The Thesis Committee for Edoardo Espinoza
Certifies that this is the approved version of the following Thesis:

Changes in Freeway Level of Service with the Introduction of Autonomous and Connected Vehicles

APPROVED BY

SUPERVISING COMMITTEE:

Randy B. Machemehl, Supervisor

Zhanmin Zhang

Changes in Freeway Level of Service with the Introduction of Autonomous and Connected Vehicles

by

Edoardo Espinoza

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Dedication

I would like to dedicate this accomplishment to my beautiful wife, Adriana Espinoza, who has been my life partner and best friend through this wonderful experience. This accomplishment is also dedicated to my unborn child who we are awaiting with open arms. To all of my future children who I will love and cherish immensely with my wonderful wife by my side. May all this hard work serve as an inspiration to you all and be the beginning of a prosperous future.

Me gustaría dedicarle este logro a mi hermosa esposa, Adriana Espinoza, quien a sido mi compañera y mejor amiga durante esta maravillosa experiencia. Este logro también es dedicado a nuestro hijo/a a quien esperamos con los brazos abiertos, y para todos los hijos por venir con mi esposa. Que todo este trabajo sirva como inspiración para todos ustedes y sea el comienzo de un futuro próspero.

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Para mis padres quienes me ensenaron el valor del trabajo duro, en especial a mi madre, Eloisa Espinoza, quien siempre me impulsó a seguir mis sueños, incluso cuando yo mismo dudaba de mí mismo.

To my beautiful wife, Adriana Espinoza, who has been by my side through the good and the bad. To her that put up with the long nights and taxing moments during this wonderful adventure.

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Abstract

Changes in Freeway Level of Service with the Introduction of Autonomous and Connected Vehicles

Edoardo Espinoza, M.S.E

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Supervisor: Randy B. Machemehl

Connected and Autonomous vehicles (CAVs) have risen in popularity in recent years and are expected to bring with them many changes including driver safety, expansion of ridership from people currently unable to drive, and more travel miles from long trip commuters. From an engineering standpoint, CAVs are expected to bring with them an increase in highway capacities because of their ability to react faster than human drivers and produce shorter time headways between successive vehicles. CAVs are not anticipated to dominate the traffic stream until another 20 to 40 years and are expected to be introduced gradually into the transportation system. Shorter time headways suggest that freeways may be positively affected by the new technology and new procedures will need to be established in order to analyze highway capacities in the future. The 6th Edition Highway Capacity Manual is one of the main sources that is used by the engineering community to estimate capacities of freeway segments. This study documents a new simulation tool to discover the capacity implications for a basic freeway segment of different CAV market penetrations and reduced time headways.

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1.0 Introduction

Over the past six decades, there has been a growing need to classify highways with respect to their ability to move vehicles, otherwise known as highway capacity. Understanding highway capacity allows traffic engineers and planning professionals to design ahead for future traffic growth and build highway facilities according to projected transportation demands.

Highway capacities and levels of service are directly affected by the number of vehicles that utilize the system at a particular time. This demand is usually referred to as volume or flow rate and is directly related to the capacity of highways, which in turn reflects the maximum volume that a highway can withstand before full congestion occurs. For the past 66 years, one of the main resources utilized to estimate a highway's capacity has been the Highway Capacity Manual (HCM) regulated by the Transportation Research Board (TRB) and in collaboration with the Federal Highway Administration (FHWA), American Association of State Highway and Transportation Officials (AASHTO), and many other individual groups within academic and professional fields. One specific detail that has remained generally consistent in the traffic stream has been the human behavioral component. However, the advancements in autonomous and connected vehicles implies that the human behavioral component will not remain homogeneous throughout the traffic stream forever, meaning traditional capacity calculations will need updating.

The idea of introducing autonomous vehicles (AVs), connected vehicles (CVs), and connected-autonomous vehicles (CAVs) into the traffic stream has become quite popular in the past few years. CAVs are expected to have a significant positive impact on highways with respect to safety issues and by allowing longer trips to be made from

farther places of origin [1]. Because of their ability to not require human interaction, CAVs are expected to remove human error behind crashes, improve public safety, and allow people to reside farther away from their place of employment. In addition to the advantages mentioned above, there are also engineering challenges that are expected to benefit from the introduction of CAVs into the traffic stream.

Two major challenges that CAVs can address are 1) improving capacity on roadways and 2) improving travel times. CAVs can increase highway capacity with the introduction of lower time headways between successive vehicles. Along with increased capacity, CAVs can also reduce travel times due to shorter headways through the use of platooning or coordinated cruise control and may yield 80% increases with 90% market penetration [2]. This field of study has been explored through a number of researchers who have estimated various differences in ranges of time headways of CAVs in comparison to human perception-reaction times and headways. Researchers anticipate that overall time headways with high percentages of CAV market penetration will reduce sufficiently to make a difference in overall highway capacity values.

The HCM has been consistently used in the past as a guide to estimate capacities and quality of level of service (LOS) of arterial, signalized & unsignalized intersections, roundabouts, freeways, and many other roadways within the transportation network. The focus of this study will be to investigate one section of the HCM's vast set of roadway scenarios and how present-day guidelines may change with the inclusion of CAVs in the traffic stream. The Basic Freeway section under Volume II Uninterrupted Flow of the 6th edition Highway Capacity Manual (HCM) will serve as the stepping stone to understand what effects can be expected from the inclusion of CAVs and their corresponding lower time headways. The main focus of the study will be to estimate new capacities and levels of service of a basic freeway with respect to lower time headways produced from CAVs.

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The underlying question behind the study is "Will basic freeways experience an increase in capacity with the inclusion of autonomous and connected vehicles?" Secondly, "What are the new capacities that can be expected at different freeway speeds?" Because the HCM is commonly used to find the level of service of a highway, the last question is "How and where will the LOS thresholds change in the Speed vs Flow chart with the introduction of autonomous and connected vehicles?" The study looks for a correlation between lower time headways and higher capacity values. Simulation of vehicle and driver behavioral models is used to devise a new speed vs. flow graph depicting potential changes in traffic stream characteristics caused by autonomous and connected vehicles.

2.0 Literature Review

There has been a certain intrigue with autonomous and connected vehicles (CAVs) in the past few years. One of the main reasons for its positive acceptance is the hope of reducing the 2.2 million Americans that are injured each year in automobile crashes, leading to approximately 30,000 fatalities per year [1]. There are many positive safety aspects that can be addressed by CAVs. It is estimated that around 90% of crashes are due to human driver error [1]. With the help of CAVs, this number can be greatly reduced and can help avoid costly results. For this and other reasons, CAVs have become increasingly popular and are expected to grow in number in the coming years [3]. Therefore, it is important to understand the effects that CAVs will have on the traffic system in the near future.

Distinction Between Levels of Autonomy

It is important to first distinguish between the different levels of vehicle autonomy before we can create a representative model. There are currently 4 levels of autonomy, not including no automation, set by the U.S. Department of Transportation's National Highway Traffic Safety Administration (NHTSA) [4]. The five levels are as follow:

Table 2- 1: Levels of Vehicle Autonomy Based on the National Highway Traffic Safety Administration (NHTSA)

Levels of Autonomy per National Highway Traffic Safety Administration
Level 0 – No-Automation. The driver is in complete and sole control of the primary vehicle controls (brake, steering, throttle, and motive power) at all times, and is solely responsible for monitoring the roadway and for safe operation of all vehicle controls.
Level 1 – Function-specific Automation: Automation at this level involves one or more specific control functions; if multiple functions are automated, they operate independently from each other. The driver has overall control, and is solely responsible for safe operation, but can choose to cede limited authority over a primary control (as in adaptive cruise control), the vehicle can automatically assume limited authority over a primary control (as in electronic stability control), or the automated system can provide added control to aid the driver in certain normal driving or crash-imminent situations (e.g., dynamic brake support in emergencies).
Level 2 - Combined Function Automation: This level involves automation of at least two primary control functions designed to work in unison to relieve the driver of control of those functions. Vehicles at this level of automation can utilize shared authority when the driver cedes active primary control in certain limited driving situations. The driver is still responsible for monitoring the roadway and safe operation and is expected to be available for control at all times and on short notice. An example of combined functions enabling a Level 2 system is adaptive cruise control in combination with lane centering.
Level 3 - Limited Self-Driving Automation: Vehicles at this level of automation enable the driver to cede full control of all safety-critical functions under certain traffic or environmental conditions and in those conditions to rely heavily on the vehicle to monitor for changes in those conditions requiring transition back to driver control. The driver is expected to be available for occasional control, but with sufficiently comfortable transition time. An example would be an automated or self-driving car that can determine when the system is no longer able to support automation, such as from an oncoming construction area, and then signals to the driver to reengage in the driving task, providing the driver with an appropriate amount of transition time to safely regain manual control.
Level 4 - Full Self-Driving Automation (Level 4): The vehicle is designed to perform all safety-critical driving functions and monitor roadway conditions for an entire trip. Such a design anticipates that the driver1 will provide destination or navigation input, but is not expected to be available for control at any time during the trip.

Figure 2- 1: Autonomous Vehicle Level 0 to Level 4
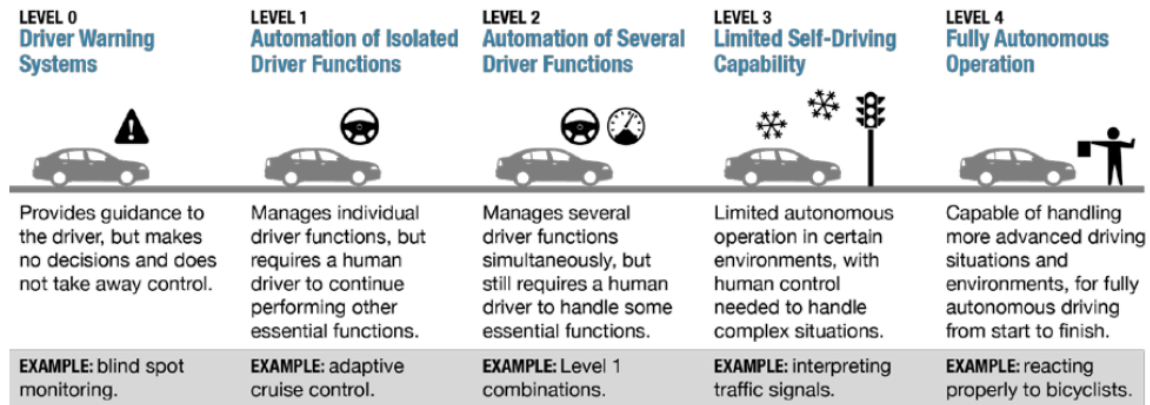
The levels of automated vehicles can be easily misunderstood; therefore, it is important to distinguish between the vehicles that will be represented in the study. The focus of this study will be concentrating specifically on fully automated vehicles that correspond to level 4 self-driving automation.

Public Opinion of Automated Vehicles

Public opinion of the different categories of automated vehicles has been researched throughout the United States. One such research surveyed 2167 Americans throughout the United States to develop an estimate of automated vehicle adoption with respect to driver willingness to pay (WTP) and annual price drop of CAV costs. The research suggested that there can be a 24.8% adoption rate of level 4 vehicles by the year 2045 with a constant 5% price drop and WTP, and as high as 87.2% adoption rate with a constant 10% price drop and WTP [2]. Nevertheless, certain policies and manufacturer incentives must first be present for level 4 automation adoption rates to increase. It is unlikely that the traffic system will reach level 4 homogeneity unless there are significant reductions in technology costs and policies promoting ownership of automated vehicles.

Time Headways and Market Penetration

One very important aspect to consider regarding the effects that CAVs will have in the traffic stream is the expected level of market penetration. The important attribute behind this idea lies in the ability of CAVs to maintain shorter following distances between consecutive vehicles. Human-driven vehicles (HVs) have longer time headways compared to CAVs because humans have a longer perception-reaction time than computers controlling CAV's. The distance that a HV or CAV must maintain as a safe driving distance ahead of preceding vehicles is dependent upon the perception-reaction time. Time headways for HVs have been found to range between 0.7 to 2.4 seconds [5]. Aggressive drivers usually fall within the lower threshold, while more passive drivers are associated with higher time headways. For the same reason that human drivers cannot accurately predict what a preceding HV will do, CAVs are not expected to accurately predict the actions of a preceding human-driven vehicle. In certain instances, this

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scenario may even negatively affect the capacity of highways by producing greater time headways than HV-to-HV interaction [6]. For this reason, similar time headways to those between HVs have been found for CAVs following HVs and vice-versa. Time headways for CAVs following HVs and vice-versa range from 0.5 to 2.6 seconds [5]. Nonetheless, time headways are mainly expected to change under the scenario where CAVs follow other CAVs. Values of time headways range from 0.3 to a maximum of 2.0 seconds [5].

Table 2- 2: Range of HV and CAV Time Headways Based on Different Studies

Table 1
A list of the reviewed studies proposing models for headway distributions.

Studies	Headway between HVs		Headway for a CAV following an HV		Headway for an HV following a CAV		Headway between CAVs	
	Distribution	Range (sec)	Distribution	Range (sec)	Distribution	Range (sec)	Distribution	Range (sec)
Neubert et al. (1999)	Uniform	0.8–1.8						
Fancher et al. (2001)	Uniform	1.0–2.0						
VanderWerf et al. (2001)			Uniform	1.0–2.0			Uniform	0.5–1.4
Bose and Ioannou (2003)	Uniform	0.7–2.2	Uniform	0.5–1.5				
Nowakowski et al. (2010)			Uniform	1.1–2.2			Uniform	0.6–1.1
Schakel et al. (2010)			Gaussian	1.2±0.15/ 1.2±0.3			Gaussian	1.2±0.15/ 1.2±0.3
Calvert et al. (2012)							Uniform	0.3–1.4
Larsson (2012)			Uniform	1.0–2.6				
Altay et al. (2013)			Uniform	0.6–2.0			Uniform	0.6–2.0
Zhao and Sun (2013)			Fixed	1.4			Fixed	0.5
Allam Ahmed et al. (2014)	Uniform	1.3–2.4						
Arnaout and Bowling (2014)	Uniform	1.0–1.8	Uniform	0.8–1.0	Uniform	1.0–1.8	Fixed	0.5
Shladover et al. (2014)					Uniform	0.6–2.2		
Gao et al. (2015)					Uniform	0.6–2.6		
Nikolos et al. (2015)			Uniform	0.8–2.2				
Roncoli et al. (2015)			Uniform	0.5–2.0				
Wang et al. (2015)			Uniform	0.5–2.0				

Due to the slow but steady nature of CAV market penetration, the environment will surely be composed of a mixed traffic stream where portions of vehicles will be made up of CAVs and others of HVs [5]. Because it is not realistic to believe that full market penetration will be achieved in the very near future, it is important to consider how the market penetration intensities of automated vehicles will play a part in the analysis. For the reasons mentioned above, it is anticipated that the greater the market penetration of CAVs in the system, the more likely time headways will drop to significant values as represented by the CAV to CAV interaction. Through the use of communication and automated control technologies, such as platooning, a reduction of

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significant time headways can be achieved [5]. Nevertheless, it is worth noting that research from literature has shown that a conservative approach of assigning higher time headways to the traffic stream may result in a decrease in capacity, while an aggressive approach can yield the opposite effect [5]. It is anticipated that the more CAVs are scattered across the system, the less likely platooning will have a significant effect on the overall throughput of the system. By the same effect, if CAVs are more clustered, then time headways might significantly decrease [5].

US Dept. of Transportation Plan for Updating the 6th ed. Highway Capacity Manual

Given that the 6th edition Highway Capacity Manual (HCM) will be utilized to estimate capacities and level of service, it is necessary to review the current state of the HCM. The HCM guidelines for basic freeways are directed at estimating conventional vehicles from empirical data where many adjustments are made regarding geometric constraints and type of human-driven vehicular traffic. In the present-day HCM manual, CAV penetration is not considered within the mixture of vehicles in the traffic stream. As CAV technology is getting more popular and available, many analysis tools are becoming outdated or limited including the HCM [7]. The U.S. Department of Transportation Intelligent Transportation System Joint Program Office has brought up the need for a research roadmap to update the current HCM. It is their intent to have continued research done on the HCM's procedures in order to continue providing the widely and trusted analysis tools. The HCM has been widely used as a tradeoff by agencies and decision makers in the past when small level of effort, as opposed to detailed modeling and extensive simulation efforts, is required [7]. Another point that is to be considered is whether data will be readily available to run an extensive modeling process or if there is a lack of data available and a more generic approach is more viable. It is important to note

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that not all roadway modeling is created equal. The HCM provides a relatively simplified equation-based analysis that lets the user estimate capacities and level of service of highways in a relatively short period of time. Nevertheless, the HCM procedure may not be the best choice to use when confronted with highly complex scenarios.

The risk of not updating the HCM may heavily impact institutions and agencies that rely highly on the HCM to perform estimates that result in less time and resource effort [7]. In many cases, institutions are not well equipped to perform extraneous simulation modeling and lack the motivation, resources, and training to migrate to a more demanding algorithm as required by more sophisticated software. It is anticipated that CAV technology will require an extensive change in the HCM manual with respect to CAV market penetration and may be handled through adjustment factors and similar modifications [7]. Nevertheless, it is assumed that full market penetration will not happen immediately, but rather, take its time before there is full CAV saturation of the market. It is during this transition period that the HCM can still provide a useful means of estimation rather than simply be abandoned or become obsolete [7].

6th Edition of the Highway Capacity Manual – Speed, Flow Rate, & Density Concept

The 6th edition of the Highway Capacity Manual (HCM) is the main source that this study will be focusing on. The concepts and contents for basic freeway analysis are found under Volume II – Uninterrupted Flow/ Chapter 12 Basic Freeway and Multilane Highway Segments. Chapter 12 focuses strictly on uninterrupted flow, where facility access is controlled and maintained excluding any merging, diverging, and weaving within a highway segment. The analysis consists of reduction factors for lane widths, lateral clearances, and heavy vehicle interaction within the traffic stream. One of the main characteristics behind the HCM's procedure is that data is obtained from empirical

records of conventional human-driven vehicles (HVs) and not entirely theoretical. Traffic flow theory is applied as a relationship between speed vs. flow where the maximum capacity occurs stochastically between different average speeds of vehicular traffic. The main component utilized by the HCM to estimate level of service of basic freeways is density which is referred to as passenger cars per mile per lane (pc/mi/ln). Because there is a known relationship between speed, flow, and density, density can be calculated by estimating speed and flow. Therefore, the objective of the level of service analysis is to be able to calculate the density of highways with respect to a specific speed and flow set.

The definition given by the HCM for a basic freeway is "a segment that is outside the influence of any merge, diverge, or weaving segment and of any signalized intersection" [8]. The speed range that is associated with basic freeways is constrained by a lower limit of 55 miles per hour (mph) to an upper limit of 75 mph. The analysis procedure for analyzing specific flow rates is also constrained to constant demand flow rates that do not vary throughout the traffic stream. Other methods of simulation modeling should be used for such scenarios where a constant demand flow rate is not expected within a freeway segment. Traffic flow within a freeway system can be categorized as three general types: undersaturated flow, queue discharge flow, and oversaturated flow. Undersaturated flow is known by HCM as a flow rate that is unaffected by upstream or downstream bottlenecks. In contrast, oversaturated flow is the point where a queue is backed up due to a downstream bottleneck and is considered to be congested. The Queue discharge flow is described as the traffic flow that has just passed through a bottleneck and is trying to get back to the drivers' desired speed. Because the congested part of the data set is very complex in nature, the HCM categorizes all flow rates above the oversaturated limit as level of service F and further details within this area are beyond the scope of the methodology applied by the basic freeway HCM guidelines.

Figure 2- 2: Speed vs. Flow Rate HCM Empirical Data Points
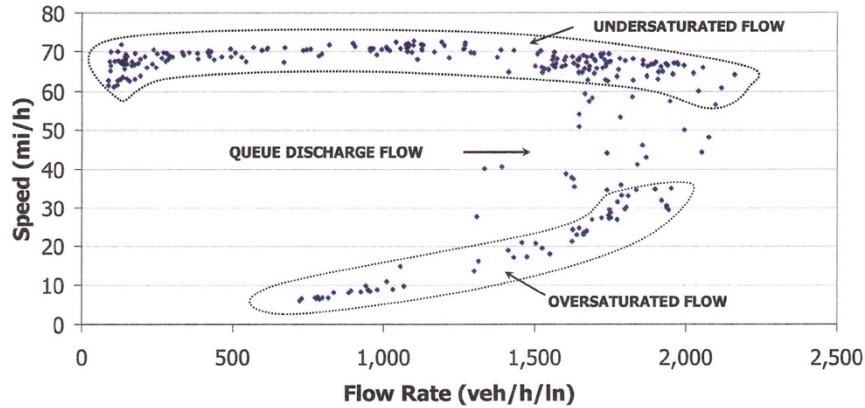
Source: California Department of Transportation, 2008.
Note: I-405, Los Angeles, California.

The key concept to take away from this is to find the maximum flow rate that is produced prior to a certain breakdown. This maximum flow rate is known as the capacity of the basic freeway and is mainly dependent upon the average speed of vehicles traveling within a specific segment. The capacity of a basic freeway is represented as passenger cars per hour per lane (pc/hr/ln). In addition, average speeds are dependent upon many geometric characteristics of the infrastructure and the type of vehicular traffic that is involved. In the HCM, lower speeds are associated with having lower capacities, while higher speeds are associated with having higher capacities.

As previously mentioned, the HCM provides a more generic way of calculating capacities and levels of service for basic freeway sections. As such, extensive preliminary data may not be required to start the process. As part of the initial process, the HCM provides generic capacities of freeways for varying average free flow speeds at ideal settings. These ideal settings are referred to as base conditions and include no heavy vehicles, driver familiarity of infrastructure, 12 ft. travel lane widths, and adequate lateral

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clearances. Interpolation between different free flow speeds is permitted and the capacities represent maximum flow rates for 15-minute intervals [8].

Table 2- 3: HCM Capacity of Basic Freeways at Different Free-Flow Speeds

FFS (mi/h)	Capacity of Basic Freeway Segments (pc/h/ln)	Capacity of Multilane Highway Segments (pc/h/ln)
75	2,400	NA
70	2,400	2,300[a]
65	2,350	2,300[a]
60	2,300	2,200
55	2,250	2,100
50	NA	2,000
45	NA	1,900

Notes: NA = not available.
[a] Capacities for multilane highways with 65- and 70-mi/h FFS are extrapolated and not based on field data.

It is important to reiterate that the flow rate values depicted above are a representation of ideal conditions that do not account for severe weather conditions, reduced lane clearance, poor lighting, and other non-ideal conditions. Calibration of specific site analysis is important and may have capacity-reducing effects that can play a part in defining the density of the highway. With respect to the national norms, basic freeways segments have been found to reach their capacity at an approximate density of 45 pc/mi/ln [8]. It is at this point that conventional vehicles are too closely spaced to adequately deal with any perturbation that produces a breakdown of the system.

Empirical data has shown that there is a common relationship between speed and flow in the analysis of basic freeways. Ultimately capacities are dependent on the free flow space mean speeds of the traffic stream. Experimental data, under base conditions, have shown that speed-flow relationships follow a common form made up of a constant speed range and a decreasing speed range. The constant range is said to extend from a flow rate of zero to a specific breaking point in the curve. Throughout this constant range, the free flow speed is estimated to remain the same without any significant change in

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speed. Once the breaking point is reached, the speed-flow curve tends to decrease in a parabolic form until it reaches a breakdown and reaches its capacity at 45 pc/mi/ln. The capacity of each run-through is directly related to the free flow speed, as is the breaking point of the curve.

Figure 2- 3: General Speed-Flow Relationship Curve of Basic Freeways
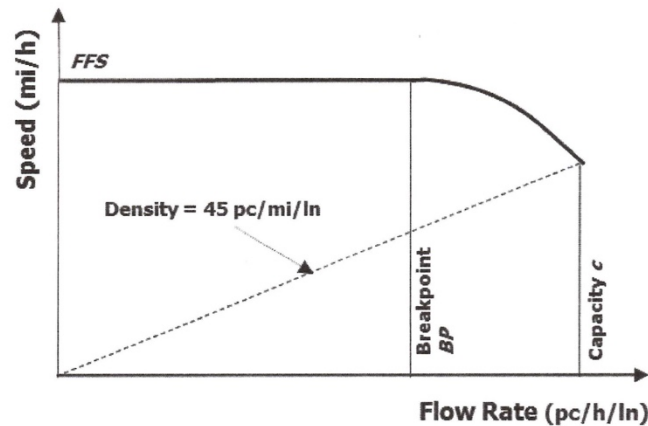
Given that there exists a common relationship between speed and flow, the HCM has developed a formula for calculating speed within the constant and parabolic ranges. These formulas use a direct relationship between flow rates and the observed breaking point with respect to the free flow speed of the traffic stream. Other parameters include adjusted free flow speed, speed adjustment factors, base segment capacity, capacity adjustment factors, adjusted segment capacity, and an exponent calibration parameter.

$$S = FFS_{adj} \qquad\qquad v_p \leq BP$$

$$S = FFS_{adj} - \frac{\left(FFS_{adj} - \frac{C_{adj}}{D_c}\right)(v_p - BP)^a}{(c_{adj} - BP)^a} \qquad\qquad BP < v_p \leq c$$

where each variable above is described by the following table:

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Table 2- 4: Model Parameters of Analytical Speed Distribution Curve

Param- eter	Definition and Units	Basic Freeway Segments
FFS	Base segment free-flow speed (mi/h)	Measured OR predicted with Equation 12-2
FFS_{adj}	Adjusted free-flow speed (mi/h)	$FFS_{adj} = FFS \times SAF$
SAF	Speed adjustment factor (decimal)	Locally calibrated OR estimated with Chapter 11; $SAF = 1.00$ for base conditions
c	Base segment capacity (pc/h/ln)	$c = 2,200 + 10(FFS - 50)$ $c \leq 2,400$ $55 \leq FFS \leq 75$
c_{adj}	Adjusted segment capacity (pc/h/ln)	$c_{adj} = c \times CAF$
CAF	Capacity adjustment factor (decimal)	Locally calibrated OR estimated with Chapter 11; $CAF = 1.00$ for base conditions
D_c	Density at capacity (pc/mi/ln)	45
BP	Breakpoint (pc/h/ln)	$BP_{adj} = [1,000 + 40 \times (75 - FFS_{adj})] \times CAF^2$
a	Exponent calibration parameter (decimal)	2.00

This study will be focusing on base conditions and will not be utilizing adjustment factors for free flow speeds or capacities. Similarly, a base free flow speed (BFFS) will be assigned as the starting free flow speed (FFS) mentioned in the above formulas. Having ideal conditions and no adjustments, a plot can be generated similar to the HCM's speed-flow generic graph for varying speeds.

***Figure 2- 4: Speed-Flow Curves of Differing Free-Flow Speeds and Level of Service E
Capacities***
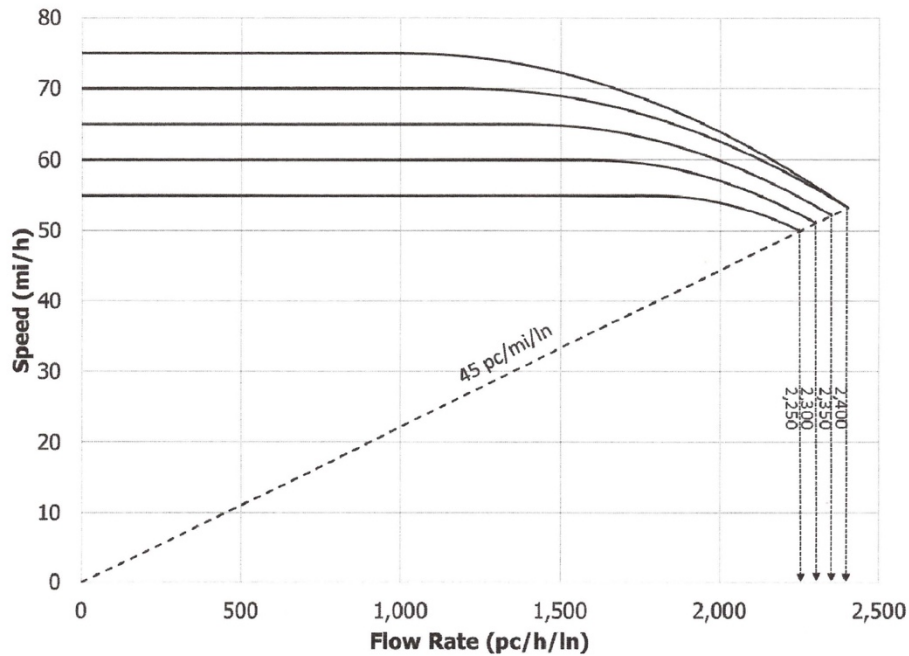
The next step of the process is to assign different levels of service to the types of
traffic streams based on differing densities. Because speeds are usually constant up to a
breaking point, it is quite difficult to assign different LOS categories based on speed
alone. Since density defines a motorist's proximity to other vehicles and is related to a
motorist's freedom to maneuver, the best parameter to describe LOS is chosen as the
density of the highway. There are six levels of service assigned by the HCM which range
from full freedom to maneuver described by level of service A to fully congested traffic
at level of service F. The details of each level of service are as follow:

Table 2- 5: Level of Service Descriptions per the 6th Edition Highway Capacity Manual

Level of Service Descriptions per the 6th Ed. Highway Capacity Manual	
LOS A	Describes fee-flow operations where vehicles are almost completely unimpeded in their ability to maneuver within the traffic stream.
LOS B	Represents reasonably free-flow operations, and FFS on the freeway is maintained. The ability to maneuver within the traffic stream is only slightly restricted, and the general level of physical and psychological comfort provided to drivers is still high. The effects of minor incidents are still easily absorbed.
LOS C	Represents flow with speeds near the FFS of the freeway. Freedom to maneuver within the traffic stream is noticeably restricted, and lane changes require more care and vigilance on the part of the driver. Minor incidents may still be absorbed, but the local deterioration in service quality will be significant. Queues may be expected to form behind any significant blockages.
LOS D	Level at which speeds begin to decline with increasing flows, with density increasing more quckly. Freedom to maneuver within the the traffic stream is seriously limited, and drivers experience reduced physical and psychological comfort levels. Even minor incidents can be expected to create queuing, because the traffic stream has little space to absorb disruptions.
LOS E	Level at which operation is at or near capacity. Operations on the freeway at this level are highly volatile because there is virtually no usable gaps within the traffic stream, leaving little room to maneuver within the traffic stream. Any disruption to the traffic stream, such as vehicles entering from a ramp or an access point or a vehicle changing lanes, can establish a disruption wave that propagates throughout the upstream traffic stream. Toward the upper boundary of LOS E, the traffic stream has no ability to dissipate even the most minor disruption, and any incident can be expected to produce a serious breakdown and substantial queuing. The physical and psychological comfort afforded to drivers is poor.
LOS F	Represents unstable flow. Such conditions exist within queues forming behind bottlenecks.

For the purposes of the HCM, levels of service beyond F are considered fully congested with no further information required. LOS F is associated with oversaturated flow that occurs when the actual or forecasted demand (volume) is greater that the capacity of the highway ($^v/_c > 1.0$). This phenomenon is associated with unstable flow where breakdowns are present and no clear correlation can be found between encountered traffic flow rates and speeds. Therefore, HCM's analysis only encompasses speed-flow relationship from LOS A to E. The only measure that is required to define a basic freeway's level of service is density in units of passenger cars per hour per lane

(pc/hr/ln). The criteria for evaluating LOS of basic freeways with respect to density is shown below.

Table 2- 6: HCM Level of Service Density Thresholds

LOS	Density (pc/mi/ln)
A	≤11
B	>11–18
C	>18–26
D	>26–35
E	>35–45
F	Demand exceeds capacity OR density > 45

As previously noted, the density of a basic freeway is directly related to the speeds and flow rates of the traffic stream by the relationship

$$q = uk \text{ or } k = {^q\!/_u}$$

where q is the flow rate of the traffic stream in passenger cars per hour per lane (pc/hr/ln), u is the speed in miles per hour (mph), and k is the density in passenger cars per hour per lane (pc/mile/ln). If the speed and flow rate of a basic freeway is known, the density of the freeway can be calculated using these two parameters. The level of service for each scenario is based on the interval between the lower limit (not inclusive) and the upper limit of the density region being calculated. The relationship of speed, flow rate, density, and level of service is shown below.

Figure 2- 5: HCM Speed-Flow Curves with Level of Service Thresholds and Regions Depicted
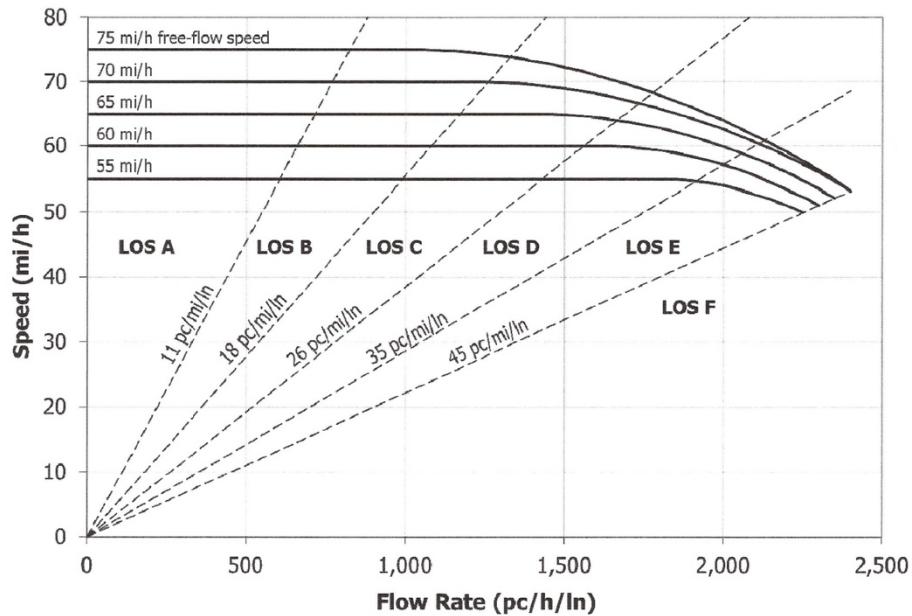
6th Edition of the Highway Capacity Manual – Computational Procedure

The calculations required for a typical analysis of a basic freeway section consist of first specifying a number of parameters including the "demand volume, number and width of lanes, lateral clearance, total ramp or access point density, percent of heavy vehicles, peak hour factor, type of terrain, driver population, speed, and capacity adjustment factors" [8]. Most of the parameters specified are used to reduce an ideal starting condition representing a perfect scenario where there is no impeding traffic congestion, ideal lane width and lateral clearances, no impacts from upstream and downstream ramps, no free flow speed or capacity constrictions, a homogeneous passenger-car traffic stream, no variation in traffic flow with respect to time, and an ideal type of terrain with limited upgrades. Each parameter has its own reduction factors and is clearly defined by specific formulas.

The free flow speed is the mean speed of passenger cars that is expected to be experienced at low levels of flow, usually around 500 pc/hr/ln. This information can be directly obtained through field measurements or estimated using the HCM's theoretical approach. Given that field measurements are not always readily available, the HCM provides a practical way to calculate the free flow speed with respect to the geometry of an existing freeway.

$$FFS = BFFS - f_{LW} - f_{RLC} - 3.22 \times TRD^{0.84}$$

where

FFS = free-flow speed of the basic freeway segment (mi/h);

$BFFS$ = base FFS for the basic freeway segment (mi/h);

f_{LW} = adjustment for lane width, from Exhibit 12-20 (mi/h);

f_{RLC} = adjustment for right-side lateral clearance, from Exhibit 12-21 (mi/h); and

TRD = total ramp density (ramps/mi).

The base free flow speed for basic freeways ranges from 55 to 75 miles per hour and can be chosen at the judgement of the user. Lane width adjustment reductions are applied from a base condition of 12ft. travel lanes. Lower widths are intended to reduce the free flow speed, while greater lane widths have no effect on the analysis.

Table 2- 7: Lane Width Adjustment Reduction Factors

Average Lane Width (ft)	Reduction in FFS, f_{LW} (mi/h)
≥12	0.0
≥11–12	1.9
≥10–11	6.6

Similarly, a 6 ft. base condition for right side lateral clearance is used as a basis to apply a reduction to the free flow speed with respect to right-side clearance and number of lanes per direction. Interpolation of right-side lateral clearance is allowed for non-integer values.

Table 2- 8: Right-Side Lateral Clearance Adjustment Factors

Right-Side Lateral Clearance (ft)	Lanes in One Direction			
	2	3	4	≥5
≥6	0.0	0.0	0.0	0.0
5	0.6	0.4	0.2	0.1
4	1.2	0.8	0.4	0.2
3	1.8	1.2	0.6	0.3
2	2.4	1.6	0.8	0.4
1	3.0	2.0	1.0	0.5
0	3.6	2.4	1.2	0.6

Note: Interpolate for noninteger values of right-side lateral clearance.

The calculation for free flow speed also includes an impact of total ramp density. This measure is defined as the number of ramps (in one direction) that are found 3 miles upstream and downstream of the midpoint of the freeway segment that is being analyzed. By dividing the number of ramps by the total length of segment being analyzed (6 miles), a density of ramps per mile is obtained. After obtaining each individual parameter reduction, the free flow speed of the basic freeway can be calculated by the previously mentioned formula.

In addition to the reduction parameters explained above, the free flow speed can be further adjusted by applying a speed adjustment factor (SAF) directly to the previously obtained FFS value. Reasons for applying a SAF include adjustments for weather concerns, work zones, poor pavement, sun glare, and/ or unfamiliar drivers. The HCM provides guidance on how to choose specific SAF values if needed. An adjusted free flow speed can be calculated with the following equation:

$$FFS_{adj} = FFS \times SAF$$

The procedure for calculating the starting capacity of a basic freeway segment is based on empirical data and can be calculated by the formula shown below. The base capacity is directly dependent on the adjusted free flow speed of the traffic stream.

$$c \text{ (basic freeway segment)} = 2{,}200 + 10 \times (FFS_{adj} - 50)$$

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A similar procedure to that of the SAF with FFS can be applied to the base capacity condition of basic freeways. This adjustment is known as the capacity adjustment factor (CAF) and is unitless. Some reasons for applying an adjustment to the base capacity include driver unfamiliarity, poor sight distances on vertical or horizontal curves, and/ or lane drops between two basic freeway segments. The adjusted capacity of a freeway segment can be calculated using the following equation:

$$c_{adj} = c \times CAF$$

where

c_{adj} = adjusted capacity of segment (pc/h),

c = base capacity of segment (pc/h), and

CAF = capacity adjustment factor (unitless).

The next step in the process is to calculate an equivalent demand flow rate of the system based on demand volumes, variation of hourly traffic flow, proportion of heavy to passenger car vehicles, and terrain grades. The HCM uses an equivalent passenger car demand formula to estimate flow rates in equivalent passenger cars per hour. The peak hour factor (PHF) is a measure of the hourly variation of flow that is usually consistent with changes in 15-minute increments. Typical PHF values range between 0.85 to 0.98. It is also typical to use higher values of PHF for urban settings and lower values for rural settings.

$$v_p = \frac{V}{PHF \times N \times f_{HV}}$$

where

v_p = demand flow rate under equivalent base conditions (pc/h/ln),

V = demand volume under prevailing conditions (veh/h),

PHF = peak hour factor (decimal),

N = number of lanes in analysis direction (ln), and

f_{HV} = adjustment factor for presence of heavy vehicles (decimal).

Because the speed-flow relationship produces a curve that is consistent with flow rates in passenger car equivalencies, an adjustment factor for presence of heavy vehicles in the traffic stream must be computed. This is done by finding the proportion of single unit trucks (SUTs) and tractor trailers (TTs) to that of passenger cars in the traffic stream. Both vehicle types are added as a single percentage value in decimal form. A passenger car equivalency factor (E_T) is applied with respect to the presence of heavy vehicles by considering the terrain of the segment of the basic freeway. When considering general terrain of segments, the user can choose from level or rolling terrains that are described as having short grades of no more than 2% and combination of grades that cause heavy vehicles to reduce their speed below that of passenger cars but do not cause heavy vehicles to operate at crawl speed, respectively. The guidelines also provide passenger car equivalency factor tables based on specific upgrades of terrain where different mixes of SUTs and TTs exist. The two methods can be utilized as appropriate to the type of terrain being analyzed.

Table 2- 9: Passenger Car Equivalency Values for General Terrain Segments

Passenger Car Equivalent	Terrain Type	
	Level	Rolling
E_T	2.0	3.0

Once each parameter has been calculated accordingly, the formula stated below can be used to find the heavy vehicle adjustment factor.

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1)}$$

where

f_{HV} = heavy vehicle adjustment factor (decimal),

P_T = proportion of SUTs and TTs in traffic stream (decimal), and

E_T = passenger car equivalent of one heavy vehicle in the traffic stream (PCEs).

After calculating the heavy vehicle factor, the equivalent passenger car demand formula can be used to calculate the equivalent passenger car demand volume.

$$v_p = \frac{V}{PHF \times N \times f_{HV}}$$

With the demand volume converted to equivalent passenger cars per hour per lane, the volume can be compared with base capacity conditions previously computed. If the computed equivalent demand volume is above the base capacity conditions, then the freeway is said to experience a LOS of F and no further analysis is required. Otherwise, the specific density of the basic freeway is calculated by dividing the demand equivalent passenger car volume by the mean speed previously calculated. This process is represented by the following equation:

$$D = \frac{v_P}{S}$$

The last step in the process is to compare the density value obtained in passenger cars per hour per lane (pc/hr/ln) to the speed vs. flow rate graph and LOS limits.

PTV VISSIM – Microscopic Traffic Flow Simulation Software

Traffic analysis has become omnipresent in the constant urbanization of communities throughout the world. Because of an ever-growing population, a rise in vehicular traffic is present in interstate highways, central business district grid networks, and local roadway systems. Many traffic software simulators have now been developed to assist the engineering community in recognizing and evaluating traffic patterns that can lead to congestion and queue spillback. The practice of traffic simulation can include

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macroscopic, mesoscopic, and microscopic simulation, depending on the objective and parameters being used. Macroscopic simulators are used to evaluate platoons of vehicles at the highest level of aggregation, while microsimulation is based on modeling individual vehicles [9]. Mesoscopic simulation has recently been included into the discipline and includes procedures of utilizing macro- and microscopic simulation to achieve detailed microscopic results [9].

VISSIM microsimulation software from PTV Group is one of the traffic analysis software tools that is widely accepted by the engineering community in the U.S. and abroad [9]. It includes a number of parameters that can be modified to fit the specific environment of a network and utilizes microsimulation procedures in order to output a number of analytical results. As such, the software is especially applicable to the focus of this study and can be utilized to run an analysis of freeways with controlled characteristics.

VISSIM utilizes the Wiedemann 99 car-following model for freeway analysis as its main driver behavior evaluation tool [10]. This driver behavior feature can be edited similar to other vehicle characteristic variables within the software. The Wiedemann 99 car following model includes 10 specific modifiable driver behavior parameters (CC0 to CC9) that can be adjusted accordingly to simulate intrinsic features of drivers and capabilities of the specific vehicles being driven. One particular parameter of special interest to this study is the CC1 parameter found within the driver behavior model that represents the time headways between vehicles. As previously mentioned, the time headway between successive vehicles is of vital concern when it comes to capacities of highways. By changing this parameter, the volume outputs can be increased with respect to desired speed distributions.

As part of the initialization process of simulation, the software must be calibrated to a specific network whenever possible. Nicholas E. Lownes and Randy B. Machemehl performed a sensitivity analysis where the 10 driver behavior parameters were modified and calibrated to an urban freeway located in Plano and Richardson, Texas [9]. A sensitivity analysis was carried out to find which parameters had a significant impact, if at all, on capacities of highways. The values of these parameters were also calibrated with respect to empirical data gathered from the subject project. The following table shows the summary of their results with respect to the project analyzed.

Table 2- 10: Lownes & Machemehl Summary of Capacity Sensitivity to Parameters Based Upon the VISSIM Wiedemann 99 Car-Following Model

Parameter	Level	Value	Resulting Capacity (vph)	SE	t-Statistic	Degrees of Freedom	Critical t-Value	Significantly Different from Value at Calibration?
CC0	Low	2.0	8062	82.5	1.35	17	2.567	N
Default = 4.92	Calibrated	4.8	8020	93.8	—	—	—	—
	Medium	5.0	8054	98.4	0.96	14	2.624	N
	High	10.0	7788	87.2	7.15	16	2.583	Y
CC1	Low	0.50	8074	99.2	1.50	14	2.624	N
Default = 0.90	Calibrated	0.85	8020	93.8	—	—	—	—
	Medium	1.00	7888	82.7	4.24	17	2.567	Y
	High	1.50	6818	53.8	49.79	27	2.473	Y
CC2	Low	5.00	8119	106.0	2.63	14	2.624	Y
Default = 13.12	Medium	10.00	8112	76.9	3.08	18	2.552	Y
	Calibrated	13.12	8020	93.8	—	—	—	—
	High	20.00	7803	89.2	6.59	16	2.583	Y
CC3	Low	−4	7976	79.4	1.44	18	2.552	N
Default = −8.00	Calibrated	−8	8020	93.8	—	—	—	—
	Medium	−10	7980	94.4	1.17	15	2.602	N
	High	−15	8030	87.5	0.31	16	2.583	N
CC4 and CC5	Low	0.1	8090	103	1.89	14	2.624	N
Default = ±0.35	Medium	0.5	8082	91.6	1.85	15	2.602	N
	Calibrated	1.0	8020	93.8	—	—	—	—
	High	2.0	7555	64.4	17.47	22	2.508	Y
CC6	Low	2.00	7989	99	0.86	14	2.624	N
Default = 11.44	Medium	8.00	7943	91.1	2.29	15	2.602	N
	Calibrated	11.44	8020	93.8	—	—	—	—
	High	20.00	8033	91.4	0.37	15	2.602	N
CC7	Low	0.50	8035	83.4	0.47	17	2.567	N
Default = 0.82	Calibrated	0.82	8020	93.8	—	—	—	—
	Medium	1.00	7990	89.2	0.90	16	2.583	N
	High	1.50	8053	92.3	0.98	15	2.602	N
CC8	Low	6.4	7638	95.3	11.03	15	2.602	Y
Default = 11.48	Medium	8.0	7925	94.3	2.75	15	2.602	Y
	High	10.0	8005	87.3	0.47	16	2.583	N
	Calibrated	11.5	8020	93.8	—	—	—	—
CC9	Low	2.10	8024	96.3	0.12	15	2.602	N
Default = 4.92	Medium	4.50	8002	83.5	0.56	17	2.567	N
	Calibrated	4.92	8020	93.8	—	—	—	—
	High	7.50	8034	84.1	0.45	17	2.567	N
Look-back distance	Low	50	8316	74.6	10.14	19	2.539	Y
Default = 656.2	Calibrated	300	8020	93.8	—	—	—	—
	Medium	5000	8193	75.6	5.87	19	2.539	Y
	High	1000	8307	67.3	10.50	21	2.518	Y

By running a t-test statistical analysis, they were able to show that parameters CC3, CC6, CC7, and CC9 were of insignificant consequence when it comes to capacity changes. In contrast, the CC1 parameter representing the time headways was one of the more critical contributors to changes in capacity, as well as parameters CC0, CC2, CC4, CC5, and CC8. The calibration of the specific values corresponding to CC0, CC1, CC2,

CC4, CC5, and CC8 is of direct consequence to the capacity of highways and must be accurately calibrated prior to running any type of simulation.

Because VISSIM allows the user to readily change many parameters dealing with vehicles characteristics, desired speed, traffic stream composition, geometry of infrastructure, volume inputs, and intrinsic driver behaviors, the software was an important tool that could be employed to collect data outputs for the analysis.

Pipes Car-Following Model / Alternative Process

The VISSIM microsimulation software is quite advantageous, nevertheless, it presently lacks the ability of analyzing autonomous and connected vehicles (CAVs) directly within the program. There are alternative software micro-simulators that exist that utilize different car-following models than that of the VISSIM software. One alternative software that is widely recognized in the U.S. is the CORSIM micro-simulator developed by the US Federal Highway Administration (FHWA) and is composed of two models dealing with urban streets (NETSIM) and freeways (FRESIM) [11]. Due to its production by way of a public agency, the software is readily available to many users and has been adopted in different territories within the U.S.

CORSIM has been steadily developed by FHWA since the 1970s, and as such, much literature on its car-following model is available. The FRESIM model within the CORSIM microstimulator utilizes the Pipes car-following model derived from the Pitt car-following model developed by the University of Pittsburgh [12]. The model incorporates the headway distance between a preceding and following vehicle with a driver's sensitivity to change. Under steady state conditions where the preceding and following vehicles experience an equal constant speed, the pipes car-following model can be applied. A constraint is also set as a maximum under the presumption that a driver will

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not rise above his or her desired speed. The following equation describes the Pipes car-following model:

$$u = min\left(\frac{h - h_j}{c_3}, u_f\right)$$

where u is the speed of the following vehicle at the next time step in feet per second, h is the distance headway between the front bumper of the preceding vehicle and the front bumper of the following vehicle in feet, h_j is the distance headway when vehicles are completely stopped in queue in feet, c_3 is the driver sensitivity factor in seconds, and u_f is the roadway free flow speed in feet per second. The h_j term can be further represented as being the inverse of the jam density of the traffic stream as follows:

$$h_j = \frac{1}{k_j} \times 5280$$

Where k_j is the jam density in vehicles per mile. Furthermore, Rakha and Crowther [11] demonstrated that the c_3 driver sensitivity parameter could be categorized as follows:

$$c_3 = 3600 \times \left(\frac{1}{q_c} - \frac{1}{k_j \times u_f}\right)$$

where q_c is the average lane capacity in vehicles per hour, and c_3 is the driver sensitivity factor in seconds.

Presently, there are no microstimulators commercially available that can simulate the effect of autonomous and connected vehicles (CAVs) within a mixed traffic stream. The CORSIM software provides a valid car-following model that can be applied to represent the intrinsic characteristics of drivers within a traffic stream. By allowing the driver sensitivity parameter to represent the perception-reaction (PIJR) time of a human driver and setting it equal to the time headway, a model can be created to represent characteristics depicting CAVs.

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Summary

With innovation comes change, and CAV inclusion in highways is no exception. The new technology is expected to enter the traffic system in the near future with early estimates starting from 20 to 40 years from now (2018) [1]. Though public opinion surveys may show a lack of enthusiasm when it comes to upgrading to level 4 autonomy, there is a push by the federal government to encourage connectivity between car manufacturers to allow the new technology to thrive [1]. Traffic networks and highways will surely be affected by the change in vehicular mode from conventional vehicles to CAVs, and it is expected that highway capacities will be directly affected by CAVs' ability to produce shorter time headways. Though reduction of time headways are expected, there is much uncertainty in the exact amount of time headway that CAVs will be able to manage. Due to the nature of public perception and economics, the market penetration of CAVs is not expected to reach complete adoption immediately, but rather disseminated gradually. It is, therefore, necessary to analyze traffic streams at different market penetrations and at varying time headways between successive vehicles.

The 6th ed. Highway Capacity Manual (HCM) has been one of the main sources utilized when planning to estimate capacities of freeways. This specific piece of literature utilizes a simplified equation-based process by which engineers are able to estimate freeway capacities with little effort. Nonetheless, the concepts behind the HCM are constrained to conventional human-driven behavior and empirical data that does not represent intrinsic behavior that may be expected from CAVs. An HCM roadway map has been laid out in order to attempt revisions of the manual to adopt CAV characteristics [7].

VISSIM microsimulation software is a powerful tool that can help model the attributes expected from CAVs. This microstimulator provides the means to decrease

time headways within its driver behavior car following model to represent faster response times and denser groups. The software will be explored in order to find if CAV characteristics can be represented accurately through this process.

An alternative approach will be presented where the method includes modifications of intrinsic driver behavior with the help of the Pipes car-following model. Randomness of human-driven PIJR times, market penetration of CAVs, and CAV time headways will be of principal interest.

3.0 Methodology

The objective of this study was to calculate and evaluate different flow rates produced by CAV driver behavior characteristics with respect to varying free flow speeds (FFS). One specific measure that this study focused on was the capacity of freeways at particular base free flow speeds (BFFS). The capacity of a freeway was represented by the point where a breakdown was observed that corresponded to "a sudden drop in speed of at least 25% below the free flow speed (FFS) for a sustained period of at least 15 minutes" [8]. By obtaining a flow rate with respect to a corresponding speed, the density was calculated by dividing the flow rate by the speed. Therefore, the first objective of this study was to find the corresponding speed to each flow rate with respect to the free flow speed desired.

3.1 Highway Capacity Manual Procedure

The first step in the process was to replicate the procedure set by the 6th edition Highway Capacity Manual (HCM) found under Volume II – Uninterrupted Flow/ Chapter 12 Basic Freeway and Multilane Highway Segments to estimate conventional vehicle capacities. Certain assumptions were taken in order to reproduce the speed vs flow plot introduced by the HCM under the Level of Service criteria.

The curves that are presented in the HCM speed vs. flow plot were comprised of attributes at ideal conditions under varying free flow speeds in basic freeways segments. These ideal conditions correspond to assumptions characterized by standard engineering practices where the geometry of a roadway, access control, traffic stream composition, variation in traffic flow, and terrain slope are limited to perfect conditions.

3.1.1 Free Flow Speed Calculation

The free flow speed corresponds to the mean speed that passenger cars travel under low volume conditions, typically at around 500 passenger cars per hour per lane. FFS is calculated by the following equation:

$$FFS = BFFS - f_{LW} - f_{RLC} - 3.22 \times TRD^{0.84}$$

Where FFS was the free flow speed of the basic freeway (mi/hr), $BFFS$ is the base free flow speed of the basic freeway segment (mi/hr), f_{LW} is an adjustment for lane width (mi/hr), f_{RLC} is an adjustment for right-side lateral clearance, and TRD is the total ramp density of the basic freeway segment. Ideal conditions were achieved by allowing each parameter equal to base conditions that consisted of the following results:

- Average travel lane width = 12 ft
- Right-side lateral clearance ≥ 6 ft
- Total ramp density = 0 ramps/ mile

Under the base conditions applied above, all adjustments to the BFFS were eliminated and the BFFS was simply equal to the FFS. Therefore, the FFS was taken as the desired free flow speed in the analysis procedure. The desired FFSs considered ranged from 55 mph to 75 mph in increments of 5 mph. The different free flow speeds were as follow:

- FFS = 55 mi/hr
- FFS = 60 mi/hr
- FFS = 65 mi/hr
- FFS = 70 mi/hr
- FFS = 75 mi/hr

3.1.2 Speed and Capacity Estimates and Adjustments

The 6th ed. HCM provides guidance in calculating the base segment capacity for a general basic freeway segment. This base segment capacity is defined by the following formula:

$$c(basic\ freeway\ segment) = 2{,}200 + 10 \times (FFS_{adj} - 50)$$

where c is the base capacity of a freeway segment in vehicles per hour per lane and FFS_{adj} is the adjusted free flow speed in miles per hour. The FFS_{adj} can be found by applying a speed adjustment factor, SAF, which defaults to 1.0 when no adjustments are applied to the free flow speed. Under base conditions, no adjustments were made to the FFS and the FFS_{adj} value was equal to the FFS previously calculated. Substituting the different FFSs obtained in the previous section, the respective base capacities for each FFS were as follow:

Table 3- 1: Base Capacity Values at Different Free-Flow Speeds

Free Flow Speed FFS [mi/hr]	Base Capacity c [veh/hr/ln]
55	2250
60	2300
65	2350
70	2400
75	2400

The base capacity of a freeway segment can also be adjusted with a capacity adjustment factor, CAF. Nevertheless, under ideal conditions, the CAF factor is equal to 1.0 and the adjusted capacity value, c_{adj}, simply equals the base capacity value c.

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3.1.3 Adjusting Demand Volume

The speed-flow curves represent volumes in equivalent passenger cars per hour per lane, and therefore, demand in vehicles per hour must be adjusted accordingly. The HCM provides a formula to make these adjustments and is described as follows:

$$v_p = \frac{V}{PHF \times N \times f_{HV}}$$

Where v_p is the demand flow rate under equivalent base conditions in passenger cars per hour per lane, V is the demand volume under prevailing conditions in vehicles per hour, PHF is the peak hour factor showing variation in traffic flow within an hour in decimal form, N is the number of lanes present in the analysis direction, and f_{HV} is an adjustment factor for the presence of heavy vehicles in decimal form.

For simplicity purposes, the study was evaluated as a single lane freeway segment with no variation in traffic flow within an hour, and the vehicular composition consisted of only passenger cars with no heavy vehicle contribution. Similarly, ideal results were obtained by applying the assumptions mentioned above. By setting PHF to its default value of 1.0, N to one lane, and f_{HV} equal to 1.0, the equivalent passenger car volume was set to the demand volume originally represented in vehicles per hour. Therefore, the demand flow rate under equivalent base conditions in passenger cars per hour per lane, v_p, was the same as any value presented in vehicles per hour in ideal conditions.

For this particular study, equivalent flow rates from 0 to each respective FFS's capacity were evaluated at 100 pc/hr/ln intervals.

3.1.4 Estimating Speed Values

Returning to the speed vs flow plot, the plot experiences two common tendencies described as the constant speed range and the decreasing speed range. The two tendencies are separated by a breaking point denoted as BP in vehicles per hour per lane. The breaking point for different free flow speeds under ideal conditions can be found by the following equation:

$$BP = [1000 + 40 \times (75 - FFS)]$$

Substituting the FFSs from the analysis into the above equation, the BP values were calculated and shown as follows:

Table 3- 2: Speed-Flow Curve Breaking Points for Different Free-Flow Speeds

Free Flow Speed FFS [mi/hr]	Breaking Point BP [veh/hr/ln]
55	1800
60	1600
65	1400
70	1200
75	1000

Lastly, speed values can be calculated by a piecewise function that depends on the break point values, BP, with respect to the FFSs. The piecewise function is described as follows:

$$S = FFS_{adj} \qquad\qquad\qquad v_p \leq BP$$

$$S = FFS_{adj} - \frac{\left(FFS_{adj} - \frac{c_{adj}}{D_c}\right)(v_p - BP)^a}{(c_{adj} - BP)^a} \qquad\qquad BP < v_p \leq c$$

where, under ideal conditions, S is equal to the mean speed of the traffic stream under base conditions in miles per hour, FFS_{adj} is the adjusted free flow speed that is equal to the desired FFS in miles per hour, c_{adj} is the adjusted capacity that is equal to the base capacity c in passenger cars per hour per lane, D_c is the density at capacity of freeway segments that is equal to 45 passenger cars per mile per lane, v_p is the equivalent demand flow rate in passenger cars per mile per lane, and a is an exponent calibration parameter equal to 2.0 in decimal form.

Evaluating each different FFS separately, the c_{adj}, D_c, and BP variables became constants and different mean speeds were obtained with respect to varying equivalent demand flow rates. A summary of all variables obtained for each FFS is shown in the table below:

Table 3- 3: Summary of Variables Applied to the HCM Speed Distribution Piecewise Function

Free Flow Speed FFS [mi/hr]	Speed Adjustment Factor SAF [decimal]	Adjusted Free Flow Speed FFS_adj [mi/hr]	Base Capacity c [pc/hr/ln]	Capacity Adjustment Factor CAF [decimal]	Adjusted Capacity c_adj [pc/hr/ln]	Break Point BP [pc/hr/ln]	Exponent Calibration Parameter a [decimal]	Density at Capacity D_c [pc/mi/ln]
55	1	55	2250	1	2250	1800	2	45
60	1	60	2300	1	2300	1600	2	45
65	1	65	2350	1	2350	1400	2	45
70	1	70	2400	1	2400	1200	2	45
75	1	75	2400	1	2400	1000	2	45

3.1.5 Speed vs. Flow Plot and Level of Service

A curve of speed distribution was then reproduced by creating a table of speed vs. flow rates by keeping a constant interval between demand flow rate values. By substituting the variables shown above into the speed piecewise function and evaluating

each *FFS*s separately, tables of speed vs. flow were generated and were consistent with the following values at ideal conditions:

**Figure 3- 1: HCM Speed-Flow Curves with Level of Service Thresholds and Regions Depicted**
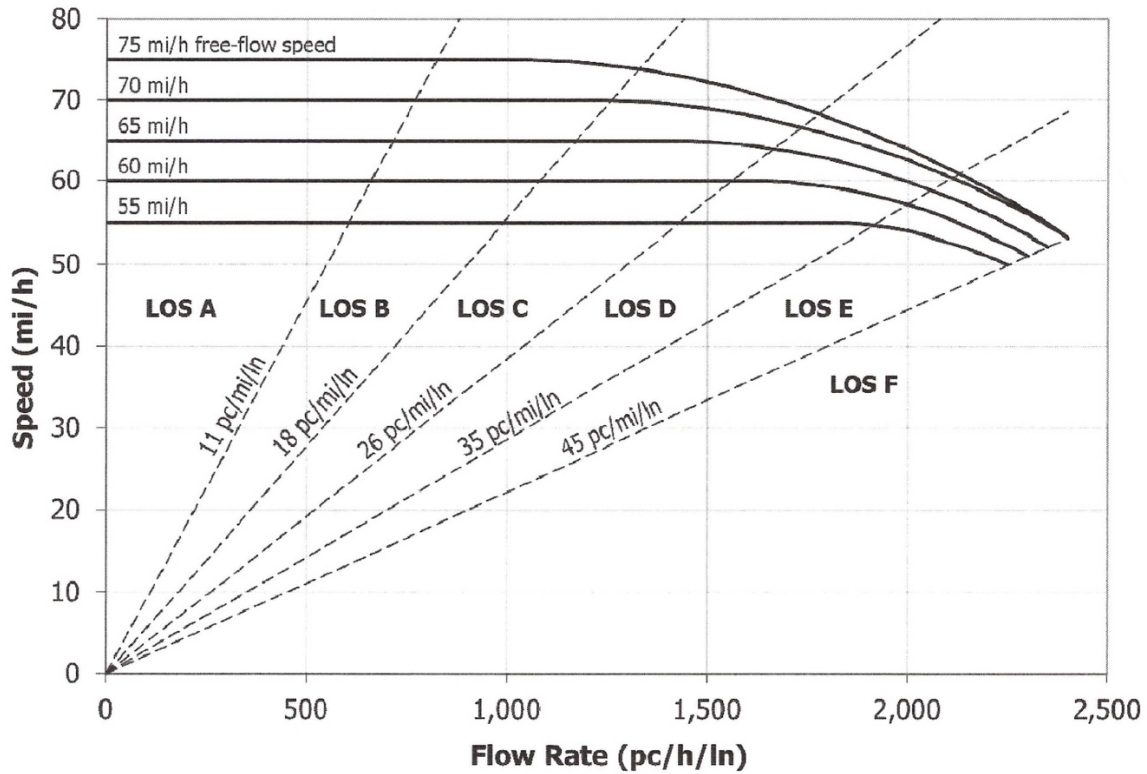
The initial speed vs. flow plot provided by the HCM with free flow speed curves was finally obtained by plotting the tables shown above. The diagonal lines representing the different Level of Service (LOS) limits were obtained by dividing the upper LOS limit of each region by the demand flow rate intervals being analyzed. The upper LOS limits are provided by the HCM and are as follow:

Table 3- 4: HCM Level of Service Density Thresholds

LOS	Density (pc/mi/ln)
A	≤11
B	>11–18
C	>18–26
D	>26–35
E	>35–45
F	Demand exceeds capacity OR density > 45

The following figure is a reproduction of the HCM's speed vs. flow plot of basic freeways at ideal conditions:

Figure 3- 2: Reproduction of the HCM Basic Freeway Speed vs. Flow Plot Based on Ideal Conditions
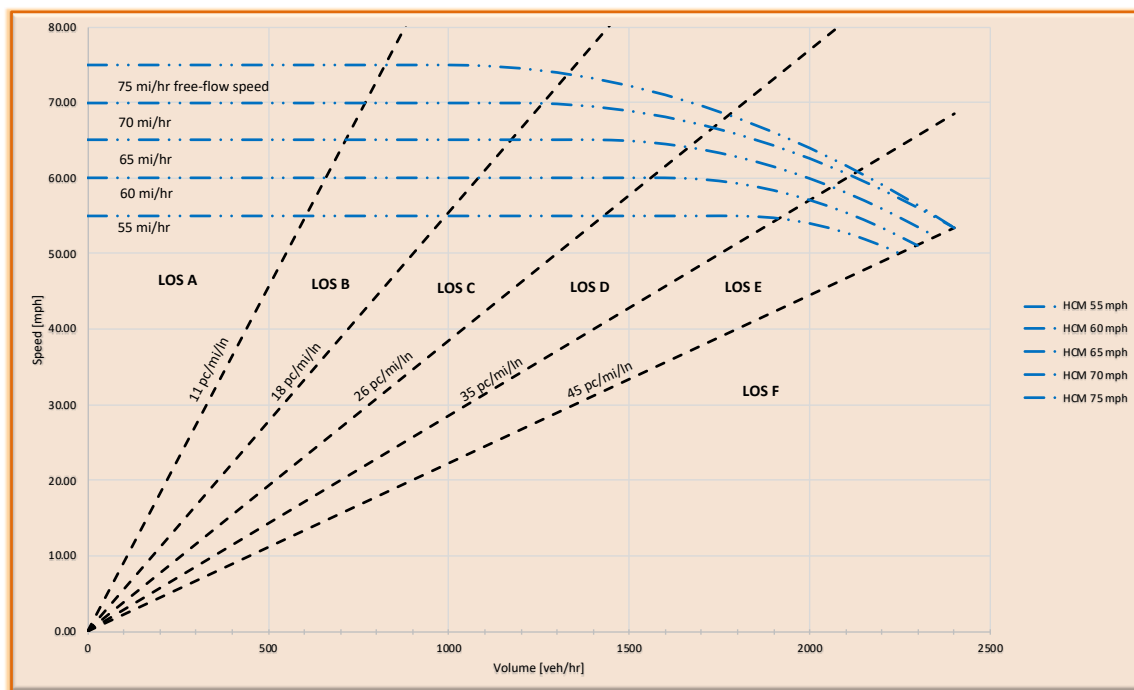
This depiction of speed vs flow served as the basis of comparison between empirical data obtained from human-driving characteristics and CAV capacities obtained by a theoretical approach.

3.2 VISSIM Microsimulation Software

The VISSIM microstimulator software was used as the first attempt to represent CAV behavior within a simple basic freeway segment model. The specific version of the micro simulator utilized was PTV VISSIM 10.00 from PTV Group, classroom version. As previously stated before, the VISSIM software is equipped with the Wiedemann 99 car-following model that allows the user to modify driver intrinsic behavior that can have a significant influence on freeway capacity values. The objective of this endeavor was to try and show a correlation between empirical data obtained from the HCM and results obtained from the VISSIM software algorithm. Once a basis for capacities is obtained, the model can be calibrated accordingly to represent as much as possible the empirical/ theoretical data provided by the HCM.

3.2.1 Software and Network Settings

Before starting to build a freeway model, the network settings were first setup appropriately to output the desired units on which the model will be based. Likewise, simulation parameters were initialized to represent the desired time of simulation and parameters required to output the desired results.

The first step in the process was to setup the working units as imperial units under the Base Data menu and network settings. The settings for length, speed, and acceleration were as follow:

Figure 3- 3: Setting Imperial Units Within Network Settings
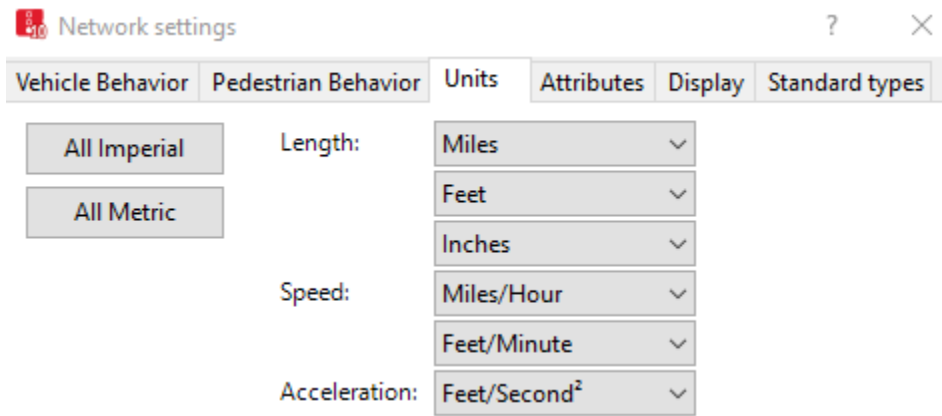
The simulation parameters for the simulation were setup up under the simulation menu and simulation parameters dialog box. An hour-long simulation period equal to 3600 seconds was chosen as the simulation time for the model with 10-time steps per simulation second. The number of 10-time steps per simulation second was found to be an adequate resolution for the model that would be run. The initial random seed was left at default value, being that this would not cause any disruption in the simulation. In addition, the number of runs were increased to two runs per simulation in order to obtain and observe variability in the results obtained produced by VISSIM's algorithm. The parameters for simulation speed were run at maximum flow or capacity to obtain results quicker. A summary of the simulation parameters chosen are as follow:

Figure 3- 4: Pre-setting Simulation Parameters

3.2.2 Geometry of Basic Freeway Model

For purposes of matching the HCM's empirical approach, the geometry of the basic freeway was chosen to represent ideal conditions based on guidance from the HCM. As such, a 12-foot wide, single lane roadway extending for approximately one mile (5280 ft) was chosen as the roadway geometry for the model to be analyzed. The behavior type was chosen as "Freeway (free lane selection)" to represent the behavior of drivers in a

freeway setting and to utilize the Wiedemann 99 car-following model intended for freeway behavior as opposed to urban roadways.

Nonetheless, after performing some initial runs of the model, the outputs displayed non-reducing speeds with respect to increasing volumes. The logic behind these results comes from the idea that drivers will follow their desired speed unless there exists a perturbance, usually initiated from a merge, diverge, or lane change. Due to this constraint, the geometry of the roadway was changed to depict two 12-foot wide lanes running for 1 mile at the upstream end that converged to a single 12-foot wide lane running for 1 mile at the downstream end. A 500 ft. connector representing the merge was added to the model representing a typical merge length in a freeway system.

Figure 3- 5: Upstream Geometric Link Details

Count: 2	Index	Width	Blocked vehicle classes	Display type	No lane change left - all	No lane change right - all	No lane change left -	No lane change right -
1	1	12.00			☐	☑		
2	2	12.00			☑	☐		

Figure 3- 6: Downstream Geometric Link Details

Figure 3- 7: Merge Section Geometry Details

3.2.3 Desired Speed Distribution

The next step in the process was to build desired speed distributions for the specific free flow speeds being analyzed. The desired speed distributions were found under the Base Data menu, distributions, and desired speeds. Within the provided software's desired speed distribution list dialog box, five distributions were created to

represent the free flow speeds ranging from 55 to 75 miles per hour in 5 mile per hour increments. A 10% distribution was applied to the lower and upper bound of each free flow speed to represent low variability of desired speed. Each free flow speed distribution was bounded by a 5 mile per hour deviation from the average free flow speed chosen. The following figures show an example of the speed distribution for the 55 mile per hour free flow speed and a table summarizing all speed distribution bounds for the different free flow speeds being analyzed.

Figure 3- 8: Desired Speed Distribution Plot at 55 mph Free-Flow Speed
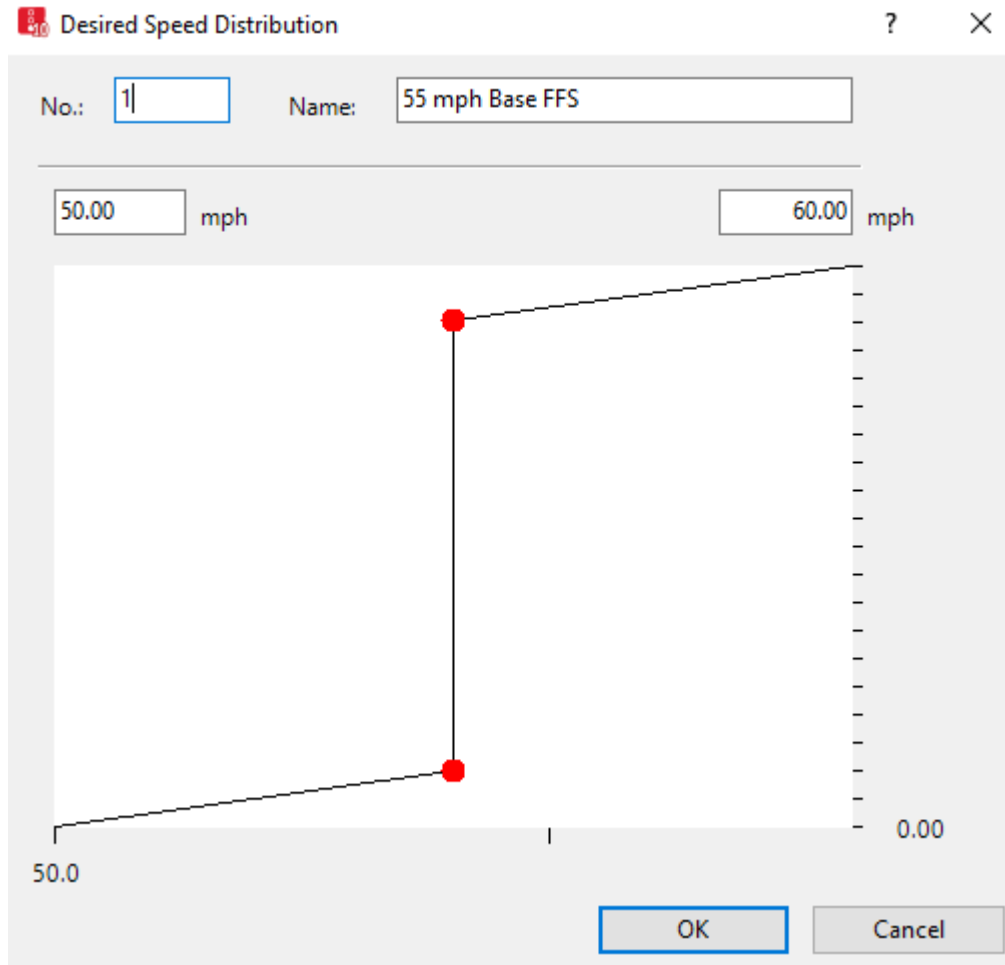
Figure 3- 9: Summary of All Analyzed Free-Flow Desired Speed Distributions

3.2.4 Vehicle Types and Composition

Following the principles set by the HCM, the vehicle type was chosen to be solely passenger car vehicles to match the equivalent passenger car demand required by the HCM's guidelines. No heavy gross vehicles (HGV) were introduced into the stream, allowing direct values of volume to represent equivalent passenger car demand. The vehicle type utilized in the stream was "car," found under the Base data menu and vehicle type list dialog box.

Figure 3- 10: Vehicle Type List

By the same concept mentioned above, the vehicle compositions are composed totally of passenger car vehicles for every desired speed distribution. The vehicle type composition was denoted by a relative flow value of 1.0 for every desired speed distribution representing the different free flow speeds being analyzed.

Figure 3- 11: Vehicle Compositions and Relative Flow List

3.2.5 Vehicle Inputs (Volumes)

After building vehicle compositions, volumes were inserted into the upstream points of available links. The vehicle demand volume was inserted by utilizing the vehicle input feature under network objects and selecting the upstream link containing both 12-foot wide lanes. Through the use of the vehicle input list dialog box, different values of demand flow rates were applied to the upstream end of the link taking into consideration the vehicle composition applied. The respective total volumes were inserted as passenger cars per hour per lane onto both upstream lanes to represent the capacity at the point of convergence and not necessarily within the upstream lanes.

3.2.6 Data Collection Points

In order to retrieve output information from the simulation, data points were inserted directly into the locations for which information was desired. Data points were inserted into the model by utilizing the "data collection points" tool under network objects. The locations for these data points were placed 100 ft. downstream of the beginning point of the model (upstream link) and at 100 ft. downstream after the merge section (downstream link). These locations served as an adequate representation of the free flow speed applied at the beginning of the traffic stream and at the recovery location after the merge section.

Furthermore, the same data points created above were further generated as collection measurements within the model. This was done under the Lists menu, measurements, and data collection category. In the data collection measurement list dialog box, all collection points were generated to allow the model to generate results at the specific locations corresponding to the data points placed previously.

3.2.7 Result Settings

Prior to obtaining results from the simulation runs, the result analysis output setting must be configured. Result settings were configured under the evaluation menu, configuration, and result attributes tab. Only the data collection check box was selected in order to retrieve data from the data points created. In order to make it easier to read results throughout the iterative process, outputs were limited to show only current runs and not save previous runs by selecting the "of current (multi) runs only" feature under the result management tab.

The data result types can be viewed by following the Lists menu, results, and data results process. Different data types can be viewed inside the data collection results list dialog box by selecting the attribute selection tool and adding the type of results desired. The following are the result types selected and the reasoning behind the selection for this particular analysis:

- Simulation run
    - Provided the simulation number which served to be beneficial when running two runs per simulation
- Time interval
    - Provided the simulation period
- Data collection measurement
    - Provided the location where data was being collected
- Vehicles (all)
    - Provided the number of vehicles that were able to cross a specific collection point. This was beneficial to see whether all vehicles were able to cross and whether a capacity may have been reached.
- Simulation Run/ Random Seed

o   Provided the random seed used for each run.

- Speed (Arithmetic Average)

    o   Provided the average speed of vehicles crossing the collection point with respect to all input parameters.

The speed resultant obtained from the above data collection table was the main factor of interest in this study. The objective of utilizing this method was to find speed distributions with respect to varying volume demands. The method constituted an iterative process by which volumes were increased within the model and speed data recorded for every iteration.

3.2.8 Run Simulation

Having set up the respective model geometry, desired distribution speeds, vehicle compositions, data collections points, and result outputs, a simulation was run for each free flow speed at different volumes. A single simulation was achieved by selecting the simulation continuous tool under the simulation menu bar and setting the speed knob at its maximum level for faster output of results. The data collection results list dialog box was continuously displayed in order to record average speed values of varying demands for each free flow speed distribution.

3.2.9 Driver Behavior Model

The tool that was utilized to show CAV characteristics and effects on a mixed traffic stream was the driver behavior model integrated within VISSIM's algorithm. The driver behavior models within VISSIM were edited under the Base Data menu and driver behavior list. Since the focus of the model was to analyze freeway behavior, the

Wiedemann 99 (freeways) as opposed to the Wiedemann 90 car-following model was used.

As previously discussed, the Wiedemann 99 car-following model counts with 10 modifiable parameters that describe the intrinsic behavior of drivers and vehicle capabilities. The parameters range from CC0 to CC9 and were found by opening the attribute selection dialog box and adding W99cc0 to W99c9 to the right side of the filter box. Lownes and Machemehl [9] describe each parameter as follows:

- W99cc0 – Stopped condition distance
- W99cc1 – Time Headway
- W99cc2 – "Following" Variation
- W99cc3 – Threshold for Entering "Following"
- W99cc4 – "Following" Thresholds
- W99cc5 – "Following" Threshold
- W99cc6 – Speed Dependency of Oscillation
- W99cc7 – Oscillation Acceleration
- W99cc8 – Stopped Condition Acceleration
- W99cc9 – Acceleration at 80 km/hr (50 mph)

Their research demonstrated that not all parameters significantly affect the capacities of highways and only 6 out of the 10 parameters were categorized as having significant effects on the capacity of the highway being analyzed. The 6 parameters that were found to have significant effects on capacity were CC0, CC1, CC2, CC4, CC5, and CC8. Through their research they calibrated values from observed data and these are summarized below:

Table 3- 5:Lownes & Machemehl Calibrated Parameters for VISSIM's Wiedemann Car-Following Model

| Wiedemann 99 Car-Following Model Calibrated Parameters (Lownes and Machemehl) ||
Description	Value
CC0:	4.8
CC1:	0.85
CC2:	13.2
CC3:	-8
CC4:	-1
CC5:	1
CC6:	11.44
CC7:	0.82
CC8:	11.5
CC9:	4.92

The study focused specifically on modifying the intrinsic behavior of time headway corresponding to the parameter CC1. While CC1 was changed to represent different average headways of CAVs, the values for the other 5 coefficients were modified to characterize more representative values of a basic freeway. The calibrated values demonstrated by Lownes and Machemehl [9] were utilized as the input values for each corresponding parameter found to be significant to capacity changes. The remaining parameters were left as default values, given that they had insignificant implications to the capacity of the model. The parameter values are summarized below:

Table 3- 6: Wiedemann 99 Car-Following Model Parameters Used in Study

Wiedemann 99 Car-Following Model Parameters	
Description	Value
CC0:	4.8
CC1:	variable
CC2:	13.2
CC3:	-8
CC4:	-1
CC5:	1
CC6:	11.44
CC7:	0.82
CC8:	11.5
CC9:	4.92

As mentioned above, the main variable that was changed to simulate CAV characteristic was the CC1 parameter. Nevertheless, a time distribution must first be established before the coefficient can be run within VISSIM's algorithm. This was done by selecting the Base Data menu, distributions, and time dialog box. A time distribution was added within the time distribution/ data point list dialog box, to represent a starting headway following a normal distribution with a mean time headway of 1 second and 0.1 seconds of standard deviation. Once the time distribution was established, the CC1 value was selected as the pre-established 1 second time headway distribution just created.

3.3 Intrinsic Driver Behavior Analysis with MATLAB Code

The main characteristic that was expected to influence highway capacity with respect to CAVs was the decrease in time headway between successive vehicles that was inherently connected to the perception reaction times experienced by human-drivers. Most, if not all, of the microsimulation software that are commercially available are

presently not equipped with the tools to analyz CAV characteristics within a traffic stream. The study succeeded in creating a micro-simulator through the use of MathWorks's MATLAB coding software, version R2017b.

3.3.1 Inputs

The inputs that were utilized in the code consist of the demand flow rate, market penetration, CAV expected time headways, jam density of freeway, and time step utilized within the simulation. The input variables were represented in the code as follows:

- Demand flow rate [pc/hr/ln]   → vh_input
- Market penetration [%]   → $market\_penetration$
- CAV time headway [seconds]   → AV_PIJR
- Jam density [veh/mi/ln]   → kj

The total simulation time for each run was chosen to be 3600 seconds representing an hour of simulation. A minimal time step value of less than a second was calculated following common practices used in dynamic assignment concepts. The time step within the simulation is therefore calculated by dividing the simulation period of 3600 seconds by the hourly demand flow rate times a multiple of 10 to decrease the time step value. The time step calculation was therefore performed by the following formula:

$$delta\_t = \frac{3600}{Vh\_input \times 10}$$

Where $delta\_t$ is the time step value for the specific simulation run with respect to the demand flow rate being analyzed.

In the analysis process, the vehicle inputs were changed iteratively in order to obtain the distribution of speeds with respect to increasing demand flow rates. The market

penetration variable was also changed periodically in order to analyze 4 penetration market values consisting of 40%, 60%, 80%, and 100% CAV inclusion into the traffic system. The CAV headway was changed as well in order to analyze different possible headway values that were expected from CAV technology. Three separate CAV headway values were analyzed that consisted of 1.0, 0.8, and 0.5 seconds followed by CAV vehicles. The code assumes that CAVs will follow preceding vehicles regardless of whether they are HVs or CAVs and that their following time headways remain the same.

3.3.2 Perception Reaction Time/ Time Headway

The perception-reaction time (PIJR) was one of the main factors that would affect the output of speed results and capacity constraints. For the purposes of this study, the PIJR value was made equivalent to the time headway of vehicles. The following variables are represented as follow:

- Perception reaction time [seconds]   →   *my_mean*
- Upper and lower bounds of PIJR [seconds]   →   *my_threshold*
- # of standard deviations/ Level of confidence   →   *my_num_stnd_dev*

The PIJR values were changed periodically with respect to the free flow speed (FFS) being analyzed. Through an iterative process, the PIJR values were calibrated with respect to the target capacities provided by the empirical/ theoretical procedures mentioned in the HCM guidelines. The calibration process was accomplished by comparing capacities attained by the code to that of the capacities calculated by the HCM guidelines for HVs. By attaining similar capacities of HVs at each respective FFS, the code is calibrated to HV characteristics and PIJR values can be initialized for every subsequent run. The calibration results with respect to free flow speeds were as follows:

Table 3- 7: Calibrated Perception-Reaction Times with respect to HCM Capacity Guidelines

Calibrated PIJR Times	
Free Flow Speed FFS [mi/hr]	Perception-Reaction Time PIJR [seconds]
55	1.64
60	1.66
65	1.66
70	1.66
75	1.71

In order to show randomness of vehicle reaction times between HVs, a normal distribution function was selected by utilizing the PIJR times as the mean of the distribution with respect to a certain standard deviation. The standard deviation was obtained by dividing the upper and lower thresholds by 3 standard deviations representing a 99% level of confidence. The formula for the standard deviation of PIJR values was represented by the following formula:

$$my\_stand = \frac{my\_threshold}{my\_num\_stnd\_dev}$$

Where my_stand is the standard deviation of the PIJR normal distribution.

3.3.3 Vehicle Length

Similar to the PIJR times, a mean vehicle length was chosen to represent the average car length within a normal distribution of traffic stream. The mean vehicle length was chosen as 15 feet with a possible threshold of $\pm$ 2 feet. The variables for the normal distribution were represented as follow:

- Average vehicle length [ft.]   →   *my_mean_length*

- Upper and lower bounds of veh. Length [ft.] →   *my_mean_lenth_threshold*

- # of standard deviations/ Level of confidence→   *my_num_stnd_dev_length*

The standard deviation was calculated in the same manner as the standard deviation

for PIJR.

$$my\_stnd\_length = \frac{my\_mean\_lenth\_threshold}{my\_num\_stnd\_dev\_length}$$

where *my_stnd_length* is the standard deviation of the average vehicle length. The

parameters within the vehicle length category were treated as constants and were not

changed with respect to the iterative input efforts.

3.3.4 Initial Speed

The initial speed within the code represented the FFS of the freeway segment at

ideal conditions. The same concepts of randomness applied to the initial speed depicting

a desired speed distribution with respect to the vehicles within the traffic stream. Similar

to the idea implemented in the VISSIM attempt, a 5 mile per hour threshold was assigned

to the average free flow speed being analyzed. The initial speed distribution followed a

normal distribution with an average free flow speed and a standard deviation equal to the

threshold divided by 3 standard deviations. The variables were represented as follow:

- Average initial FFS [mi/hr]   →   *my_mean_initial_speed*

- Upper & lower bounds of FFS [mi/hr]→   *my_mean_initialspeed_threshold*

- # of standard deviations/ LOC   →   *my_num_stnd_dev_initialspeed*

The standard deviation of the free flow speed is calculated by the following formula:

$$my\_stnd\_initialspeed = \frac{my\_mean\_initialspeed\_threshold}{my\_num\_stnd\_dev\_initialspeed}$$

The average initial FFS variable was periodically changed in order to analyze each individual FFS scenario with respect to the variable inputs of the code. The average FFS variable specifically represented the different FFSs being analyzed for each speed-flow distribution curve being plotted.

3.3.5 Market Penetration

A very important process required within the analysis scheme was to be able to change the fraction of CAVs within the traffic stream and account for them through proportions of the overall whole. This was achieved by applying the concepts stated by the cumulative distribution function (CDF) into the code. A random number generator was created within the code to represent an unbiased probability of vehicle generated and compared to the supplied market penetration percentage for CAVs. For any randomly generated number that fell below the supplied market penetration value, the immediate vehicle input PIJR value would be set to the smaller, constant CAV time headway value supplied. The relationship between the random number generator and the market penetration is shown in the formula below:

$$if \quad random\_gen \leq \frac{market\_penetration}{100}$$

$$then, \quad Vh\_pijr(i) = AV\_PIJR$$

where $random\_gen$ was the random number being generated, $market\_penetration$ was the CAV market penetration in percentage, $Vh\_pijr(i)$ was the HVs' perception-reaction times in seconds, and AV_PIJR was the time headway of CAVs in seconds.

3.3.6 Vehicle Speed & Position Matrices

In order to obtain the position and speed of each vehicle input at the initial stage, a speed and position matrix was created by initializing the position of each vehicle with zeros and by setting each vehicle's speed to the initial speed captured by the average FFS variable established previously. The random initial speeds are captured in a matrix of initial speeds which are utilized to calculate vehicle positions at a later time step. The vehicle position for latter time steps was calculated by adding the current vehicle position to the traveled distance within one-time step. The latter vehicle position was represented as follows:

$$Vh\_POS(i, time + 1) = Vh\_POS(i, time) + 1.47 \times Vh\_Speed(i, time) \times delta\_t$$

where $Vh\_POS(i, time + 1)$ was the vehicle position at the next time step in feet, $Vh\_POS(i, time)$ was the vehicle position at the current time step in feet, $Vh\_Speed(i, time)$ was the vehicle speed at the current time step in miles per hour, $delta\_t$ was the time step with respect to the demand flow rate in seconds, and the 1.47 constant was for unit conversion purposes applied to the vehicle speed in miles per hour. By utilizing the formula above, a position matrix was stored with all vehicle positions at all time steps within the simulation period.

3.3.7 Pipes Car-Following Model

The main component that characterized the simulator was found within the car following model utilized in the code. As previously mentioned in the literature review section, the Pipes car following model that is found in the CORSIM microsimulation software utilizes the headway distance between a preceding and following vehicle with a driver's sensitivity to change. The formula for the car following model is as follows:

$$u = min\left(\frac{h - h_j}{c_3}, u_f\right)$$

where u is the speed of the following vehicle at the next time step [ft/s], h is the distance headway between the front bumper of the preceding vehicle and the front bumper of the following vehicle [ft.], h_j is the distance headway when vehicles are completely stopped in queue (ft), c_3 is the driver sensitivity factor [sec], and u_f is the roadway free flow speed [ft/s]. The code was developed such that the distance headway is represented by the difference in vehicle position between preceding and latter positions, and then subtracting the jam density spacing including the average vehicle length previously established in the code. The driver sensitivity factor was taken to be the vehicle's PIJR time established previously. In addition, the free flow speed was taken to be the desired speed of the vehicle being analyzed with respect to the current time step. The algorithm was set to choose the minimum value between the calculated speed considering the first term of the equation, and the desired free flow speed of the vehicle being analyzed. The jam density was kept constant throughout the entire process and the vehicle speed for every time step was calculated with the following formula in the code:

$$Vh\_Speed(i, time + 1)$$
$$= min\left(\frac{Vh\_POS(i - 1, time) - Vh\_POS(i, time) - \left(\frac{1}{k_j} \times 5280\right) - my\_mean\_length}{Vh\_pijr(i)}, my\_mean\_initialspeed\right)$$

Where $Vh\_Speed(i, time + 1)$ is the vehicle speed at the next time step in feet per sec, $Vh\_POS(i - 1, time)$ is the position of the preceding vehicle in feet, $Vh\_POS(i, time)$ is the position of the current vehicle in feet, k_j is the jam density in vehicles per mile per lane, my_mean_length is the average length of vehicles in the traffic stream in feet, $Vh\_pijr(i)$ is

the average headway of the vehicle in seconds, *my_mean_initialspeed* is the desired initial speed of the current vehicle being analyzed in miles per hour, and the 5280 constant is for unit conversion purposes applied to the vehicles per mile jam density.

An exception to the above calculation was established only for the first vehicle in the traffic stream. The first vehicle in the iteration did not receive any constraint by the car-following model because the first vehicle did not experience any reduction in speed and always maintained its own desired speed throughout the simulation. Therefore, the vehicle speed for the first vehicle was always its initial speed at all time steps throughout the whole simulation.

3.3.8 Detector Location

In order to avoid the same error encountered in VISSIM when analyzing a single freeway lane with no perturbation, the code included a detector location that records the number of vehicles that were able to cross an upstream location with respect to the demand flow rate input. The detector location was set at 100 feet downstream of the starting point of the simulation. Temporary vehicle positions as well as speeds were recorded for all vehicle inputs at all time steps. A detector speed matrix was stored showing the speeds of the vehicles that were able to cross the detector location with respect to the demand input. The detector speed matrix was calculated by interpolating the temporary positions and speeds with respect to the detector location.

Finally, an average speed of all vehicles able to cross the detector location was obtained from the values saved under the detector speed matrix. The number of vehicles able to pass the detector location was represented by the size of the detector speed matrix. If the size of the detector speed matrix matched the number of vehicles of the demand input, then all vehicles were said to have passed through the freeway segment and speed

values were recorded. Once the detector speed matrix size was found to be lower than the demand input, the maximum detector speed matrix size was recorded as the capacity of the freeway segment. For simplicity purposes and to account for error, capacity values were rounded down to the nearest units with intervals of 50 vehicles per hour per lane.

4.0 Analysis

This chapter describes the results obtained from the prior experiments conducted utilizing the VISSIM microsimulation software and MATLAB code simulator. The two experiments were conducted with the same objective in mind; to estimate capacities of a basic freeway segment considering possible lower time headways produced by proportions of CAVs in the traffic stream. Both approaches intended to modify the intrinsic behaviors of vehicles by decreasing the time headway after calibrating the models with the empirical data and theoretical formulas provided by the HCM. The results were unfavorable when utilizing the VISSIM software to represent CAV characteristics. Nevertheless, the MATLAB code simulator did result in generally accepted values of expected capacities and speed distributions.
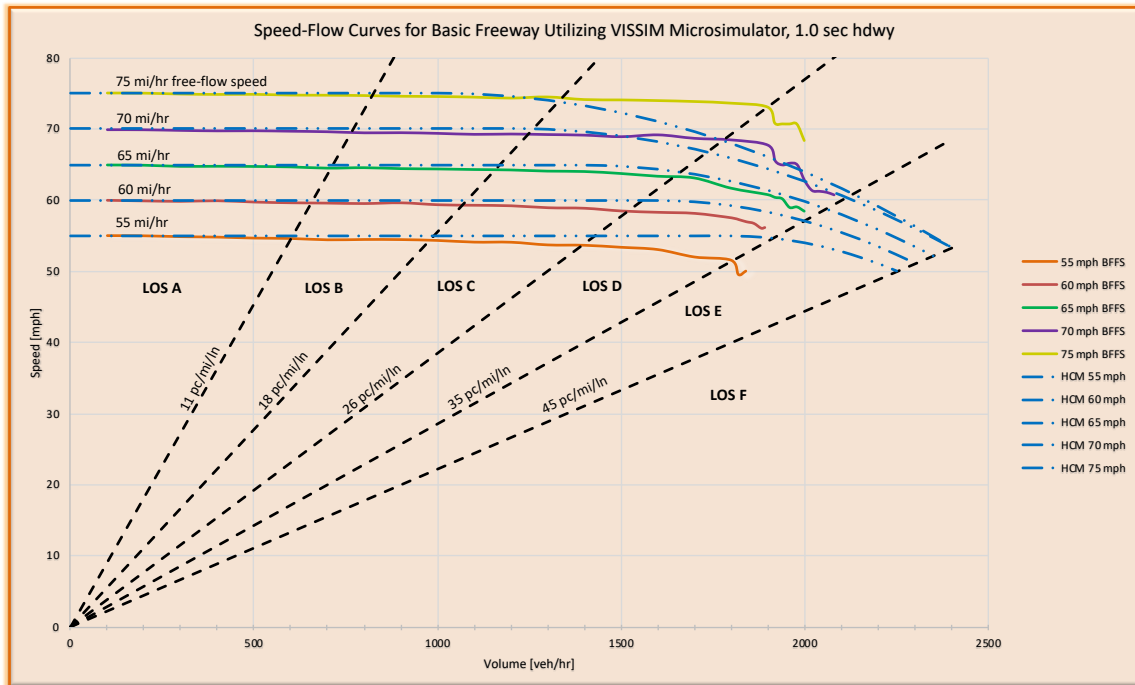
4.1 VISSIM Software

Speed distribution curves were plotted against varying demands representing a traffic stream composed of only passenger car vehicles in a basic freeway environment. Each simulation outputted two runs, of which one average speed was obtained with respect to different random seeds. The different seeds used simulated randomness of desired speed and intrinsic behavior for every vehicle in the traffic stream. The respective speed of each varying demand was obtained by taking the average of both post-convergence speed outputs of each run from the hour-long simulation. The average downstream speed was plotted against a steadily increasing demand creating speed distribution curves with respect to each starting free flow speed. Furthermore, the resulting speed-flow curves were overlaid on top of the predetermined HCM speed-flow plot that represents the procedure following HCM guidelines.

The Level of Service (LOS) lines creating the LOS regions were obtained by dividing the varying demand volumes by the constant upper bound of each respective LOS region set by the HCM's guidelines. The resultant values represent the speed values with respect to each flow rate at a constant density.

4.1.1 Calibration Effort

The starting point of the calibration effort was conducted using a 1 second time headway by setting the CC1 parameter under the Wiedemann 99 driver behavior list and keeping all other parameters constant at the previously stated values. The following figure shows the speed-flow distribution curves for the five FFSs being analyzed:

Figure 4- 1: Results of VISSIM Analysis at CC1 Parameter Equal to 1.0 Second Time Headway

Speed-Flow Curves for Basic Freeway Utilizing VISSIM Microsimulator, 1.0 sec hdwy

The result of the plot shows that the VISSIM algorithm outputs a decaying speed distribution curve at higher demands, as expected. Nevertheless, the capacities of the curves produced from the VISSIM software fall short of the capacities obtained from the theoretical formulas provided by the HCM guidelines. The capacity differences between the simulation results and HCM's guidance vary from approximately 320 to 400 passenger cars per hour per lane. At this point, the overall tendency of the curves was still taken as favorable given that the curves have a pattern similar to the expected decreasing speeds as volumes approach capacities.

4.1.2 Car-Following Constraints

Following the initial data obtained for 1 second headways, the study focused on increasing or decreasing the CC1 parameter keeping all else constant. The results yielded values that were unfavorable to the study. The study found that by simply decreasing the time headway in the driver behavior model, the results only slightly changed the capacity values of the model with respect to high volumes.

Smaller time distributions were setup in order to simulate average time headways produced from CAVs in the traffic stream consisting of values ranging from 0.3 to 1.0 second. An effort was made to analyze the model at the extreme case of 0.3 second headways and the outputs showed a maximum capacity of approximately 2600 vehicles per hour per lane. Based on prior literature review, the volumes obtained at the extreme case of 0.3 second headway were not satisfactory to the range of volumes expected. For this reason, the VISSIM attempt was abandoned and focus was switched to the MATLAB code simulator to analyze the intrinsic behavior of CAV penetration.

The inference that was made as to why the VISSIM software was not able to produce higher capacities was due to software constraints placed upon the car following model, based on conventional vehicle characteristics.

4.2 MATLAB Code Simulator

Continuing the aforementioned approach, an effort was made to capture the intrinsic characteristics expected from CAV inclusion into the traffic stream. A series of speed data points were recorded with respect to different combinations of market penetrations and expected CAV time headways. The speed data points captured were based on the average of speeds within the detector speed matrix created. Two runs were

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completed for every simulation scenario in order to obtain an average of different outcomes for the varying volumes being analyzed.

The different scenarios included analyzing CAV market penetration rates of 40%, 60%, 80%, and a full penetration of 100%. Each penetration rate scenario included three different CAV expected time headways below the calibrated PIJR values for HVs. The three time headways analyzed consisted of 1.0, 0.8, and 0.5 second headways. Each scenario was evaluated with respect to the initial density thresholds set by the HCM and the LOS regions were modified accordingly to the speed distribution curves. The LOS E density value was used as the upper bound, consistent with the approximate capacities obtained from the curve distributions at differing free flow speeds. A percent change scheme was applied to LOS E based on HCM's LOS threshold bounds, thereby, keeping consistent ranges between the different LOS regions A through E.

4.2.1 Code Calibration with HCM Data

The first step in the process was to calibrate the MATLAB code simulator with capacity data obtained from the HCM. The market penetration variable was set to 0, representing a traffic stream composed of only conventional human-driven vehicles. Each initial free flow speed was held constant, while the PIJR time variable was calibrated based on the capacity output of the speed detector matrix and corresponding average speed. Once each calibrated PIJR time was obtained with respect to the different FFSs, average speed data points were recorded for every volume analyzed with respect to the FFS.

Speed-flow curves of the analysis were plotted and overlaid on top of the HCM's calculated curves. The following figure shows the relationship between the code's plotted curves vs. the HCM's curve:

Figure 4- 2: Speed-Flow Curves of MATLAB Code Simulator at Calibration Stage
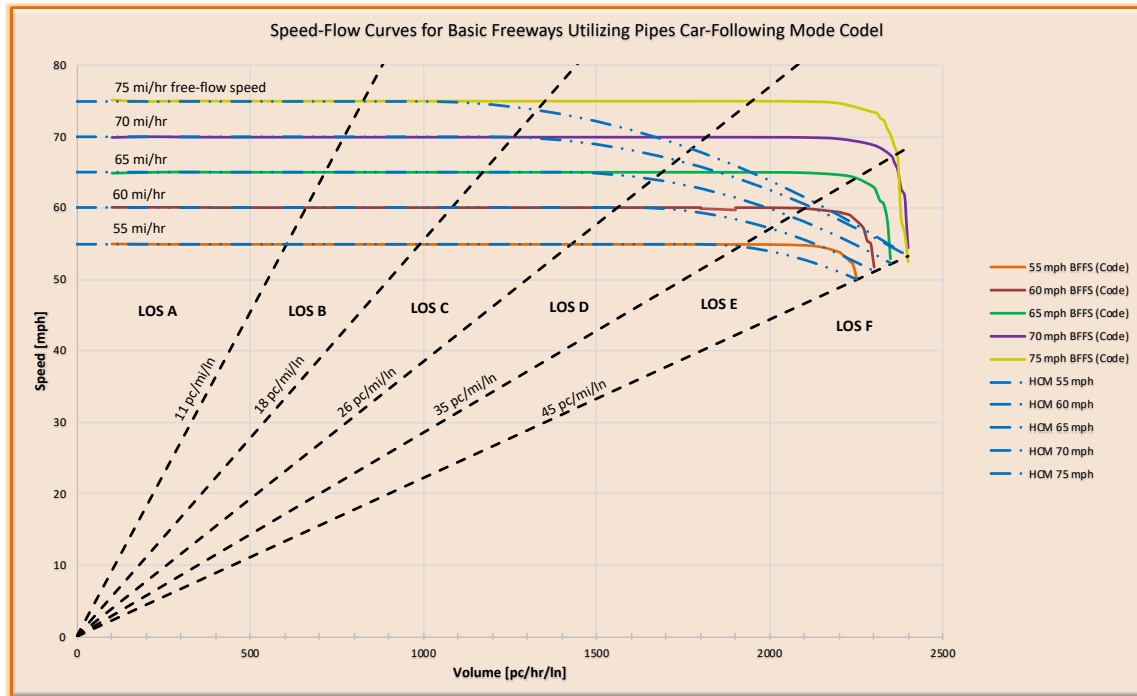

The plot above, shows that the approximate capacities obtained from the MATLAB code are very close to the capacities set by the HCM at different FFSs. Nevertheless, the study observed that the curves based on the Pipes Model did not necessarily decay as smoothly as the HCM's approach based on empirical data. Even though the speed-flow curve distributions do not exactly align with the HCM curve behaviors, the capacities obtained were still deemed beneficial to the study and the analysis was set to continue.

**4.2.2 40% Market Penetration of CAVs with 1, 0.8, & 0.5 Second Headways**

The first experiment consisted of changing the market penetration from 0 to 40% and equating the CAV time headway to 1.0 seconds. The PIJR time for HVs were modified according to the FFS being analyzed to depict HV behavior, while the CAV headways were kept constant throughout the differing FFS simulations. Simulations for each free flow speed were then run, while changing the vehicle inputs at 100 vehicle per hour increments until the capacity of the model was reached. Similar to prior procedures, an average speed was calculated from the different runs of each FFS-flow simulation and data plotted against the HCM curves. The following figure shows the results from an applied 40% CAV market penetration and 1.0 second CAV headway:

Figure 4- 3: Code Simulator Speed-Flow Curves at 40% Market Penetration and 1.0 Second CAV Time Headway
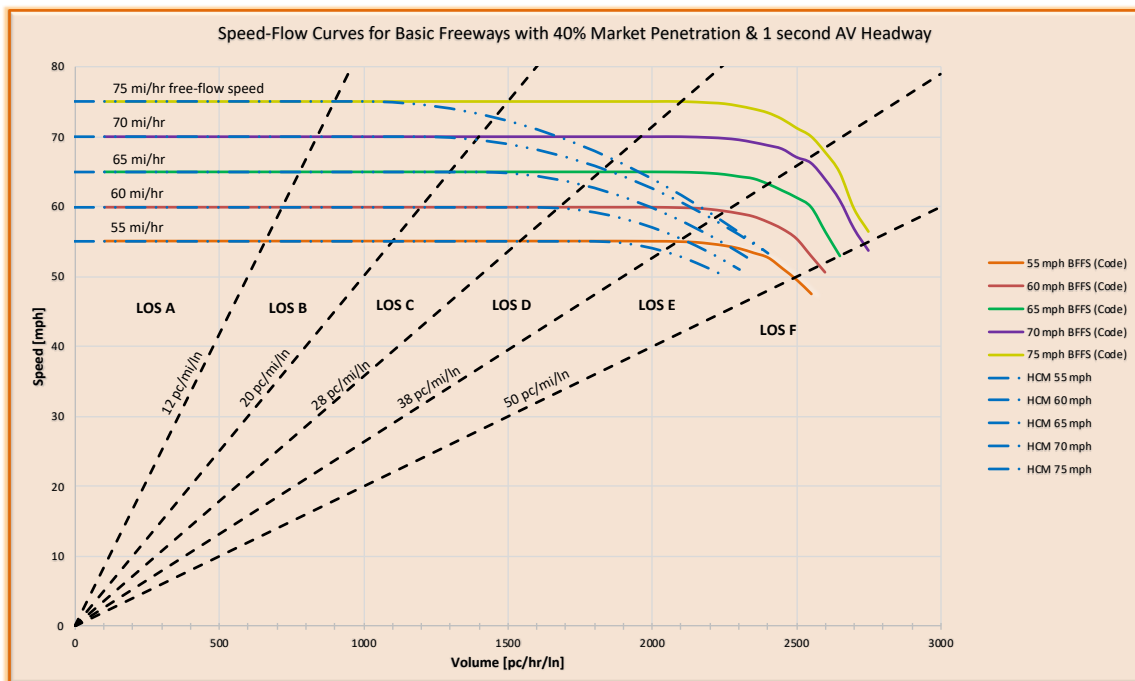
A plot of the 40% CAV market penetration with CAV time headway of 0.8 seconds is presented in Figure 4-4.

Figure 4- 4: Code Simulator Speed-Flow Curves at 40% Market Penetration and 0.8 Second CAV Time Headway
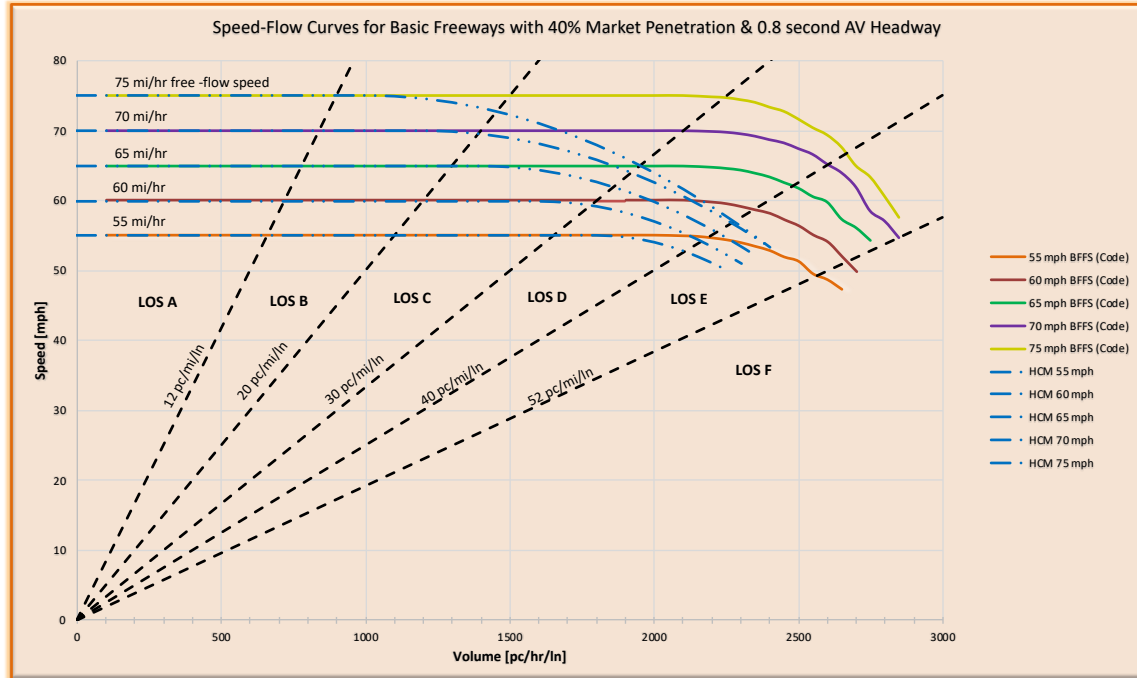
The 40% CAV market penetration with CAV time headway of 0.5 second plot was as follows:

Figure 4- 5: Code Simulator Speed-Flow Curves at 40% Market Penetration and 0.5 Second CAV Time Headway
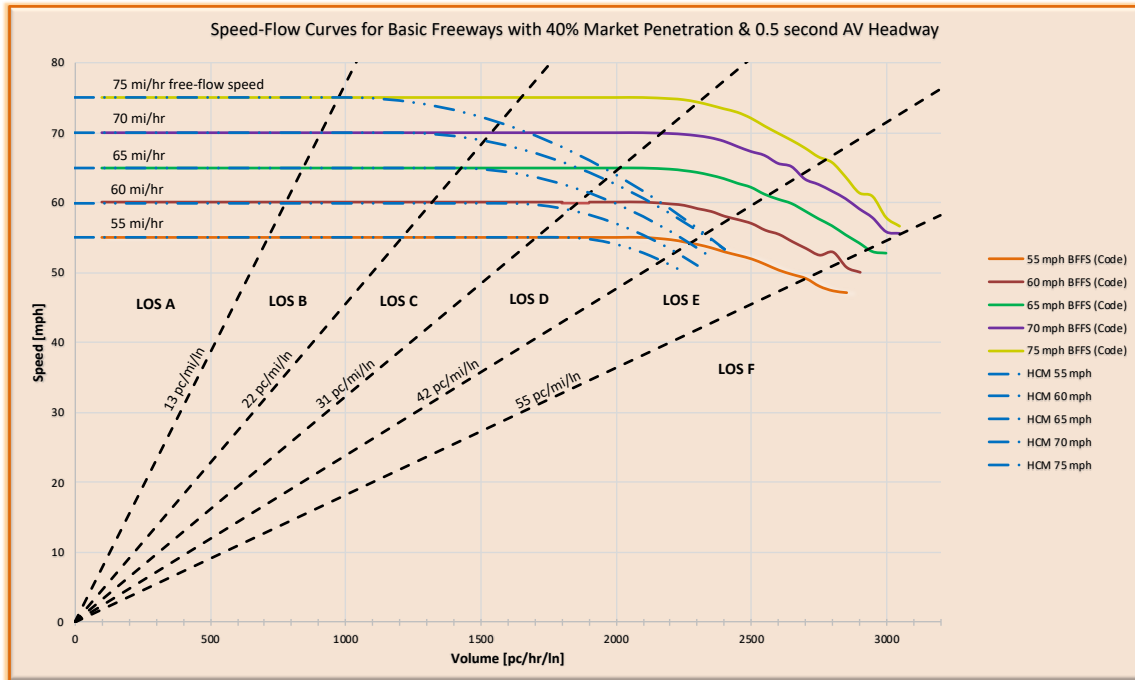
The figures above show a similar behavior of speed-flow curves to that of the HCM curves, but at higher capacities. The LOS constant density line increased by an estimated 5, 7, & 10 passenger cars per mile per lane for the 1.0, 0.8, and 0.5 second CAV headways for the LOS E threshold compared to the 0% CAV market penetration scenario, respectively. The characteristics based on the Pipes Car Following model did tend to show an increasing trend in capacity of the basic freeway model and a decaying speed distribution behavior.

4.2.3 Capacity and Level of Service Thresholds for 40% CAV Market Penetration

A summary of the new capacities obtained from the 40% market penetration simulation and varying CAV time headways were tabulated and shown below:

Table 4- 1: Density Thresholds at 40% Market Penetration and 1.0, 0.8, & 0.5 second Time Headways

Density Thresholds at 40% CAV Market Penetration					
CAV Headway Time [seconds]	LOS A [pc/mi/ln]	LOS B [pc/mi/ln]	LOS C [pc/mi/ln]	LOS D [pc/mi/ln]	LOS E [pc/mi/ln]
1.0	12	20	28	38	50
0.8	12	20	30	40	52
0.5	13	22	31	42	55

The subsequent sections followed the same procedure applied to the 40% market penetration scenario, with the exception of replacing the market penetration value and CAV time headway with different values consisting of 1.0, 0.8, or 0.5 seconds.

4.2.4 60% Market Penetration of CAVs with 1, 0.8, & 0.5 Second Headways
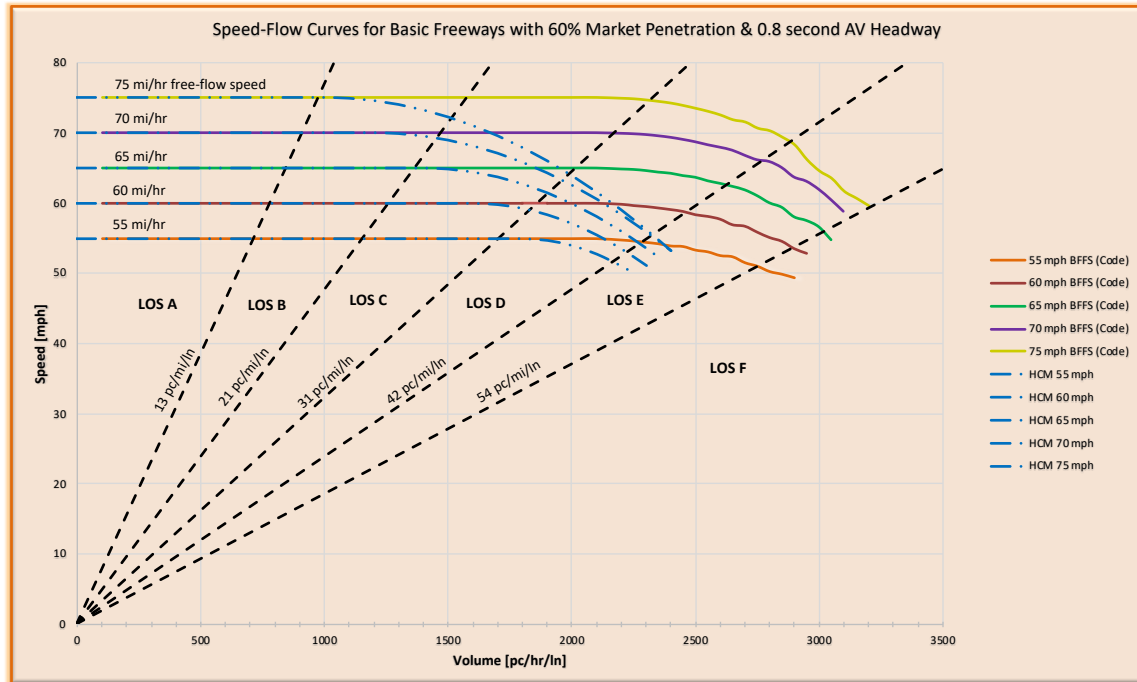
The 60% CAV market penetration with varying CAV time headway plots are as follow:

Figure 4- 6: Code Simulator Speed-Flow Curves at 60% Market Penetration and 1.0 Second CAV Time Headway
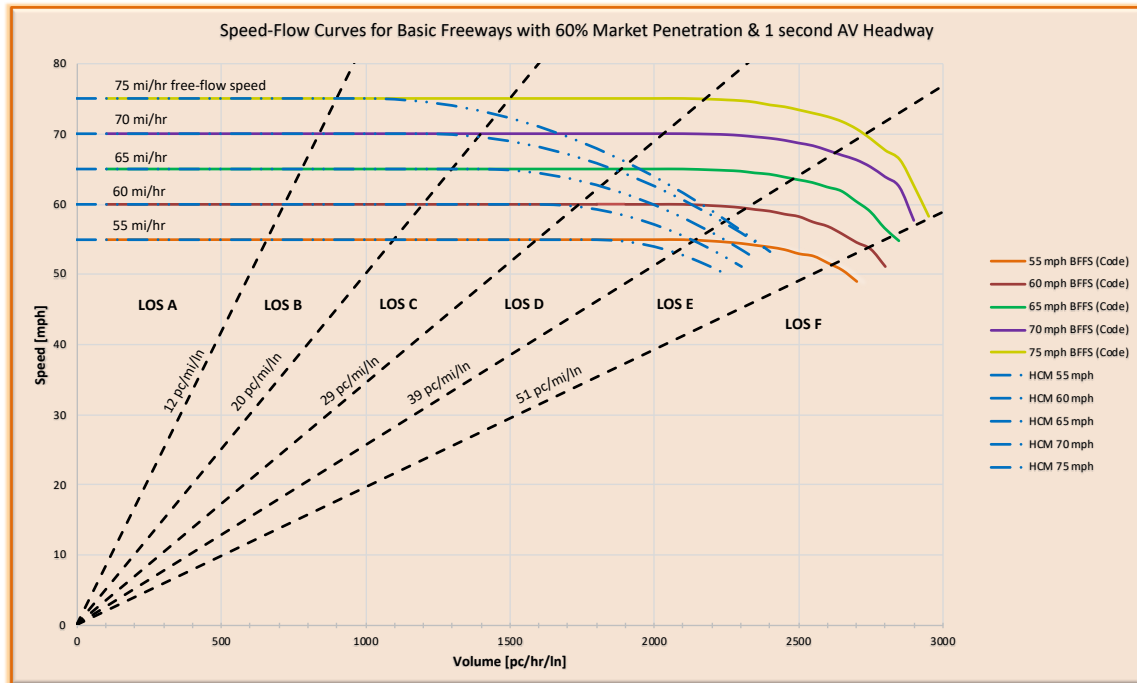
The 60% CAV market penetration with CAV time headway of 0.8 seconds plot was as follows:

Figure 4- 7: Code Simulator Speed-Flow Curves at 60% Market Penetration and 0.8 Second CAV Time Headway

The 60% CAV market penetration with CAV time headway of 0.5 seconds plot was as follows:

Figure 4- 8: Code Simulator Speed-Flow Curves at 60% Market Penetration and 0.5 second CAV Time Headway
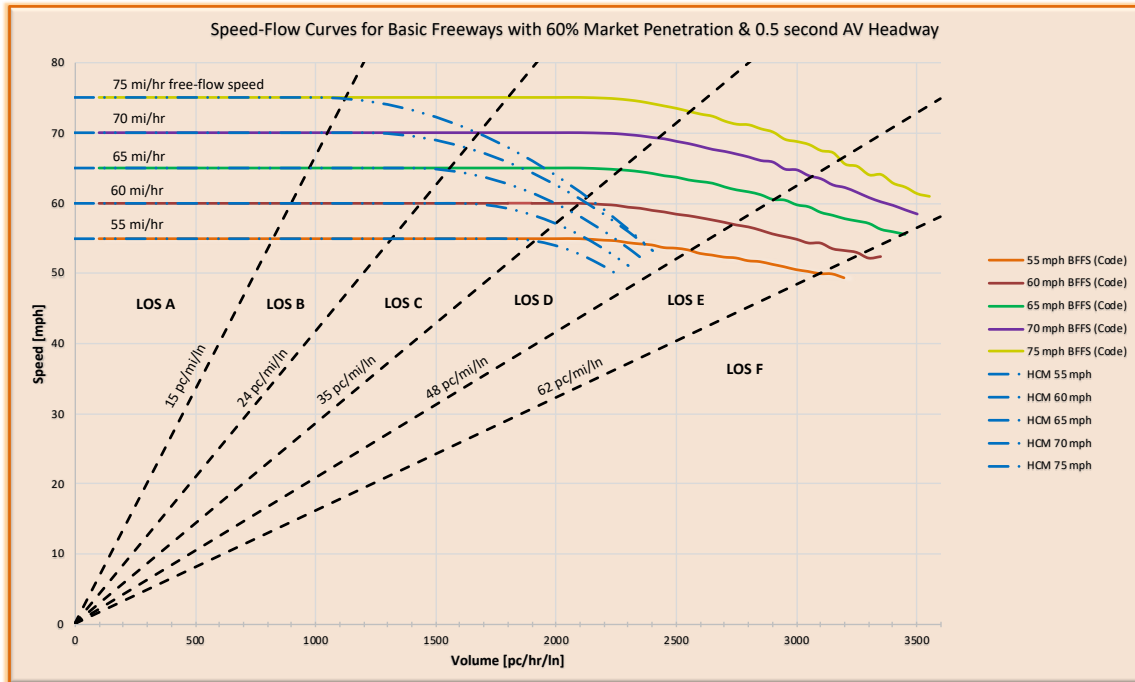
The LOS constant density line increased by an estimated 6, 9, & 17 passenger cars per mile per lane for the 1.0, 0.8, and 0.5 second CAV headway based on the LOS E threshold when compared to the 0% CAV market penetration scenario, respectively.

One of the characteristics that was observed by comparing the difference in shorter time headways between the 40% and 60% market penetration scenarios was that the speed-flow curves became flatter and less pronounced with lower CAV time headways. This may be representative of the constant connection and immediate stopping potential of CAVs between each other.

The higher 70 and 75 mile per hour per lane curves are observed to reach capacities at lower density values than those described by a constant LOS E diagonal line.

4.2.5 Capacity and Level of Service Thresholds for 60% CAV Market Penetration

A summary of the new capacities obtained from the 60% market penetration simulation and varying CAV time headways were tabulated and shown below:

Table 4- 2: Density Thresholds at 60% Market Penetration and 1.0, 0.8, & 0.5 second Time Headways

Density Thresholds at 60% CAV Market Penetration					
CAV Headway Time [seconds]	LOS A [pc/mi/ln]	LOS B [pc/mi/ln]	LOS C [pc/mi/ln]	LOS D [pc/mi/ln]	LOS E [pc/mi/ln]
1.0	12	20	29	39	51
0.8	13	21	31	42	54
0.5	15	24	35	48	62

4.2.6 80% Market Penetration of CAVs with 1, 0.8, & 0.5 Second Headways

The 80% CAV market penetration with varying CAV time headways plots are as
follow:

Figure 4- 9: Code Simulator Speed-Flow Curves at 80% Market Penetration and 1.0
second CAV Time Headway
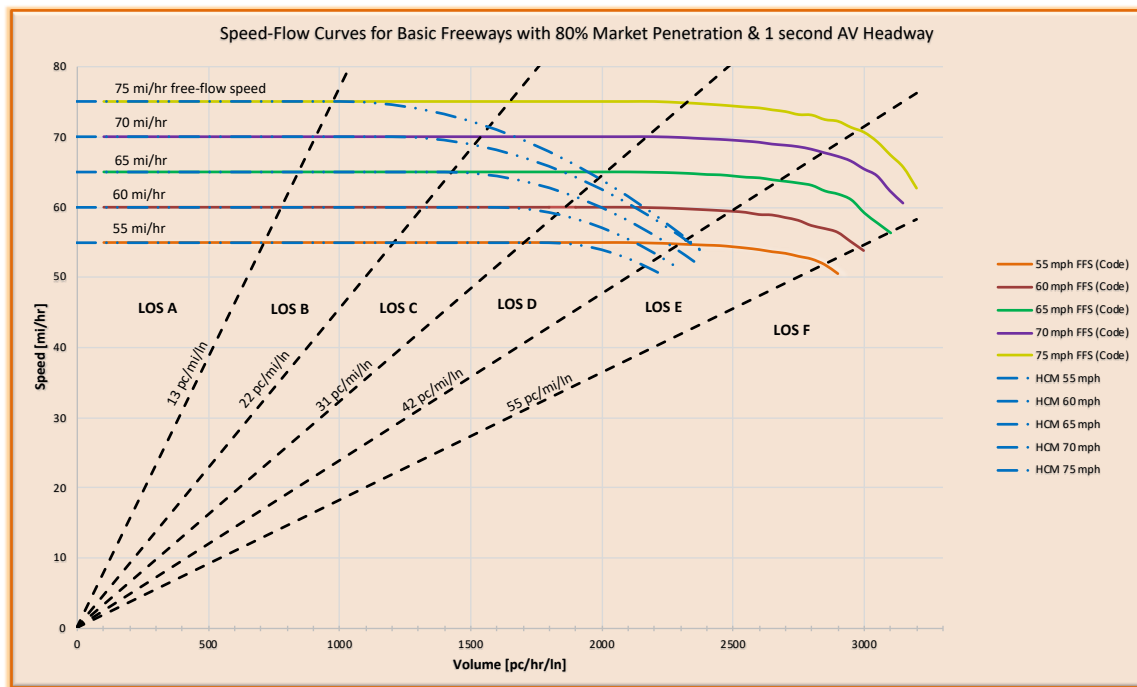
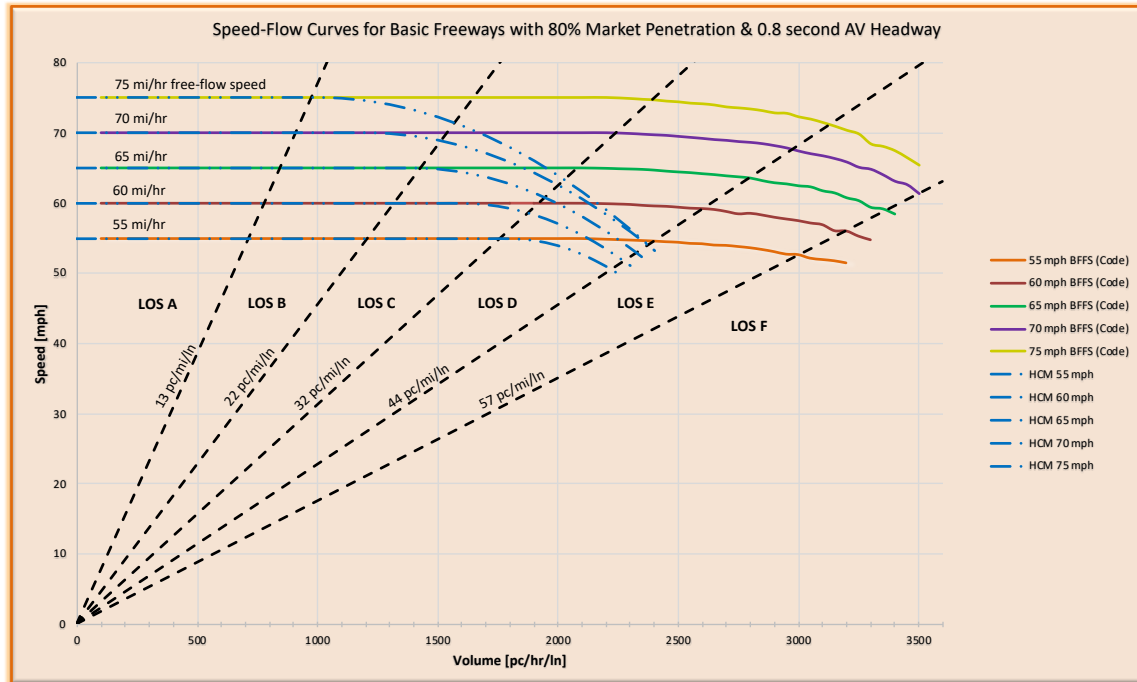The 80% CAV market penetration with CAV time headway of 0.8 seconds plot was as follows:

Figure 4- 10: Code Simulator Speed-Flow Curves at 80% Market Penetration and 0.8 second CAV Time Headway

The 80% CAV market penetration with CAV time headway of 0.5 seconds plot was as follows:

Figure 4- 11: Code Simulator Speed-Flow Curves at 80% Market Penetration and 0.5 second CAV Time Headway
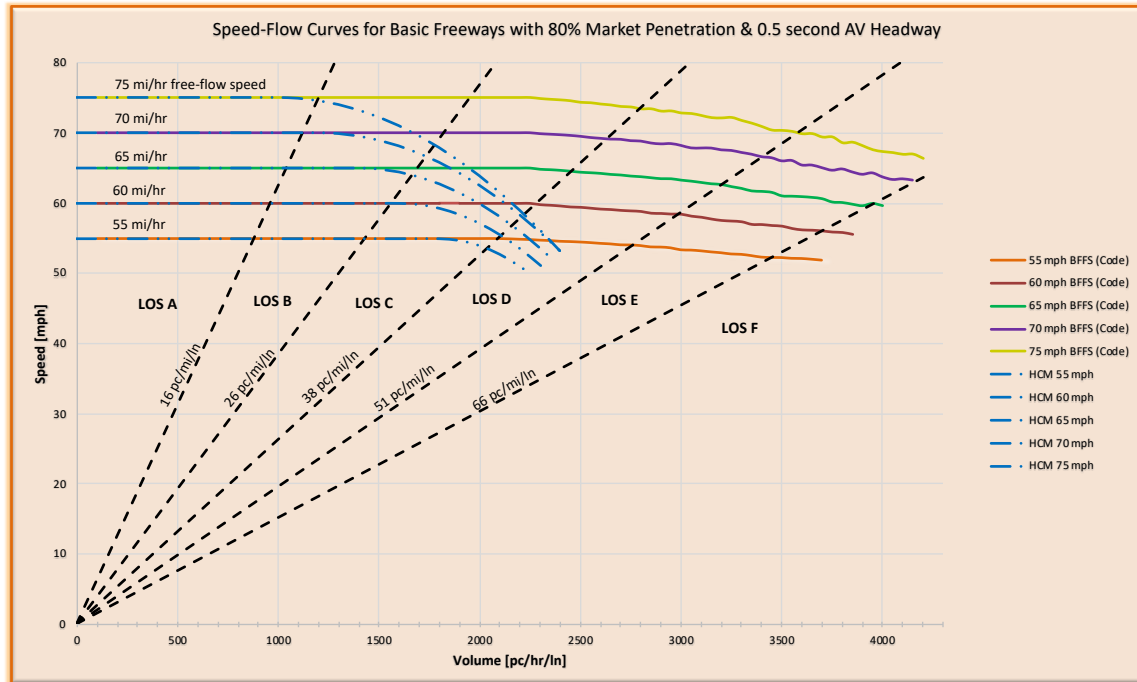
The LOS constant density line increased by an estimated 10, 12, & 21 passenger cars per mile per lane for the 1.0, 0.8, and 0.5 second CAV headway based on the LOS E threshold when compared to the 0% CAV market penetration scenario, respectively.

Results from the 80% CAV market penetration scenario reinforced the idea that shorter time headways produce flatter curves as the basic freeway approached capacities. The 0.5 second time headway at 80% market penetration figure demonstrated a much flatter characteristic than those experienced by longer time headways at the same market penetration. This characteristic demonstrated that traffic streams with a higher inclusion

of CAVs at shorter time headways are expected to experience a more abrupt slowdown of the whole traffic stream.

4.2.7 Capacity and Level of Service Thresholds for 80% CAV Market Penetration

A summary of the new capacities obtained from the 80% market penetration simulation and varying CAV time headways were tabulated and shown below:

Table 4- 3: Density Thresholds at 80% Market Penetration and 1.0, 0.8, & 0.5 second Time Headways

Density Thresholds at 80% CAV Market Penetration					
CAV Headway Time [seconds]	LOS A [pc/mi/ln]	LOS B [pc/mi/ln]	LOS C [pc/mi/ln]	LOS D [pc/mi/ln]	LOS E [pc/mi/ln]
1.0	13	22	31	42	55
0.8	13	22	32	44	57
0.5	16	26	38	51	66

4.2.8 Full Market Penetration of CAVs with 1, 0.8, & 0.5 Second Headways

The 100% CAV market penetration with varying CAV time headways plots are as follow:

Figure 4- 12: Code Simulator Speed-Flow Curves at 100% Market Penetration and 1.0 second CAV Time Headway
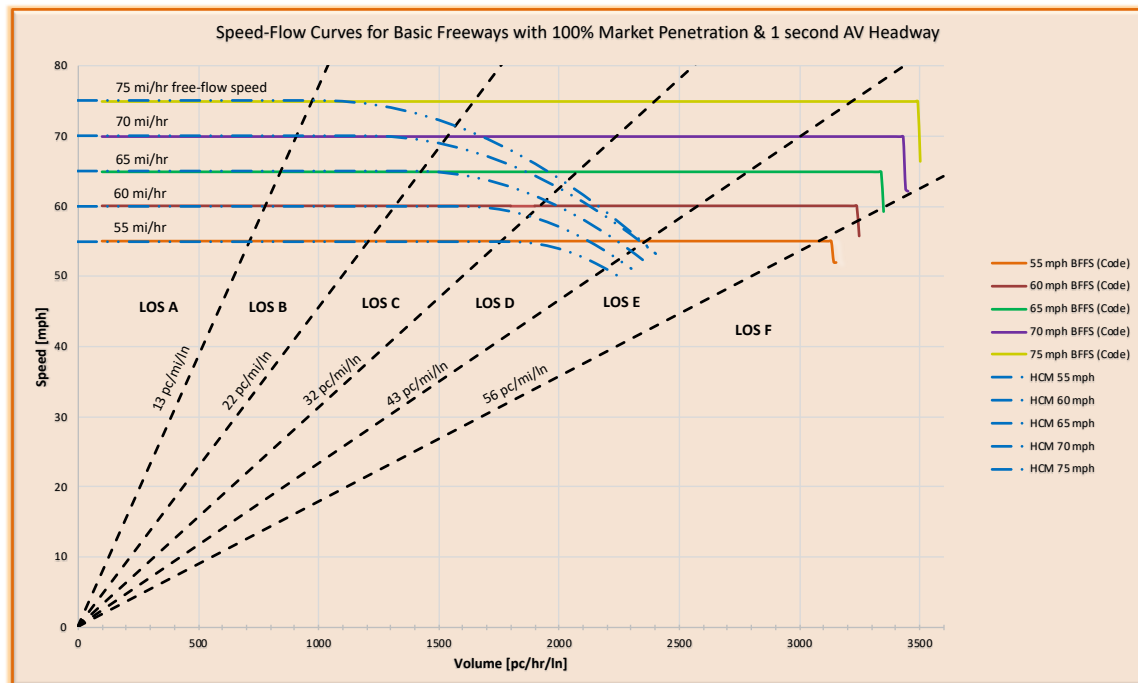
The 60% CAV market penetration with CAV time headway of 0.8 seconds plot was as follows:

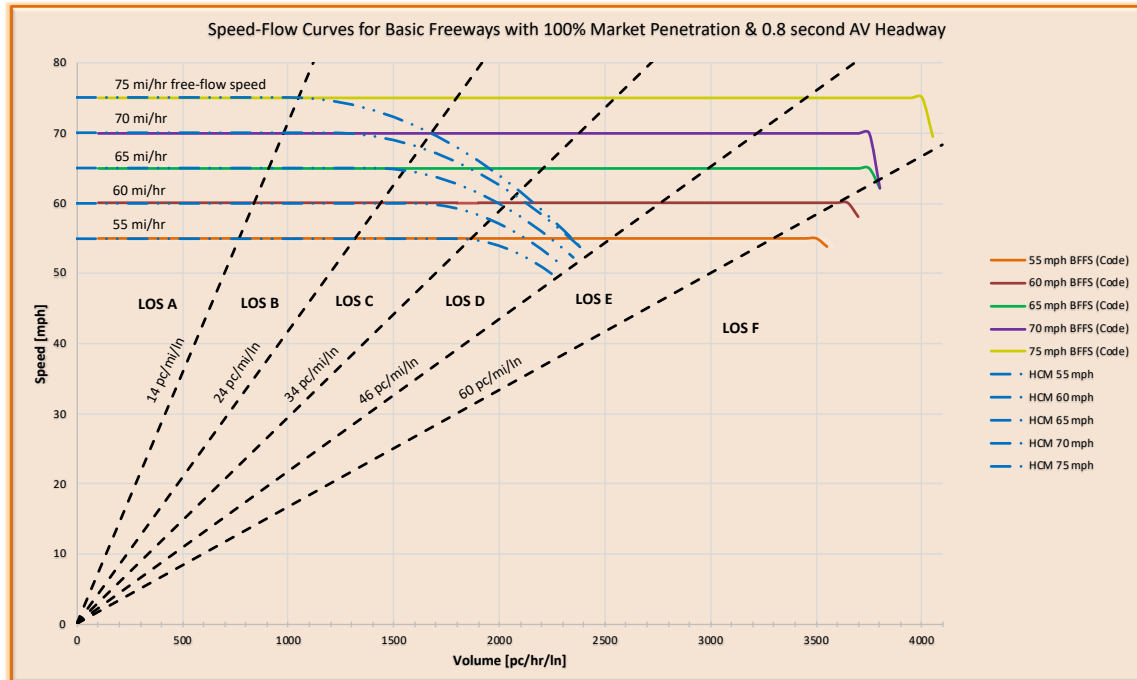*Figure 4- 13: Code Simulator Speed-Flow Curves at 100% Market Penetration and 0.8 second CAV Time Headway*

The 60% CAV market penetration with CAV time headway of 0.5 seconds plot was as follows:

Figure 4- 14: Code Simulator Speed-Flow Curves at 100% Market Penetration and 0.5 second CAV Time Headway
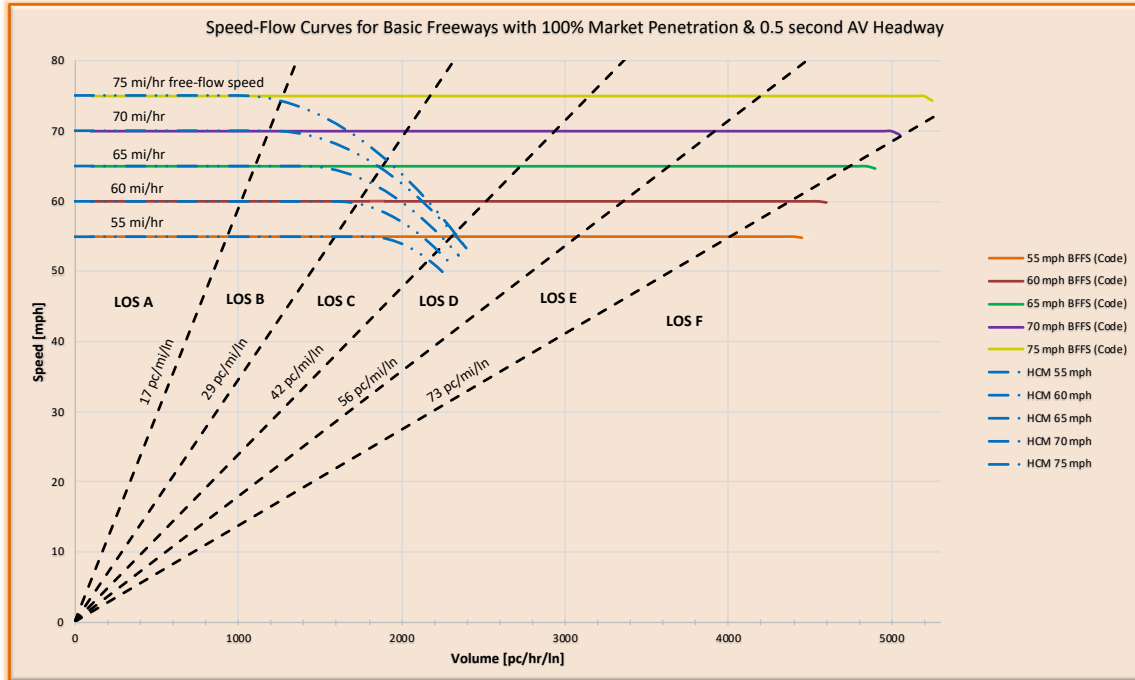
The LOS constant density line increase by an estimated 11, 15, & 28 passenger cars per mile per lane for the 1.0, 0.8, and 0.5 second CAV headway based on the LOS E threshold when compared to the 0% CAV market penetration scenario, respectively.

As previously observed by the earlier experiments, the speed-flow curves for the 100% market penetration were observed to behave in a constant manner throughout the entire range of flow rates up to the capacity of the basic freeway. Results from the 1 second time headway demonstrated an abrupt change in speed of approximately 5 mph in a very small amount of change between volumes (in the range of 10 vehicles per hour).

The 0.5 second time headway curves reveal an almost indistinguishable decay in speed as the freeway gets to its capacity. As previously mentioned, the data suggests that the connectivity between CAVS may produce a traffic stream that will experience immediate congestion without a gradual decrease in speed range.

4.2.9 Capacity and Level of Service Thresholds for Full CAV Market Penetration

A summary of the new capacities obtained from the 100% market penetration simulation and varying CAV time headways were tabulated and shown below:

Table 4- 4: Density Thresholds at 100% Market Penetration and 1.0, 0.8, & 0.5 second Time Headways

Density Thresholds at 100% CAV Market Penetration					
CAV Headway Time [seconds]	LOS A [pc/mi/ln]	LOS B [pc/mi/ln]	LOS C [pc/mi/ln]	LOS D [pc/mi/ln]	LOS E [pc/mi/ln]
1.0	13	22	32	43	56
0.8	14	24	34	46	60
0.5	17	29	42	56	73

4.2.10 Summary of Capacities with Respect to Market Penetration and Time Headways

A summary of the calculated capacities obtained from the varying market penetrations and time headways was tabulated with respect to FFSs. The following table shows the new estimated capacities with increasing market penetrations at varying headway times per the analysis obtained from the MATLAB code simulator.

Table 4- 5: Capacities with Respect to Variable Market Penetration and Time Headways at Different Free Flow Speeds

| | Basic Freeway Capacities with Varying Market Penetration at Different Headway Times [pc/hr/ln] | | | | | | | | | | | |
| | 0% MP | 40% MP | | | 60% MP | | | 80% MP | | | 100% MP | | |
FFS	Avg 1.6 sec Hdwy	1 sec Hdwy	0.8 Hdwy	0.5 Hdwy	1 sec Hdwy	0.8 Hdwy	0.5 Hdwy	1 sec Hdwy	0.8 Hdwy	0.5 Hdwy	1 sec Hdwy	0.8 Hdwy	0.5 Hdwy
55	2250	2550	2650	2850	2700	2900	3200	2900	3200	3700	3150	3550	4450
60	2300	2600	2700	2900	2800	2950	3350	3000	3300	3850	3250	3700	4600
65	2350	2650	2750	3000	2850	3050	3450	3100	3400	4000	3350	3800	4900
70	2400	2750	2850	3050	2900	3100	3500	3150	3500	4150	3450	3800	5050
75	2400	2750	2850	3050	2950	3200	3550	3200	3500	4200	3500	4050	5250

5.0 Conclusion

The study set out to find the implications of connected and autonomous vehicles (CAVs) within a basic freeway segment composed of a mixture of CAVs and human-driven vehicles (HVs). The first questions that were asked was whether basic freeways would experience a capacity increase by the addition of CAVs into the traffic stream and by how much. From the gathered data, the study in effect does give evidence that capacities actually do increase with higher market penetrations of CAVs. Average HV perception-reaction times were found to be close to 1.6 seconds, based on the empirical data provided by the Highway Capacity Manual. By allowing CAV time headways to be lowered between the ranges of 1.0 seconds to 0.5 seconds, the traffic stream experienced greater capacity values at each FFS scenario. Data showed that capacities increased by around 300 to 450 pc/hr/ln at only 40% market penetration and 1.0 second headways. The capacity changes are further increased if the CAV penetration is allowed to reach minimum time headways of 0.5 seconds, approximately a 600 to 650 pc/hr/ln jump from 0% CAV inclusion. As the market penetration proportions increase, so do the capacities of each individual free-flow speed scenario. The simulations ran by the code simulator showed changes in capacities that range from as low as 300 pc/hr/ln at 40 % penetration at 1.0 second time headways to as high as 2850 pc/hr/ln at the extreme case of full CAV penetration at 0.5 second time headways.

An additional behavior was observed from the characteristics followed by the curves as they approached their respective capacities. The higher the CAV market penetration applied, the smoother and flatter the curves became. This behavior was indicative of a more abrupt and immediate change in speed as the traffic streams reached their capacities. It was, therefore, assumed that CAVs would experience an immediate

halt at capacity rather than experience a parabolic decaying change as is demonstrated by the Highway Capacity Manual's speed distribution formulas.

The third question was related to the level of service (LOS) of basic freeway segments and how these would be affected by the introduction of CAVs. By demonstrating that the capacities would indeed increase with lower headway times, the LOS thresholds were compared and modified against the capacity trend at different free flow speeds. New LOS thresholds were calculated with respect to the new capacities observed for each scenario, and a constant density line with LOS E was adjusted to the trend of the capacity values. The new LOS values were tabulated with respect to each market penetration and individual time headways that showed an overall increase in LOS thresholds.

The study aimed to investigate whether CAV inclusion into a traffic stream could essentially increase a basic freeway segment's capacity. By allowing shorter time headways and greater market penetration of CAVs, the study has set forth a starting point for further research in the topic of freeway capacity with market penetration of connected and autonomous vehicles.

Appendix

Figure A- 1 MATLAB Code

```matlab
clear all

% INPUTS
Vh_input = 100;
market_penetration = 0;
AV_PIJR = 0.8;
kj = 100;
delta_t = (3600/(Vh_input*10));

% PERCEPTION REACTION TIME/ HEADWAY TIMES
my_mean = 1.60;
my_threshold = 0.5;
my_num_stnd_dev = 3.0;
my_stnd = my_threshold/my_num_stnd_dev;

% VEHICLE LENGTH
my_mean_length = 15;
my_mean_length_threshold = 2;
my_num_stnd_dev_length = 3.0;
my_stnd_length = my_mean_length_threshold/my_num_stnd_dev_length;

% INITIAL SPEED
my_mean_initialspeed = 55;
my_mean_initialspeed_threshold = 5;
my_num_stnd_dev_initialspeed = 3.0;
my_stnd_initialspeed = my_mean_initialspeed_threshold /
my_num_stnd_dev_initialspeed;

 % MARKET PENETRATION
for i = 1:Vh_input
Vh_pijr(i) = normrnd(my_mean,my_stnd);
random_gen = rand;
if random_gen <= (market_penetration/100)
    Vh_pijr(i) = AV_PIJR;
end

% NORMAL DISTRIBUTION OF VEHICLE LENGTH AND INITIAL SPEEDS
Vh_length(i) = round(normrnd(my_mean_length,my_stnd_length),1);
Vh_initialspeed(i) =
round(normrnd(my_mean_initialspeed,my_stnd_initialspeed),1);
End
```

```matlab
% TIME STEPS FOR ALL VEHICLE INPUTS AND INITIAL VEHICLE SPEEDS
for time = 1:Vh_input*10
Vh_Speed(:,time) = Vh_initialspeed;
end

% INITIALIZE VEHICLE POSITIONS AND PITT CAR-FOLLOWING MODEL
Vh_POS = zeros(Vh_input, Vh_input*10);
for time = 1:Vh_input*10 - 1
for i = 1:floor((time-1)/10)+1
Vh_POS(i, time + 1) = Vh_POS(i,time) + 1.47*Vh_Speed(i, time)*delta_t;
if i == 1
Vh_Speed(i, time+1) = my_mean_initialspeed;
else
Vh_Speed(i, time+1) =max(min(((Vh_POS(i-1,time) - Vh_POS(i,time)) -
(((1/(kj))*5280)-my_mean_length))/Vh_pijr(i),my_mean_initialspeed),0);
end
end
end

% SETTING A DETECTOR LOCATION AND SPEED DETECTOR MATRIX
detector_loc = 100;
for i = 1:Vh_input
for j = 1:Vh_input*10
if Vh_POS(i,j) >= detector_loc
loc_temp = Vh_POS(i,j);
loc_temp2 = Vh_POS(i,j-1);
speed_temp = Vh_Speed(i,j);
speed_temp2 = Vh_Speed(i,j-1);
Speed_store(i) = (speed_temp-speed_temp2)/(loc_temp-loc_temp2)*(detector_loc
- loc_temp2)+speed_temp2;
break
end
end
end

% OUTPUTTING AVERAGE SPEEDS FROM SPEED DETECTOR MATRIX
mean (Speed_store)
```

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