414	.589988
	40%	.360347	.1990194	.371	-.188854	.909548
	80%	.275415	.1990194	.639	-.273786	.824616
	100%	.534432	.1990194	.061	-.014769	1.083633
80%	20%	-.234628	.1990194	.763	-.783829	.314573
	40%	.084933	.1990194	.993	-.464268	.634134
	60%	-.275415	.1990194	.639	-.824616	.273786
	100%	.259018	.1990194	.691	-.290183	.808219
100%	20%	-.493646	.1990194	.100	-1.042847	.055555
	40%	-.174085	.1990194	.906	-.723286	.375116
	60%	-.534432	.1990194	.061	-1.083633	.014769
	80%	-.259018	.1990194	.691	-.808219	.290183

Based on observed means.
The error term is Mean Square(Error) = .713.

Homogenous Subsets

Travel Time Gain

Tukey HSD[a,b]

Market Penetration	N	Subset 1
100%	36	-.331671
40%	36	-.157586
80%	36	-.072653
20%	36	.161975
60%	36	.202762
Sig.		.061

Means for groups in homogeneous subsets are displayed.
Based on observed means.
The error term is Mean Square(Error) = .713.

a. Uses Harmonic Mean Sample Size = 36.000.
b. Alpha = 0.05.