AUBURN UNIVERSITY

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Final Report
Project Number 930-923

APPLICATION OF THE NATURALISTIC DRIVING STUDY DATASET TO IMPROVE DESIGN GUIDES & ASSOCIATED PRACTICES

Submitted to

Alabama Department of Transportation

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JANUARY 2022

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1. Report No.	2. Government Accession No.	3. Recipient Catalog No.
4. Title and Subtitle Application of the Naturalistic Driving Study Dataset to Improve Design Guides & Associated Practices		**5. Report Date** January 2022 **6. Performing Organization Code**
7. Author(s) Huaguo Zhou, Dan Xu, Beijia Zhang, Humayra Kabir Priyanka, and Rod Turochy		**8. Performing Organization Report No.** ALDOT 930-923
9. Performing Organization Name and Address Highway Research Center Department of Civil and Environmental Engineering 238 Harbert Engineering Center Auburn, AL 36849		**10. Work Unit No. (TRAIS)** **11. Contract or Grant No.** **FHWA/ALDOT 930-923**
12. Sponsoring Agency Name and Address Alabama Department of Transportation 1409 Coliseum Boulevard Montgomery, AL 36130		**13. Type of Report and Period Covered:** Technical Report **14. Sponsoring Agency Code**

15. Supplementary Notes

Project performed in collaboration with the Alabama Department of Transportation.

16. Abstract

This research project explored two innovative applications of Second Strategic Highway Research Program (SHRP 2) Naturalistic Driving Study (NDS) data for studying driver behaviors at unsignalized intersections and freeway diverge areas. For the unsignalized intersection study, the driver visual workload, driver speed change, and stopping behavior were analyzed. For the freeway diverge area study, the speed change distribution and driver brake behavior were analyzed. Based on the results, a new method was developed to determine the minimum length of the freeway deceleration lane.

The first part of the study on unsignalized intersection focused on two types of traffic movements: direct left turns from the minor road and right-turn followed by U-turn movements at restricted crossing U-turn (RCUT) intersections. A total of 430 left-turn trips and 40 right-turn followed by U-turn trips were collected from the SHRP 2 NDS database. The entropy rate of each trip was calculated as an indicator of the driver visual workload and was treated as the dependent variable for the statistical analysis. The statistical analysis results indicated that drivers at the RCUT intersections had less random scanning and longer average fixation and spent more than 70% of time looking forward during the entire movement. Additionally, the study found that younger drivers and drivers at intersections with higher AADT (≥ 30,000) had higher overall average entropy rates.

A new method was developed to determine the minimum length of the deceleration lane at freeway diverge area based on the naturalistic driving speed distribution. The results indicated that a deceleration lane might not be required for serving a decelerating purpose when the ramp length is more than 1,550 ft. This number is specific to the diamond interchange with a stop controlled off-ramp terminal. For off-ramps with relatively high volume, the queue length should be considered. The study also found that drivers were not effectively using the deceleration lane for the speed reduction. The advisory speeds posted on off-ramps had no significant impact on drivers' naturalistic driving speeds.

17. Key Words: Naturalistic Driving Study, Driver Behavior, Freeway Diverge Area, Unsignalized Intersection, Speed		**18. Distribution Statement** No restrictions	
19. Security Classification (of this report) Unclassified	**20. Security Classification (of this page)** Unclassified	**21. No. of Pages** 112	**22. Price** None.

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January 2022

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Acknowledgement

The authors would like to thank the Project Advisory Committee (Steve Walker, John Michael Walker, Kerry NeSmith, and Kristy Harris) for their guidance through the entire project period. The team would also like to thank Virginia Tech Transportation Institute (VTTI) for helping with the data collection and arranging in-person visit to extract the driver behavior data. The following graduate students at Auburn University helped with data collection: Quan Li, Qing Chang, Yukun Song, and Caleb Rogers.

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1 INTRODUCTION

The Second Strategic Highway Research Program (SHRP 2) as developed new and comprehensive Naturalistic Driving Study (NDS) data about what happens in the vehicle during every trip taken by 3,147 volunteer drivers (ages 16-90+) over three-year period. SHRP 2 new data consists of two large databases: the NDS database and the roadway information database (RID) (FHWA 2021). Through video cameras and other recording devices, the NDS compiled an unprecedented amount of data about actual driver behavior. The data includes detailed video of the driver and the roadway, as well as data on the vehicles' speed, acceleration, braking, and other maneuvers. NDS trip data can be linked to roadway data from the RID, such as the roadway location, curvature, grade, lane widths, and intersection characteristics. These two databases can support innovative research leading to new insights into the current highway design guides and associated practices.

The purpose of this project is to explore the ways to use NDS and RID datasets to address specific needs that the Alabama Department of Transportation (ALDOT) has. More specifically, this research will delve into two specific research topics: driver behaviors at unsignalized intersections on rural divided highways and freeway diverge areas.

1.1 Research Questions

1.1.1 Application of NDS for Improving Unsignalized Intersection Design

According to the Federal Highway Administration (FHWA) publication on highway statistics, there are 90,716 miles of rural principal arterials in the United States (FHWA, Highway Statistics Series 2020). These highways tend to have wide medians (>30 ft) that provide the safety benefits for a relatively large degree of separation of the opposing direction of traffic. Median openings on rural divided highways, sometimes referred to as expressways with partial access control, provide some of the greatest potentials for frequent and severe crashes on the highway systems. A study conducted by the National Cooperative Highway Research Program (NCHRP) revealed that 30-40 percent of all rural divided highway crashes were intersection-related crashes (Maze et al. 2010). These high-speed facilities afford greater access to adjoining property owners than full-access-controlled freeways and are much less expensive to construct and maintain. However, since these facilities are not designed and operate as freeways, intersections are predominantly at-grade. At-grade intersections on divided highways with wide medians in rural settings have the potential for severe crashes due to the numerous conflict points and high speeds at these locations. With the lack of conclusive recommendations on treatments to improve the safety at median openings on rural divided highways, the wide variety of design elements and features that exist among them, and the propensity for severe crashes in these scenarios, there is a need for study of these locations.

The purpose of this study is to identify the driver behaviors that may contribute to crashes or near-crash events at unsignalized intersections on rural divided highways with relatively wide medians. The key sources of information in this effort include the new NDS and RID datasets at these locations. Two specific research questions are proposed to pursue: (1) What are the driver workloads for two types of maneuvers: direct left turn from the minor road, and right-turn followed by U-turn at the downstream intersections; and (2) How do the median geometric elements and the traffic control devices (TCDs) affect driver performance. The research results can provide valuable insights and guidance on geometric design and TCDs for updating the current practice by ALDOT and guidelines in national manuals, such as *A Policy on Geometric Design of Highways and Streets* (the *Green Book*) published by the American Association of State Highway and Transportation Officials (AASHTO) and the Manual on Uniform Traffic Control Devices (MUTCD), published by the FHWA.

1.1.2 Application of NDS for Improving Freeway Diverge Area Design

Freeways are essential components in the highway system that are designed under the highest design standards. In the United States, the interstate highway system constitutes only 2% of the nation's total rural lane miles, yet it conveys 25% of the annual rural vehicle miles traveled (VMT) (Pisarski and Reno 2015). Similarly, the urban interstate with under 4% of lane miles carries 24% of urban VMT (Pisarski and Reno 2015). Freeways are controlled access multilane divided facilities for safer high-speed operation of automobiles through the exclusion of at-grade junctions. Freeway diverge areas, including deceleration lanes and off-ramps, are critical elements that provide exits for traffic from freeway mainline segments via off-ramps to adjacent crossroads. The design intent of freeway diverge areas is to provide drivers with an effective, safe, and smooth transition from high-speed mainline to low-speed off-ramps and crossroads.

As early as the 1960s, the operational and safety performance at freeway diverge areas had raised the attention of the public and the transportation agencies. The California Department of Public Works conducted a three-year study of 722 freeway ramps with 1,643 crashes to investigate the impact of ramp geometric features on crash rates. It was found that the crash rates on exit ramps were consistently higher than those on entrance ramps and the largest percentages of exit ramp crashes occurred on the deceleration lane (Lundy 1965). In the 1970s, the Highway Users Federation for Safety and Mobility investigated the relationship between interchange design features and traffic safety. It was claimed that crashes are more frequent and severe at interchanges (Oppenlander and Dawson 1970). A 1980s study suggested to upgrade and rehabilitate freeway interchanges and ramps to guarantee capacity, efficiency, and safety (Harwood and Graham 1983). A more recent study in Virginia (McCartt, Northrup and Retting 2004) examined a sample of 1,150 crashes that occurred on heavily traveled urban interstate ramps and found that about half of all crashes occurred when drivers were exiting interstates in the diverge areas, 36% occurred when drivers were entering, and 16% occurred at the ramp terminal areas. The study recommended increasing ramp design speed, using surveillance systems to alert drivers, and extending the length of speed-change lanes. According to the NCHRP report 730 (Torbic, et al. 2012), the average crash rate at freeway deceleration lanes was 0.68 crashes per million vehicle miles traveled (MVMT), which is three times higher than crashes on acceleration lanes (0.16 MVMT) and 15% higher than crash rates at freeway mainline sections near exit ramps (0.59 MVMT). Furthermore, 42% of the crashes that occurred at freeway deceleration lanes were rear-end crashes resulting from speed differential. In Alabama, 74% of total freeway deceleration lane crashes occurred at diverge areas when drivers were existing interstates and 71% were rear-end crashes from 2012 to 2016 (Critical Analysis Reporting Environment (CARE) (Computer Software) 2018).

Previous studies revealed that crash rates could be related to the deceleration lane length (Cirillo 1970, Chen, et al. 2009, Bauer and Harwood 1998, Bared, Giering and Warren 1999, Lord and Bonneson 2005). Referring to the deceleration lane design, three aspects that determine the deceleration lane length are recommended by the AASHTO *Green Book* (AASHTO 2018), including (1) the speed at the beginning of the auxiliary lane; (2) the speed at the end of the deceleration lane; and (3) the manner of deceleration. Additionally, it requires the consideration of the speed differential between the mainline and the ramp. However, the *Green Book* only provides the minimum lengths of deceleration lanes according to the design speed differential between the freeway mainline and off-ramp. Moreover, the method to determine the minimum length remained the same between the latest version (AASHTO 2018) and the 1965 edition. Data used in both editions were collected in the 1930s.

1.2 Summary

The SHRP 2 NDS data has been used in many safety studies on driver behaviors. Few past studies focused on its application on studying highway geometric design and traffic operations. Past studies heavily relied on roadside data collection methods (e.g., radar gun, camera) to collect vehicle speed and trajectories data. These data contain little information about the driver. To fill this gap, a new approach using the SHRP 2 NDS data has been developed to investigate the driver behaviors during daily trips through unobtrusive data gathering equipment and without experimental control (Van-Schagen, et al. 2011). This study explored two innovative applications of NDS data on investigating the impact of geometric design features and TCDs on driver behaviors at the unsignalized intersections and freeway diverge areas.

This report summarizes the research activities and findings for the project funded by ALDOT on the application of the NDS dataset to improve design guides & associated practices. Chapter 2 summarizes the literature review results on past SHRP 2 NDS studies, and design guidelines on unsignalized intersections and freeway diverge areas. Chapter 3 discusses the NDS data collection and reduction methods. Chapter 4 provides the study site description and data analysis methods. Chapter 5 summarizes the analysis results of speed distribution and driver behaviors. Chapter 6 provides conclusions and recommendations on improving the design practices for unsignalized intersections and freeway deceleration lane.

2 LITERATURE REVIEW

2.1 SHRP 2 NDS Data Overview

The purpose of SHRP 2 was to identify strategic solutions to three national transportation challenges: improving highway safety, reducing congestion, and improving methods for renewing roads and bridges (The National Academies of Sciences, Engineering, and Medicine 2020). Extensive data collection was conducted to achieve the goals of SHRP 2, which offers a unique opportunity to address different research questions that were not able to study before. To fulfill the critical gap in data about driver behavior, the SHRP 2 Safety Program conducted the most comprehensive NDS that collected large-scale data from six states, including Florida, Indiana, New York, North Carolina, Pennsylvania, and Washington (Strategic Highway Research Program 2014). This section summarizes details of the NDS background, how to access the dataset, and various applications.

2.1.1 Background

SHRP 2 NDS aims at improving safety and reliability for motorists and providing answers to key traffic- and safety-related questions (Dingus, et al. 2015). It involves understanding how the driver interacts with and adapts to the vehicle, environmental condition, roadway geometric characteristics, and TCDs (Campbell 2012). More than 3,000 volunteer drivers participated in this study with their everyday or "natural" driving behavior recorded. During three years of data collection, over five million trips with nearly 50 million miles of driving were monitored. The volunteer drivers were distributed with similar numbers of males and females in all age groups, as presented in **Figure 1**. Six data collection areas were selected to represent a mix of road types and weather conditions. The participants were recruited through call centers and traditional methods (Campbell 2012).

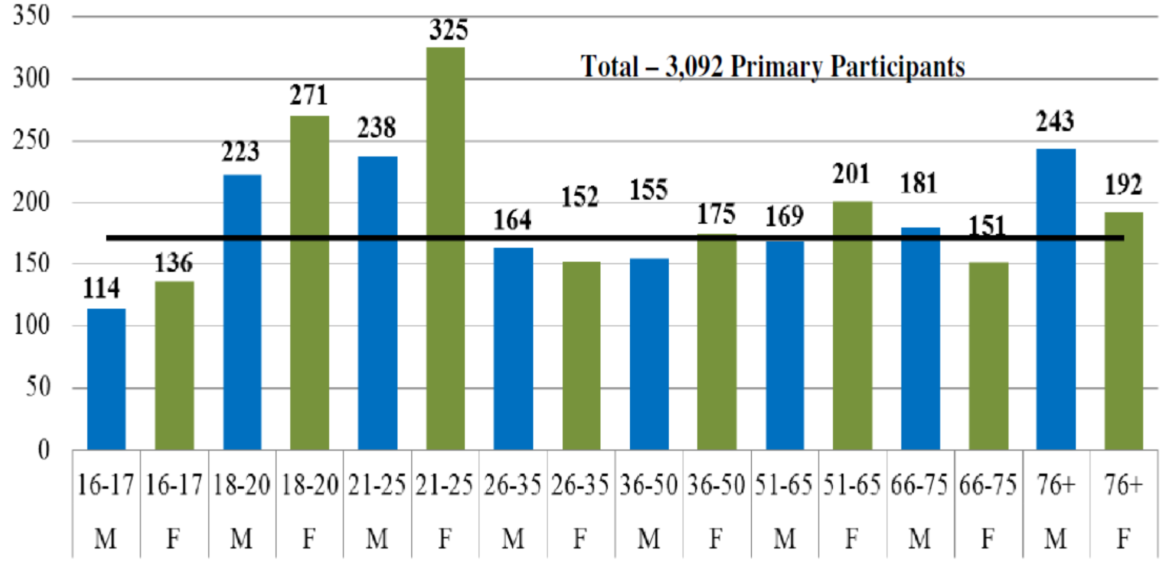
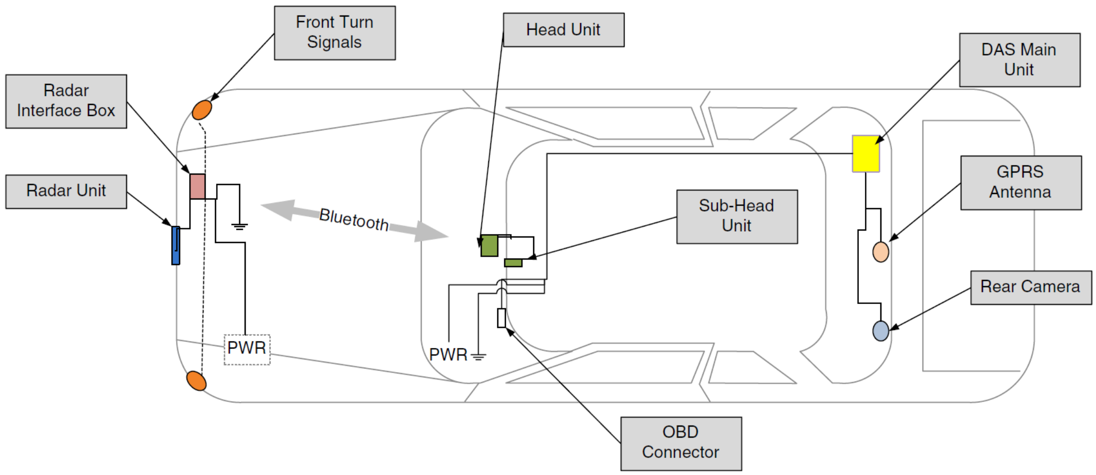


Figure 1 Primary participants enrolled in NDS by age and gender (Dingus, et al. 2015). (Blue = male; Green = female)

Virginia Tech Transportation Institute (VTTI) developed the Data Acquisition System (DAS) to collect and maintain data of all trips made during the study period (Campbell 2012). The DAS was manufactured by American Computer Development, Inc., which includes forward radar, four video cameras, accelerometers, vehicle network information, Geographic Positioning System (GPS), on-board computer vision lane tracking plus other computer vision algorithms, and data storage capability (Dingus, et al. 2015). **Figure 2** shows the schematic view and key components of the DAS used in the data collection process. As demonstrated in **Figure 3**, the participant vehicle was equipped with forward view (upper left), driver and left side view (upper right), instrument panel view (lower left), and rear and right view (lower right) cameras to record both the in-vehicle and out-of-vehicle environment. Data were continuously recorded while the participant's vehicle is operating. The study resulted in the successful collection of 2 petabytes (2 million gigabytes) of real-world driving video and sensor data (Strategic Highway Research Program 2014).



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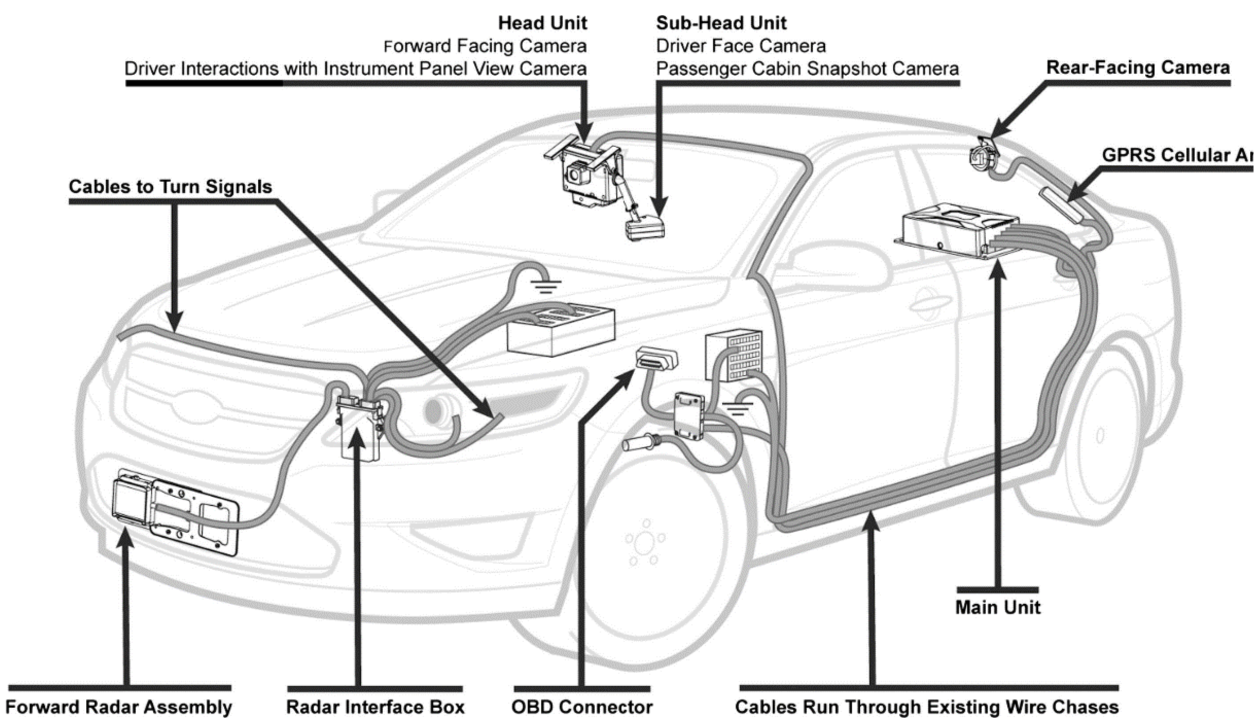
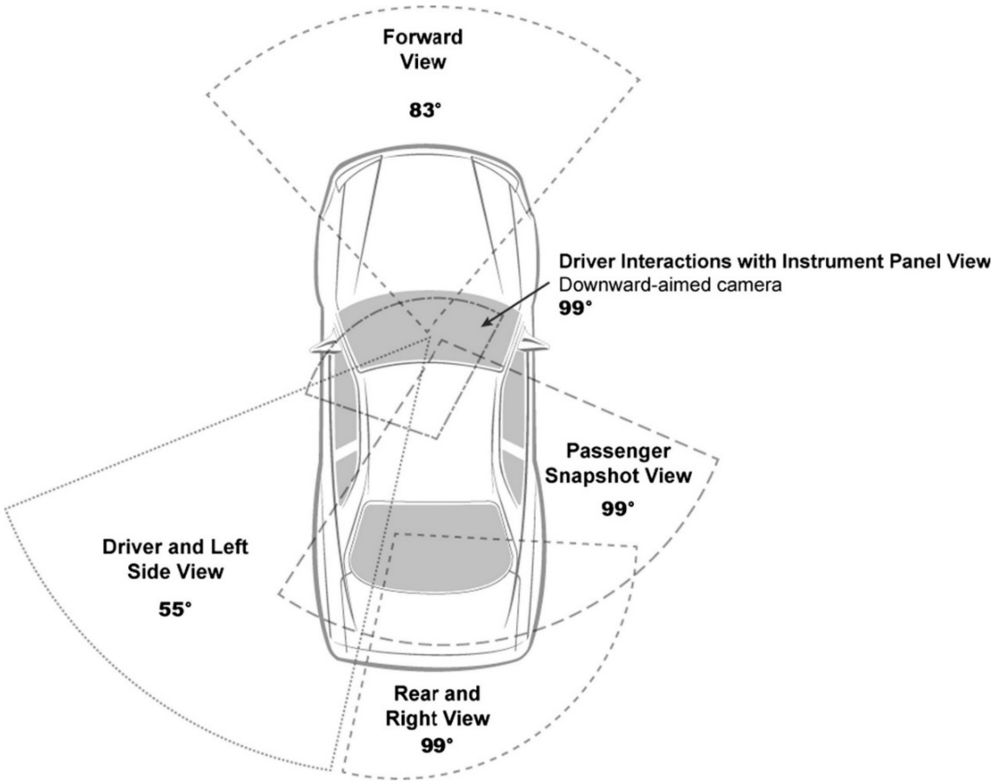


(*b*)

Figure 2 Installed DAS schematic: (a) top view diagram of DAS components (Dingus, et al. 2015); and (b) side view diagram of DAS components (Antin, et al. 2019)



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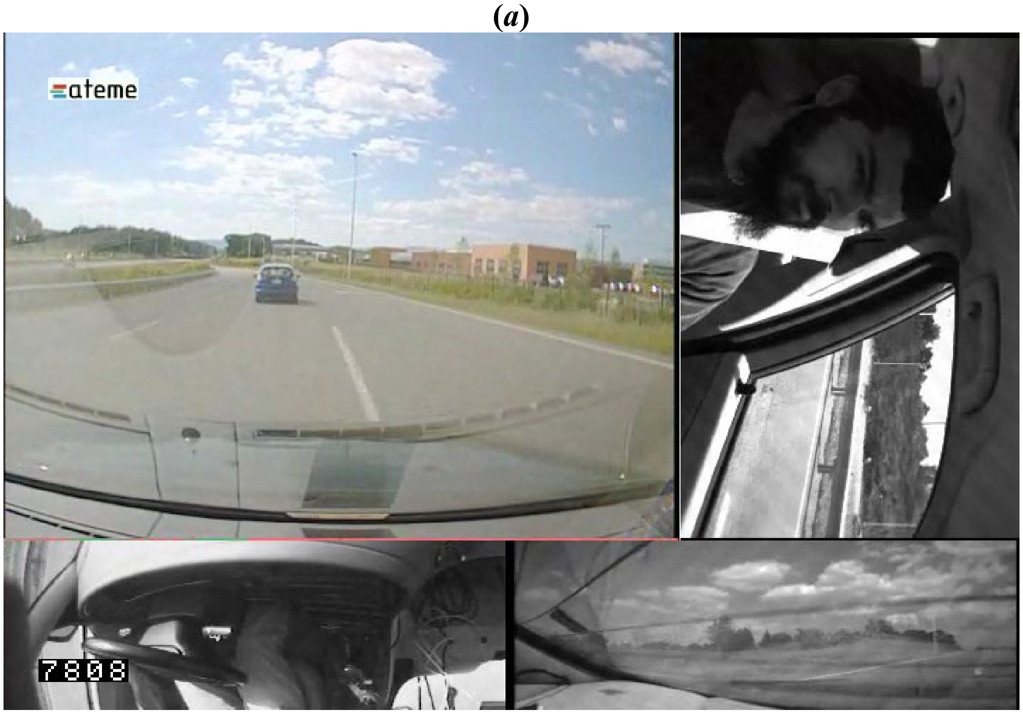


(*b*)

Figure 3 Video camera views: (*a*) fields of view for the DAS (Antin, et al. 2019); (*b*) quad image of four video camera views (Dingus, et al. 2015)

The NDS adheres to appropriate informed consent and privacy requirements as it deals with human subjects (Campbell 2012), which has been approved by the Institutional Review Board (IRB) for the National Academies of Science (Dingus, et al. 2015). The IRB, also known as Independent Ethics Committee (IEC), Ethical Review Board (ERB), Research Ethics Board (REB), etc., is an administrative body established to protect the rights and welfare of human research subjects recruited to participate in research activities conducted under the auspices of the institution with which it is affiliated (U.S. Food & Drug Administration 2019). The data were protected from the moment they were collected through migration from vehicle to the final research repository (Dingus, et al. 2015). Human subjects' protection in the NDS required secure usage of Personally Identifying Information (PII), which includes any data that could potentially be used to identify a particular person. As NDS data collected driver face video, GPS traces that might contain the participant's home, work location, or school, etc., a Certificate of Confidentiality was secured from the National Institute of Mental Health (NIMH) to protect PII data collected during the data collection period, so that the researchers and study sponsors cannot be forced to disclose information that may identify any participants (Dingus, et al. 2015). Nonidentifying and deidentified data are allowed to be widely shared, and the data was encrypted from the moment it was collected. Only qualified researchers can access the PII through a Secure Data Enclave (SDE), which is a physically isolated environment that restricts data access and protects the PII (Dingus, et al. 2015). Thus, the NDS data were divided into two portions (InSight and InDepth) with regard to their nature.

2.1.2 InSight and InDepth

The InSight Website (https://insight.shrp2nds.us/) contains a subset of NDS data that is publicly available. Any registered researchers who had successfully taken the IRB training can extract this type of data through InSight Website. The InSight data are divided into four categories: vehicle, driver, trip, and event. **Table 1** summarizes the information under each category. The data was either directly captured by the DAS during data collection period, or through questionnaire surveys. A query builder is provided on the website to select variables and conditions, submit query, assess results, build cross tabulations, view graphs of output and table of individual records. This allows for preliminary analysis of aggregated data for further request and analysis. The data on the InSight Website has been extracted and coded through manual review of the videos by VTTI in the SDE that can only be viewed without any sort of extraction or export of data. Unique identifiers were developed for each event, trip, driver, and vehicle to allow for efficient PII protection and easy linkages among them to perform analysis. A driver may have multiple trips and events associated, and a trip may consist of several events.

Table 1 Summary of InSight data categories

Vehicle	Vehicle type (car, truck, van, etc.)
	Vehicle age and condition
	Amount of data collected per vehicle
	Quantities of vehicles installed
	Vehicle technologies and equipment
Driver	Number of participating drivers
	Amount of data collected per driver
	Driver demographics and driving history
	Driver physical and psychological state
	Driver participation experience
Trip	Summary measures describing trips
	Trip length, duration, start time, stop time
	Min, max, mean for speed, acceleration
	Trip summary record table
	Trip density map
Event	Crash, near crash, and baseline event record
	Event by type and severity
	Event viewer

The second portion of NDS data is known as InDepth, which includes information that may potentially result in identifying the participants. These data contain time-series data and video data, which are not available online (InSight).

2.1.3 NDS Data Applications on Transportation

Past NDS data applications in transportation paid more attention on safety aspects, as the intent of NDS is to address traffic safety-related questions (Dingus, et al. 2015). A study investigated the changes in driving behavior, before, during, and after near-crash events on freeways by applying NDS data to identify the driving patterns (Ali, Ahmed and Yang 2020). Victor et al. conducted a study to determine the relationship between driver inattention and crash risk in lead-vehicle pre-crash scenarios by utilizing NDS data (Victor, et al. 2015). The results of this study were reported to support distraction policy, regulation, and guidelines; improve intelligent vehicle safety systems, and teach safe glance behaviors. Wu and Lin explored driver perception time prior to the occurrence of a safety-related event by using NDS data (Wu and Lin 2019). They analyzed a total of 1,417 rear-end crashes and near-crashes and reported that critical driving situations, driving environment, and driver behavior are influential factors in explaining the variation of driver perception times in safety-related events. Hao et al. performed an in-depth investigation of crashes involving roadway objects and animals based on 2,689 events data (Hao, et al. 2020). The results indicated that driver errors, involvement of secondary tasks, roadway characteristics, lighting condition, and pavement surface condition are significant factors that contributed to the occurrence and increased severity outcomes of crashes.

NDS data can also be used to evaluate the effects of geometric design features and TCDs. A study conducted by Hallmark et al. evaluated driving behavior on rural two-lane highways to propose appropriate countermeasures for mitigating crash rates on rural horizontal curves (Hallmark, et al. 2015). The results suggested that better curve delineation with delineation countermeasures would allow drivers to better gauge upcoming changes in roadway geometry with better speed selection and decreased risk of encroachment. Wu and Xu analyzed right-turn driver behavior at signalized intersections (Wu and Xu 2017). This research revealed that drivers have higher acceleration and lower observation frequency under Right-Turn-On-Red (RTOR) controlled intersections. It suggested that the implementation of traffic safety countermeasures at signalized intersections is necessary to reduce right-turn crashes.

There have been some studies focused on the impacts of adverse weather conditions on driver behaviors. A group of researchers from the University of Wyoming investigated the effect of adverse weather on driver speed selection by using SHRP 2 NDS data (Khan, Das and Ahmed 2020). They suggested that weather-specific distribution should be used to model driver behavior more representatively in microsimulation platforms. By employing NDS data, another study showed how the driver compensated differently according to weather conditions to avoid the crash event and provided a discrimination threshold between normal and risky driving patterns in both rainy and clear weather conditions (Ali, Ahmed and Yang 2020). The same group of researchers also explored the impacts of heavy rain on speed and headway selections. They compared driver behavior in clear and heavy rain conditions using the trips with the same driver, same vehicle, and same traversed routes. The study concluded that drivers were more likely to reduce their speed by more than 5 kilometers per hour below the speed limits in heavy rain than in light rain (Ahmed and Ghasemzadeh 2018).

NDS data can help with Connected Vehicle (CV) application development as well. A study utilized trajectory analysis and unsupervised machine learning techniques to identify normal and risky driving patterns based on vehicle kinematics data from NDS (Ali, Ahmed and Yang 2020).

It stated that the identification of these patterns can distinguish between different driving patterns in a CV environment using basic safety messages.

2.2 Research on Unsignalized Intersection with Wide Medians

2.2.1 Current Design Guidelines

There have been many guidelines on the traffic design or access control at unsignalized intersections with wide median openings. The MUTCD (2009) provides the guidance **(Figure 4)** that where divided highways are separated by median widths at the median opening itself of 30 feet or more, median openings should be signed as two separate intersections. The ONE-WAY signs, double yellow line and stop bars are suggested to be installed at the intersection. The National Committee on Uniform Traffic Control Devices (NCUTCD) suggests divided highway crossings with median widths between 30 ft, and 85 ft may function as either one or two intersections depending upon the interaction of the opposing left-turn vehicle paths and the available interior storage in the median for a crossing vehicle, as shown in **Figure 5.** For crossings treated as two intersections, it suggests removing the bullet-nose, installing two stop lines at the median opening, and using a double yellow line at the middle to separate the traffic movements from opposite directions. The stop sign, yield sign, and one-way sign are also suggested.

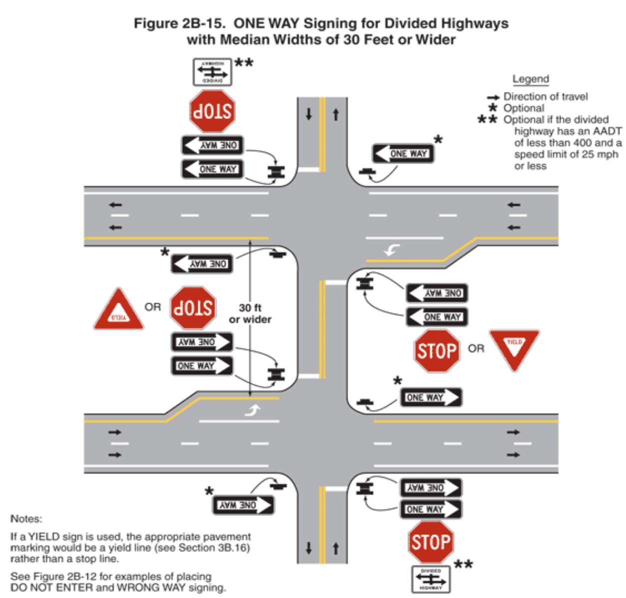


Figure 4 2009 edition MUTCD guidance on TCD design of divided highways with medians of 30 feet or wider

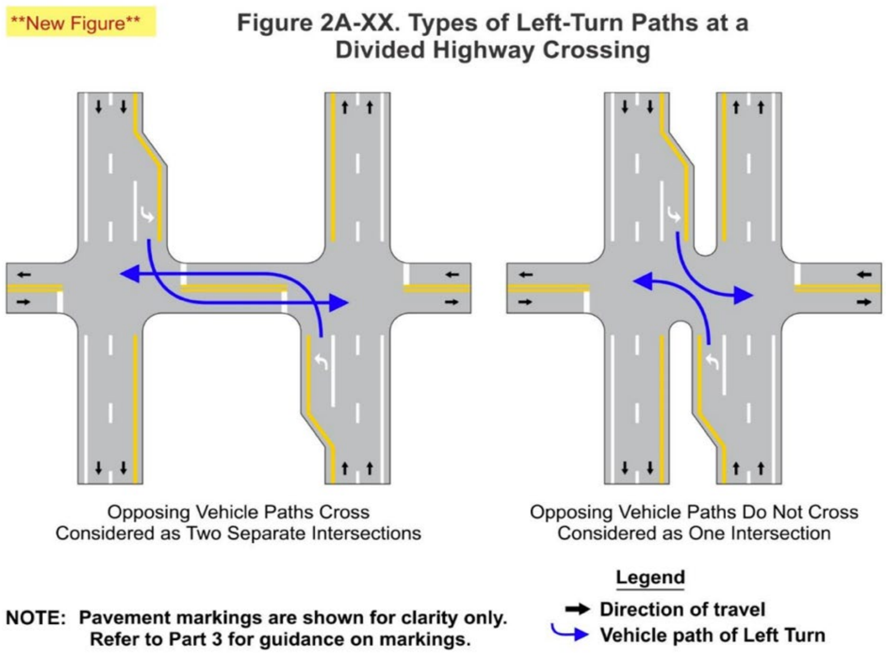


Figure 5 NCUTCD recommended treatments for divided highways with medians of 30 ft or wider (NUCTCD)

Three relevant NCHRP projects provided guidelines on unsignalized intersection design. NCHRP Report 650 summarized the current design guidance and recommended revision. **Figure 6** shows the recommended countermeasure matrices for two-way stop-controlled (TWSC) rural expressway intersections. The report also suggested improving the current design guide (Maze et al. 2010). NCHRP Report 500 suggested providing a double yellow line at the median opening of a divided highway to avoid the side-by-side queuing and angle stopping within the median area (Neuman et al. 2003). NCHRP Report 375 suggested that opposing left-turn drivers leaving the expressway tend to turn in front of one another (i.e., simultaneous left-turns) when the median width is 50 feet or less but tend to turn behind one another (i.e., interlocking left-turns) when the median width is greater than 50 ft, as shown in **Figure 7**. There is some other literature related to median designs (Qi et al. 2012**;** Stamatiadis et al. 2009; and Dissanayake et al. 2003)), however, they mainly focused on median openings on urban or suburban highways.

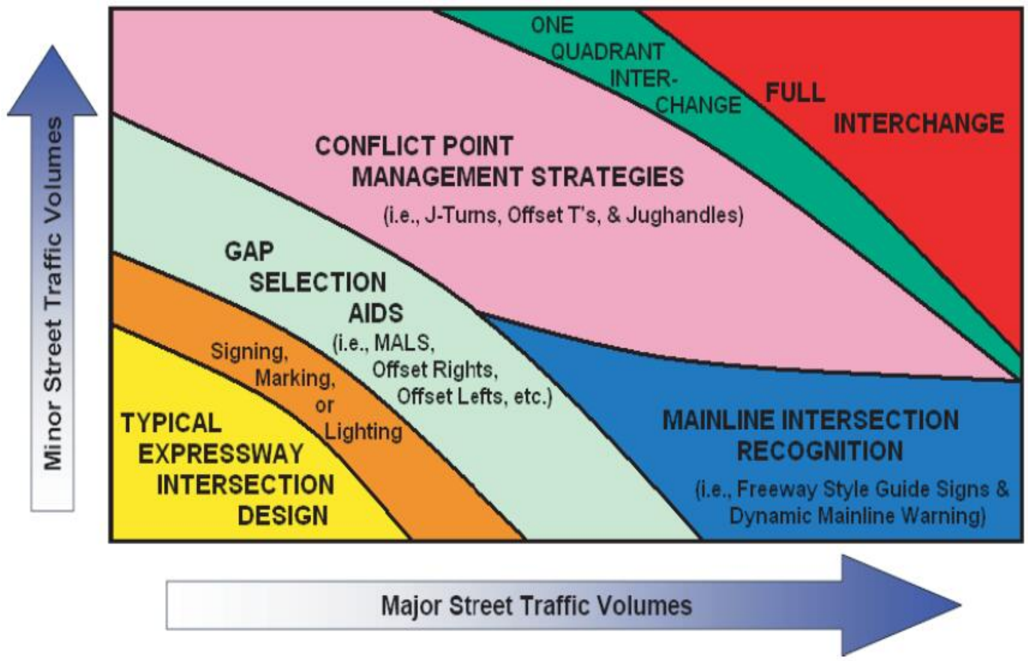


Figure 6 TWSC rural expressway intersection countermeasure matrices (NCHRP Report 650 2010)

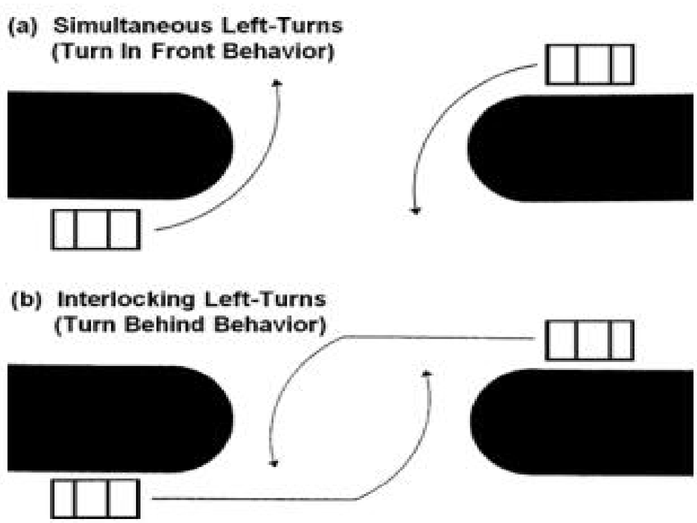


Figure 7 Two types of opposing left-turn behavior at median openings (NCHRP Report 375 1995)

State transportation agencies have developed their own guidelines for median opening design. Florida Department of Transportation (FDOT) has a comprehensive median design handbook that provides guidelines on safety improvements at unsignalized intersections (FDOT 2008). Besides the treatments for wide median openings based on MUTCD, this median handbook suggests using vehicle actuated flashing beacons for two-stage crossing, especially when an extraordinarily wide median results in an increased observance of accidents occurring at the far end of the intersection. Along with the continuous flashing beacons on the existing stop signs of the intersecting roadway, it recommended that loop sensors can be placed within the median to activate flashing red beacons on the 2^{nd} set of stop signs as well as flashing yellow beacons in advance of the intersection on the major roadway (FDOT 2014). Some other states, like Minnesota, developed design guidelines on rural intersection conflict warning systems and design guides for roundabout and other alternative intersections, such as *Restricted Crossing U-Turn (RCUT) Intersection* (MnDOT 2016). Additionally, FHWA (2014) published a *Manual for Selecting Safety Improvements on High-Risk Rural Roads*. It includes 31 selected countermeasures for unsignalized intersections. Some have the crash modification factors (CMFs), performance ratings and estimated costs.

2.2.2 Driver Behavior Studies at Unsignalized Intersections

2.2.2.1 SHRP 2 NDS-based Studies

A few studies on driver behaviors have recently been conducted based on SHRP 2 NDS data since 2016. Oneyear et al. (2016) compared the driver braking behavior at the rural stop-controlled intersections with different TCDs by using the SHRP 2 NDS data. They developed a linear regression model and found overhead flashing beacons and pavement marking increased the distance at which the driver began braking. Dinakar et al. (2019) studied driver responses in left turn across the path from opposite direction (LTAPOD) crash and near crash events at signalized intersections by comparing the driver brake behavior, second task, age and perception-reaction time. LTAPOD scenario involves two vehicles initially traveling in opposite directions, and one of the vehicles turns left across the path of the other straight moving vehicle. The statistical test results showed that the drivers responded significantly faster when subjected to shorter time to near-crash events compared to longer ones. Other shorter reaction times at near crash events included when the turning vehicle did not stop before entering the intersection or when the turning vehicle was visible for a short duration. But factors such as age, gender or secondary task engagement did not significantly influence response times. Lv et al. (2019) studied the influence of different factors, such as road geometry, environmental factors, and traffic conditions, on right-turn distracted driving behavior at intersections by using the logistic model and random forest. They found that vehicle lane occupied and traffic control are significantly correlated to right turn distracted driving behaviors.

2.2.2.2 Other Driver Behavior Studies at Intersections

Researchers started to study driver behavior at intersections before SHRP2 NDS data was collected. Some common driver behaviors were studied at the intersections, including eye-glance patterns (Kim et al. 2018, Romoser 2008); reaction times according to driver age and mental workload (Makishita et al. 2008); stopping behavior (Muttart et al. 2011); and abnormal trajectory (Zhang 2017). Kim et al. (2018) compared different driver distractions at intersections in car following models based on driver's eye glance behavior from 100-car NDS database by using decision tree analysis. Romoser et al. (2008) studied the glance patterns of older and younger drivers while approaching and entering the intersection with no medians. They compared the average amount of time spent in each region and found older adults are more likely to remain fixated on their intended path of travel and look less randomly than younger drivers. Muttart et al. (2011) compared the glancing and stopping behavior of motorcyclists and car drivers at intersections, and repeated-measure analysis of variance was utilized to test the effects of the two modes. They found motorcyclists were less likely to come to a complete stop and frequently failed to make proper glances. Zhang et al. (2017) studied the factors affecting the paths of left-turning vehicles from minor road approaches at unsignalized intersections by observing vehicle trajectories. Six different trajectories were identified. The statistical analysis results implied that higher vehicle speed on major road and less minor road lanes can cause more abnormal trajectories for left turns from minor road. A further review of past literature on eye-glance behavior studies and stopping behavior for two-stage left turns was conducted for this project since they are directly related to the proposed research question. The below is the summary of the review results.

Eye-glance Behavior Studies

Traditional measurements of eye-glance features, such as duration and frequency, have been used by many studies as an indicator of drivers' visual workload (Romoser et al. 2013; Shaaban et al. 2017; Bao 2009; Victor et al. 2005; Engström et al. 2005). The quantitative metric of eye-glance behavior, entropy, was derived from the information theory. It is commonly used in the flight area of analyzing pilot visual workload since the year 1990 (Boer 2000; and Ellis 2009). Recently, several studies showed that the entropy rate can be a better approach to quantify visual workload in the driving domain when compared to mean glance duration (Gilland 2008; Bao 2009; and Wang et al. 2014). The entropy rate can provide measures on how drivers react to the visual locations. Wang et al. (2014) studied drivers' eye-glance patterns during distracted driving, and the entropy rate was calculated and used to assess the randomness associated with drivers' scanning patterns. Gilland (2008) suggested that the entropy metric proved to be more sensitive to attentional demands than all alternative visual metrics and it is useful for understanding the correlation between driver age and task-induced cognitive demands within the context of real-world driving. A larger entropy rate means larger randomness or higher scanning to various areas in shorter average fixation duration. Different studies hold different ideas on the relationship between randomness of glancing locations and workload. Hilburn (2004) suggested that an increase in mental workload will increase the randomness of the glancing locations, while research by Di Nocera (2006) inferred that as workload increases, the observed glancing locations will become less random. No past studies were found to use the entropy rate to quantify the driver workload at the conventional intersections and the alternative intersections (i.e., RCUT intersections).

Stopping Behavior Studies

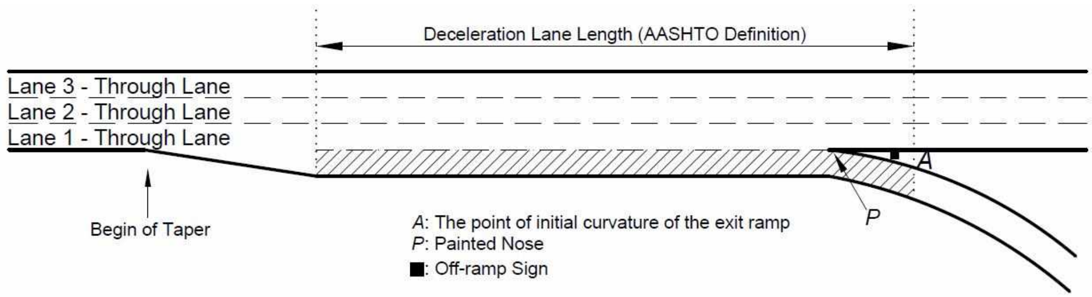
Two-stage left turn stopping behavior at TWSC intersections involves stopping at both minor road and median openings (California DMV 2020). Several past studies found that more than half of the drivers did not make complete stops at stop-controlled intersections (Cody 2013; Stockton 1981; Beaubien 1976). Crashes due to stop sign violation accounted for 60% to 70%, according to past studies (Moon Y.-J. 2009; Retting 2003). Therefore, driver stopping behavior analysis at a stop-controlled intersection is very critical from the safety point of view.

Studies have been conducted to identify factors that affect stopping behavior at a stop-controlled intersection. Woldeamanuel and McKelvie investigated the impact of the subjective definition of stop condition on the violation charged (Woldeamanuel 2012; McKelvie 1986). Shaaban and Kolarik found that approaching speed, driver age, gender, type of vehicle, time, weekdays/weekends, residential and commercial vehicles, sightline obstruction had a significant impact on the stopping behaviors (Shaaban 2017; Kolarik 2020). Past studies on driving behavior analysis at a stop-controlled intersection have been conducted mainly through manual observations, taking survey opinions and video-based trajectory (Shaaban 2017; McKelvie 1986; McKelvie and Schamer 1988; DeVeauuse and Kim 1999). Manual observation is hectic and problematic and video data cannot collect driver characteristics. The NDS data can overcome all these drawbacks. This study aims to evaluate the impact of driver socioeconomic and demographic factors on two-stage left turn stopping behavior using NDS data.

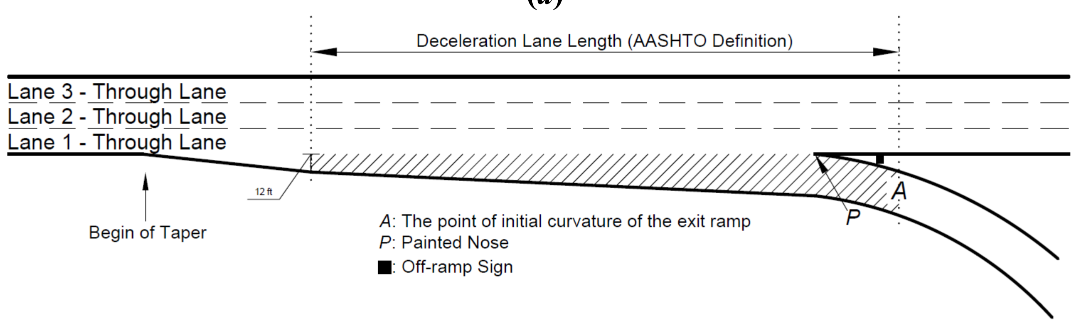
2.3 Research on Freeway Diverge Area Design

2.3.1 AASHTO Design Policy

A deceleration lane is a speed-change lane that intends to minimize conflicts between vehicles on the mainline and diverging area (AASHTO 2018; Bared, Giering and Warren 1999). There are two general forms of declaration lane (**Figure 8**): the parallel-design which has an added lane for changing speed and the tapered design which provides a direct exit at a flat angle (AASHTO 2018). The length of a deceleration lane is measured from the point of a 12-ft right-tapered wedge, or a 12-ft added parallel lane to the point of the exit ramp curvature beginning (AASHTO 2018). In practice, it is hard to control and measure the beginning of the exit ramp alignment. Thus, this study measured the deceleration lane length to the point of the physical gore (the painted nose).



(a)



(b)

Figure 8 Definition of deceleration lane length: (*a*) parallel-design deceleration lane; and (*b*) tapered-design deceleration lane

Equations 1 and 2 present the procedure of calculating the minimum deceleration lane length in the 1965 *Blue Book* (AASHTO 1965). AASHTO policies used basic two-step process for establishing design criteria (Torbic, et al. 2012). Deceleration is accomplished as follows: the driver removes his or her foot from the gas pedal, the vehicle slows in gear for a period of time (assumed to be 3 seconds) without a break, and then the driver applies the brake pedal and decelerates at a comfortable rate. The length is primarily determined by the speed differential between the average speed on the mainline and the off-ramp.

$$\boldsymbol{L_{Decel} = 1.47V_h t_n - 0.5d_n(t_n)^2 + \frac{(1.47V_r)^2 - (1.47V_a)^2}{2d_{wb}}} \qquad \textbf{(1)}$$

$$\boldsymbol{V_a = \frac{1.47V_h + d_n t_n}{1.47}} \qquad \textbf{(2)}$$

Where, L_{Decel} = Deceleration lane length, ft
V_h = Highway speed, mi/h
V_a = Speed after t_n seconds of deceleration without brakes, mi/h
V_r = Entering speed for controlling exit ramp curve, mi/h
t_n = Deceleration time without brakes (assumed to be 3 s), s
d_n = Deceleration rate without brakes, ft/s^2
d_{wb} = Deceleration rate with brakes, ft/s^2

Two assumptions were made during calculation (Fitzpatrick, Chrysler and Brewer 2012) that (1) most vehicles travel at the average speed instead of the design speed when traffic volumes are low (e.g., on a freeway with a 70 mi/h design speed, the assumption is that a driver will enter the auxiliary lane at 58 mi/h as presented in **Table 2**.); and (2) a 3s deceleration before braking is applied on the taper section, which results in two deceleration rates (d_n and d_{wb}).

Table 2 Minimum deceleration lane lengths for exit terminals with flat grades of less than 3%, adopted from AASHTO *Green Book* (2018)

Deceleration Lane Length, L_a (ft) for Design Speed of Controlling Feature on Ramp, V' (mph)										
Highway Design Speed, V (mph)	Diverge Speed, V_a (mph)	Stop Condition	15	20	25	30	35	40	45	50
		Average Running Speed at Controlling Feature on Ramp, V'_a (mph)								
		0	14	18	22	26	30	36	40	44
30	28	235	200	170	140	—	—	—	—	—
35	32	280	250	210	185	150	—	—	—	—
40	36	320	295	265	235	185	155	—	—	—
45	40	385	350	325	295	250	220	—	—	—
50	44	435	405	385	355	315	285	225	175	—
55	48	480	455	440	410	380	350	285	235	—
60	52	530	500	480	460	430	405	350	300	240
65	55	570	540	520	500	470	440	390	340	280
70	58	615	590	570	550	520	490	440	390	340
75	61	660	635	620	600	575	535	490	440	390
80	64	705	680	665	645	620	580	535	490	440

V = design speed of highway (mph)
V_a = average running speed on highway (i.e., diverge speed) (mph)
V' = design speed of controlling feature on ramp (mph)
V'_a = average running speed at controlling feature on ramp (mph)
L_a = deceleration lane length (ft)

The *Green Book* provides minimum requirements to the design of deceleration lengths for exit terminals. The AASHTO's deceleration lane lengths from **Table 2** in the *Green Book* 2018 edition were calculated from old studies which data was collected on 1930s (Fitzpatrick, Chrysler and Brewer 2012). The only difference between the 2004 *Green Book* and the 1965 *Blue Book*, regarding minimum lengths of freeway deceleration lanes, is that the taper length was included in the deceleration lane length in the 1965 *Blue Book* while being listed separately in the 2004 *Green Book*. Given that vehicle technology is being improved, and driver behaviors and driving patterns are being changed, these values need to be updated based on the current drivers' diverging behavior and vehicle braking mechanisms.

2.3.2 Safety and Operational Impacts of Freeway Deceleration Lane

If the deceleration distance was judged to be short, a harder deceleration would be applied by the driver and start on the mainline before the deceleration lane. When the following vehicles did not take proper actions, a rear-end crash would happen. Similarly, if the deceleration lane was too long, the diverging driver would be accelerating on the deceleration lane. When the leading vehicles on the deceleration lane is at a lower speed, and the diverging driver does not accommodate accordingly, a conflict or a collision may occur. These examples relate operational performance to safety. There have been numerous studies that explored safety and operational impact on the freeway deceleration lane.

FHWA conducted a study to model crash frequency on deceleration lanes (Bauer and Harwood 1998). The study applied the negative binomial regression in modeling crash frequency and the geometric design and traffic volume characteristics of ramps. This method has been widely used among researchers on the safety performance of freeway diverge area. There have been a number of studies utilized regression models to optimize the deceleration lane length and the configuration of off-ramps (Cirillo 1970, Chen, et al. 2009, Bauer and Harwood 1998, Bared, Giering and Warren 1999, Lord and Bonneson 2005). However, the results from past studies were inconsistent or sometimes contradictory.

Some studies suggested that increasing the deceleration lane length would reduce crash rates. A study conducted in 1970 identified the relationship of crashes to lengths of deceleration lanes

(Cirillo 1970). Data from 20 states were collected between 1950 to 1965 and 700 weaving areas were analyzed. The lengths of deceleration lane were categorized in less than 200 ft, between 200 ft and 299 ft, between 300 ft and 399 ft, …, and more than 700 ft. It was found that longer deceleration lane has few crashes with the percentage of diverging traffic being less than 6% of the mainline volume. The results also compared the benefits from additional lengths of deceleration lane and acceleration lane. It was concluded that the deceleration lane has less reduction in crashes compared to acceleration lane with the length increasing. This study used the crash rates in the analysis, which could be misleading as locations with lower traffic volumes would have high rates and vice versa. Similar to this study, Bared, Giering and Warren evaluated the safety performance of acceleration and deceleration lane lengths (Bared, Giering and Warren 1999). They developed crash predictive models for ramps from a sample of Interstate highways in Washington State, which contained 276 exit ramps equally located in rural and urban areas. The results of this study illustrated the importance of providing longer deceleration lane since deceleration lane has higher number of crashes than acceleration lane related to their lengths due to the higher complexity of the driver's tasks on deceleration lanes compared to acceleration lanes. Similar findings were achieved by Twomey et al., who identified that deceleration lane of 900 ft or more can reduce traffic friction on through lanes, therefore, reducing crash rates (Twomey, et al. 1993). Wang et al. also evaluated the impacts of various factors on injury severity at freeway diverge areas (Wang, Chen and Lu 2009). They collected crash data and roadway information from 231 freeway exit segments in Florida and applied partial proportional odds regression to predict injury severity. They found that the length of deceleration lanes is a significant factor affecting injury severity and concluded that a longer deceleration lane is more likely to reduce injury severity.

On the contrary, the other studies implied that increasing the deceleration lane length would increase crash occurrence. Garcia and Romero found that a long deceleration lane would encourage accelerating maneuver on the deceleration lane (Garcia and Romero 2006). The study found that some drivers initially made an acceleration maneuver on the long deceleration lane before starting brake. They also found that overtaking scenarios often occurred on the excessively long deceleration lane in order to precede the vehicle on the mainline, thus increasing crash risks at freeway diverge areas. These results are also consistent with other studies on this topic (Chen, et al. 2009, Chen, et al. 2011, Chen, Zhou and Lin 2014). They developed a crash predictive model to identify the factors that contribute to the crashes. The model revealed that the crash frequency increases with the lengthening of the deceleration lane for both left-side and right-side diverge areas. Their results indicated that the optimal deceleration lane length between 500 ft and 700 ft can significantly reduce the crash severity and delay for through traffic. These studies also suggested that when the deceleration lane is too long, crash frequencies and crash rates will start increasing. When considering different types of off-ramps, Lu et al. concluded that parallel-designed sites with a one-lane exit had the lowest crash frequency and crash rate (Lu, et al. 2010).

2.3.3 Diverge Maneuver

A series of decisions should be made when drivers proceed to exit a freeway. First, a satisfactory gap and a diverge point with appropriate diverge speeds must be selected. Second, drivers need to select a point to start decelerating with an initial deceleration rate. The deceleration rates often increase when approaching the gore area. The final speeds on ramps are determined by the controlling features (e.g., advisory speed limit, curvature, and control type at ramp terminals) on off-ramps.

The AASHTO *Green Book* provides a table on minimum lengths of freeway deceleration lane but does not offer a table on deceleration rates used to determine deceleration lane length (Torbic, et al. 2012). NCHRP report 730 provided the two methodologies to back-calculate deceleration rates used in the *Green Book* 2004 edition. One was using Equations 1 and 2, and the other using a constant deceleration approach (Torbic, et al. 2012). **Table 3** and **Table 4** summarized corresponding deceleration rates used in the *Green Book*. These deceleration rates were then compared with the rates measured from the field. It was found that field measured deceleration rates were lower than the derived AASHTO values.

Table 3 Corresponding deceleration rates for minimum deceleration lane lengths, adopted from AASHTO *Green Book* (2004)

Deceleration Rate (ft/s^2) for Design Speed of Controlling Feature on Ramp (mph)										
Highway Design Speed (mph)	Diverge Speed (mph)	Stop Condition	15	20	25	30	35	40	45	50
		Average Running Speed at Controlling Feature on Ramp (mph)								
		0	14	18	22	26	30	36	40	44
1st Deceleration Rates (ft/s^2) While Coasting in Gear used to Reproduce Deceleration lane Lengths										
30	28	-1.04	-1.04	-1.04	-1.04	—	—	—	—	—
35	32	-1.53	-1.53	-1.53	-1.53	-1.53	—	—	—	—
40	36	-1.52	-1.52	-1.52	-1.52	-1.52	-1.52	—	—	—
45	40	-2.01	-2.01	-2.01	-2.01	-2.01	-2.01	—	—	—
50	44	-2.51	-2.51	-2.51	-2.51	-2.51	-2.51	-2.51	-2.51	—
55	48	-2.01	-2.01	-2.01	-2.01	-2.01	-2.01	-2.01	-2.01	—
60	52	-2.98	-2.98	-2.98	-2.98	-2.98	-2.98	-2.98	-2.98	-2.98
65	55	-2.50	-2.50	-2.50	-2.50	-2.50	-2.50	-2.50	-2.50	-2.50
70	58	-2.50	-2.50	-2.50	-2.50	-2.50	-2.50	-2.50	-2.50	-2.50
75	61	-2.99	-2.99	-2.99	-2.99	-2.99	-2.99	-2.99	-2.99	-2.99
2nd Deceleration Rates (ft/s^2) While Braking used to Reproduce Deceleration lane Lengths										
30	28	-5.75	-6.42	-5.49	-8.97	—	—	—	—	—
35	32	-5.83	-5.75	-5.38	-6.68	-10.29	—	—	—	—
40	36	-5.66	-5.22	-5.11	-5.66	-5.95	-4.42	—	—	—
45	40	-6.38	-6.05	-5.68	-6.91	-6.83	-6.18	—	—	—
50	44	-6.74	-6.57	-6.28	-7.20	-7.86	-6.55	-6.23	N/A	—
55	48	-7.10	-7.08	-6.86	-7.55	-7.86	-7.46	-7.49	-7.86	—
60	52	-7.07	-7.19	-6.99	-7.45	-7.70	-7.34	-6.98	-7.43	-16.84
65	55	-7.55	-7.43	-7.27	-8.03	-7.76	-7.49	-7.51	-7.66	-9.60
70	58	-7.40	-7.41	-7.27	-7.77	-7.53	-7.60	-7.45	-7.54	-8.17
75	61	-7.76	-8.02	-7.90	-8.06	-7.85	-7.65	-7.70	-7.62	-8.49

Table 4 Corresponding deceleration rates for minimum deceleration lane lengths assuming a constant deceleration rate, adopted from AASHTO *Green Book* (2004)

Deceleration Rate (ft/s^2) for Design Speed of Controlling Feature on Ramp (mph)										
Highway Design Speed (mph)	Diverge Speed (mph)	Stop Condition	15	20	25	30	35	40	45	50
		Average Running Speed at Controlling Feature on Ramp (mph)								
		0	14	18	22	26	30	36	40	44
30	28	-3.59	-3.16	-2.91	-2.30	—	—	—	—	—
35	32	-3.93	-3.56	-3.59	-3.14	-2.50	—	—	—	—
40	36	-4.36	-4.01	-3.95	-3.72	-3.60	-2.75	—	—	—
45	40	-4.47	-4.31	-4.22	-4.07	-3.98	-3.42	—	—	—
50	44	-4.79	-4.62	-4.50	-4.40	-4.30	-3.91	-3.06	-2.07	—
55	48	-5.16	-4.98	-4.84	-4.77	-4.61	-4.31	-3.80	-3.22	—
60	52	-5.49	-5.39	-5.33	-5.19	-5.07	-4.79	-4.33	-3.96	-3.44
65	55	-5.71	-5.63	-5.59	-5.47	-5.38	-5.19	-4.77	-4.51	-4.18
70	58	-5.88	-5.78	-5.74	-5.63	-5.56	-5.41	-5.06	-4.86	-4.52
75	61	-6.06	-5.97	-5.89	-5.80	-5.70	-5.67	-5.32	-5.18	-4.92

Most past studies employed field observation method to study diverging driving behaviors on deceleration lanes and off-ramps. Garcia and Romero concluded that the drivers started to decelerate before exiting the mainline with a speed reduction of 10.5 mph even on a long deceleration lane (Garcia and Romero 2006). Another study found that vehicles that diverge early on the deceleration lane are likely to diverge at speeds that are close to freeway speeds while late diverging vehicles have lower diverging speeds (Torbic, et al. 2012). A recent study conducted by Ma et al. observed an average10 kmph (6.21 mph) speed reduction on the mainline before entering the deceleration lane at tapered-designed locations (Ma, et al. 2019).

Driving simulators were also used for this topic. Calvi did three driving simulator studies on diverging performance on deceleration lanes (Calvi, Benedetto and De Blasiis 2012, Calvi, Bella and D'Amico 2015, Calvi, et al. 2020). The first study simulated three different traffic scenarios to analyze driving performance while approaching a diverge area and decelerating during the exiting maneuver (Calvi, Benedetto and De Blasiis 2012). Thirty drivers were recruited to collect their lateral position, speed, and deceleration. This study revealed that lower traffic volumes result in higher existing speeds, higher average and maximum deceleration rates, and earlier braking on the mainline. The second study was conducted to analyze the effects of traffic flow and deceleration lane geometry on the driving performance of diverging drivers (Calvi, Bella and D'Amico 2015). This study recruited 31 volunteers in the experiments with parallel and tapered designed deceleration lanes under low and high traffic flow conditions. Findings from the second study indicated that the taper type of deceleration lane contributes to the significantly higher speed difference. Furthermore, lower traffic volumes lead to higher deceleration rates. The third study validated the driving simulator for use by designers in adopting the best solution for freeway acceleration and deceleration lanes (Calvi, et al. 2020). A total of 90 participants took part in the experiment. They were recorded in real and simulated scenarios using an instrumented vehicle and a driving simulator for driving performance of merging and diverging maneuvers. The authors compared the field and simulation data in terms of driving speeds and trajectories and validated the simulator usage. This study suggested that drivers significantly reduced their speeds before diverging from the mainline and entering the deceleration lane.

2.3.4 Related Research Utilizing NDS Data

Few studies on freeway diverge area utilized NDS data. Brewer and Stibbe used SHRP 2 NDS data to identify relationships between ramp design speed characteristics and drivers' choices of operating speeds on those ramps (Brewer and Stibbe 2019). The study results suggested that the type of traffic control at crossroad terminals had a larger effect on off-ramps speed selection. Recent research explored the lane-change behaviors in freeway off-ramp areas by utilizing Shanghai NDS data (Zhang, et al. 2018). The authors identified 433 lane-change events with trajectory data and applied the speed variance of the following vehicle on the deceleration lane as a safety surrogate index. This study did not provide practical insights into freeway diverge area design.

2.4 Summary of Literature Review

The literature review results indicated that SHRP 2 NDS data have not been applied to study driving behaviors at both unsignalized intersections and freeway diverge areas. As SHRP 2 NDS data consists of detailed information about the driver's interaction with the vehicle, the traffic environment, and roadway characteristics, it provides an opportunity to quantify the visual workload and stopping behaviors of minor road left turn drivers at unsignalized intersections and to determine minimum deceleration lane lengths based on naturalistic driving speeds and deceleration rates on freeway diverge areas.

3 NDS DATA COLLECTION AND REDUCTION

This section provides the detailed data collection and reduction methods for the two research areas: unsignalized intersections and freeway diverge areas. The data delivered for the study is a subset of the SHRP 2 NDS dataset, including video clips of the forward-view and rear-view videos, corresponding time-series report for each trip, driver risk perception, driver demographics, and vehicle information. The time-series report contains speeds (km/h), acceleration-deceleration rates (*g*), the brake pedal status (0 or 1), etc. **Table 5** provides the data dictionary in time-series reports. The time-series report of each traversal provides all data at 0.1-s intervals. By reviewing the forward-view videos, which were taken from cameras mounted inside the vehicles to provide drivers' views, the traffic condition (free-flow or non-free flow), environmental condition (lighting and weather), roadway geometric features, and the presence of TCDs can be collected for further analysis.

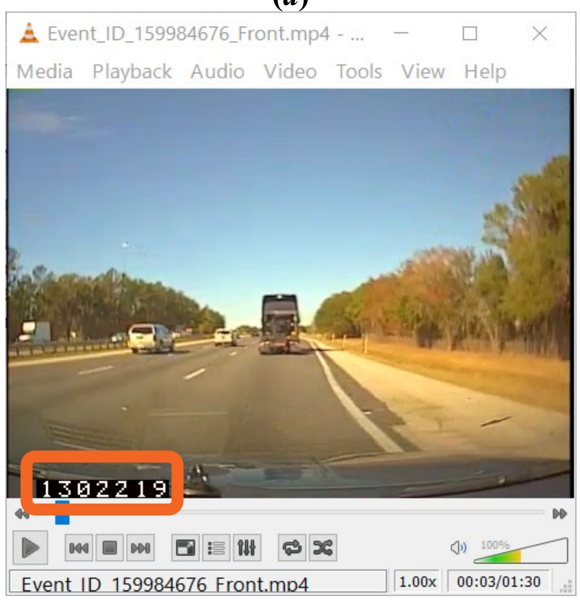
Table 5 NDS time-series data dictionary

Variable Name	Description
vtti_timestamp	Time since the beginning of trip, in milliseconds
vtti_speed_network	Vehicle speed indicated on speedometer collected from network, in km/h
vtti_accel_x	Vehicle acceleration in the longitudinal direction versus time, in *g*
vtti_pedal_brake_state	On or off press of brake pedal, 0 = off; 1 = on

Figure 9 illustrates an example of a time-series report and the forward-view video. The continuous timestamp is provided on both of them at 0.1-s intervals. The information, such as the vehicle speed from the speedometer, the longitudinal acceleration rate, and the brake pedal status, in the time-series report, can be verified by reviewing forward-view video.

System.Time_Stamp	vtti.timestamp	vtti.file_id	vtti.speed_network	vtti.speed_gps	vtti.accel_x	vtti.accel_y	vtti.pedal_brake_state	vtti.gyro_z	vtti.video_frame
22	1301200	8003352	102.93		-0.0493	0.0145	0	0	19468
23	1301300	8003352	102.71		-0.0174	0.0174		0	19469
24	1301400	8003352	102.48	115.3326	-0.0348	-0.0116	0	0	19471
25	1301500	8003352	102.31		-0.0493	0.0058	0	0	19472
26	1301600	8003352	102.15		-0.0377	0.0522	0		19474
27	1301700	8003352	101.81		-0.0145	0.0638	0	0.975586	19475
28	1301800	8003352	101.75		-0.0464	0.0957	1	1.625977	19477
29	1301900	8003352	101.41		-0.0435	0.116	1	1.951172	19478
30	1302000	8003352	101.3		-0.0435	0.1015	1	2.276367	19480
31	1302100	8003352	100.8		-0.0174	0.1189	1	2.276367	19481
32	1302200	8003352	100.57		-0.0551	0.1189	1	1.951172	19483
33	1302300	8003352	99.95		-0.0812	0.0957	1	1.625977	19484
34	1302400	8003352	99.73	110.666	-0.0957	0.0609	1	0.650391	19486
35	1302500	8003352	99.11		-0.0841	0.0406	1	0.325195	19487
36	1302600	8003352	98.94		-0.087	0.029	1		19489

(*a*)



(*b*)

Figure 9 NDS example data: (*a*) time-series report; and (*b*) forward-view video

Driver demographics and driver risk perception information was also requested. Driver demographics information mainly includes driver gender, age, and education levels. The driver risk perception score was estimated based on a survey questionnaire designed to gauge the participant's perception of dangerous or unsafe driving behaviors or scenarios (Transportation Research Board 2013). The questionnaire includes 32 driving-behavior-related questions, which indicated their perceptions of risk associated with different driving behaviors. For example, how would the participant evaluate the risk when not yielding the right-of-way, the participants' associated risk with passing other cars on the right side or the shoulder of the road, the participants' associated risk with turning without signaling, etc. Each question was assigned a score from 1 (No Greater Risk) to 7 (Much Greater Risk); thus, a higher score indicates that the driver self-reported to be cautious and obedient to traffic rules with greater risk perceptions. The total risk perception score of a driver is the sum of all the scores from questions in the questionnaire that ranges from 32 to 224.

3.1 Unsignalized Intersections

The NDS data collection for unsignalized intersections includes three steps: (1) selecting the study locations; (2) collecting eye-glancing data; and (3) collecting stopping behavior data.

Step 1: Selecting the study locations

To select the unsignalized intersection study locations, the researchers first used Google Maps to identify the locations with the required geometric design features, and then check the traversal density map (**Figure 10)** from the SHRP 2 NDS database to select the intersections with minimum number of trips (>= 30 trips). **Figure 11** shows the two study scenarios, one at a conventional intersection and the other at a RCUT intersection. The study trips start when drivers begin to decelerate at the minor-road and end when drivers approach a stable speed on the major-road. The study facility types are four lane divided highways with wide median in rural or suburban areas. The major-road speed limit ranges from 45 to 55 mph.

There are more trips at the conventional intersections with wide median in NDS database than the RCUT intersections. Researchers found a total of 636 direct left turn trips at conventional intersections and 577 right-turn followed by U-turn trips at the RCUT intersections based on the density maps at the beginning. After further reviewing the video clips, only 470 trips in total meet the study requirements, including 430 direct left turn trips and 40 right-turn followed by U-turn trips. All trips were in Florida or North Carolina.

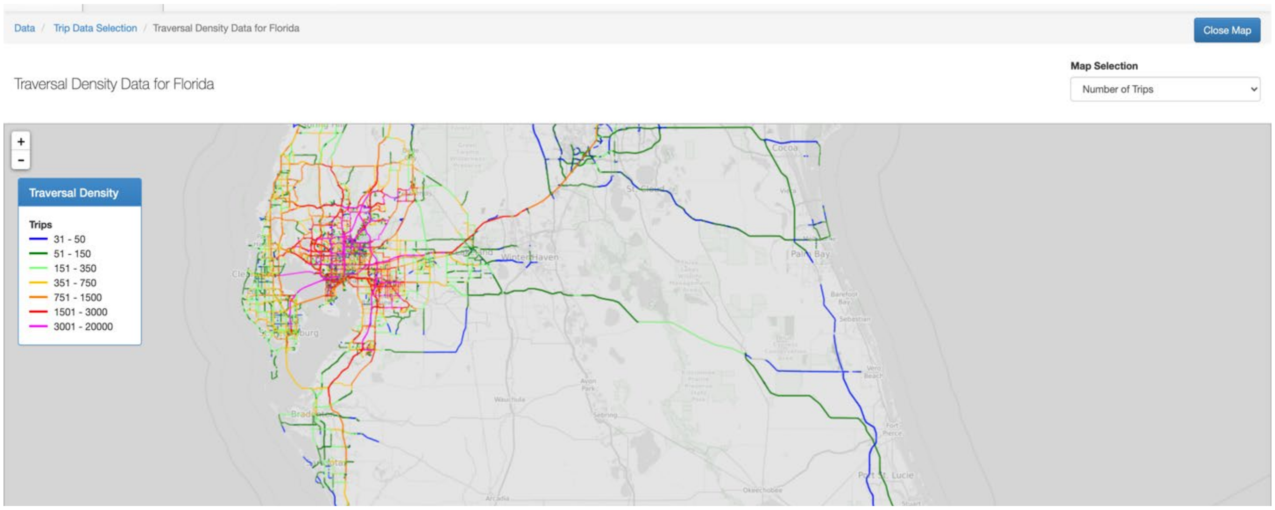


Figure 10 Example of traversal density map from the SHRP 2 NDS database

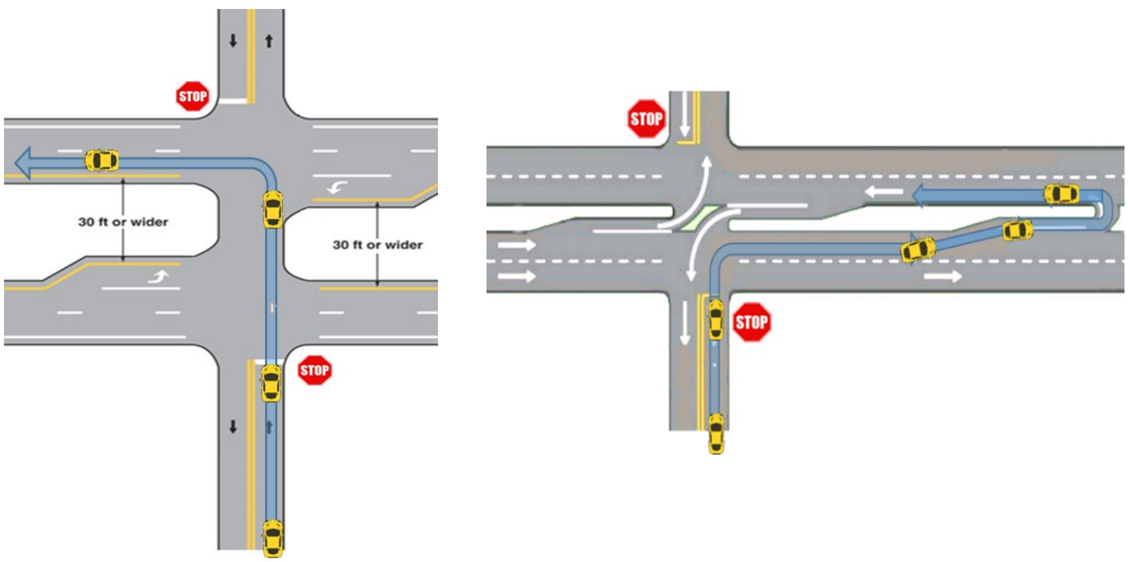


Figure 11 Scenario one: direct left turn at a conventional intersection (Left); scenario two: right-turn followed by U-turn movement at a RCUT intersection (Right)

Step 2: Collecting driver eye-glance data

To collect driver eye-glance data, four researchers requested permission to watch the drivers' face videos in the VTTI Secure Data Enclave. The driver eye-glance annotation tool (**Figure 12)** was used to manually code the defined eye-glancing areas frame by frame. As listed in **Figure 12**, there are a total of 12 defined eye-glancing areas, such as the left or right-side mirrors, the windshield, over the shoulder, passenger, cell phone, interior object, center stack, eye off-road, rearview mirror, etc. Only eight eye-glancing areas were used in studying workload of minor-road left turns or right-turn followed by U-turn movements. The trip videos last from 40 to 120 seconds. Each one-second video contains about 14 frames.

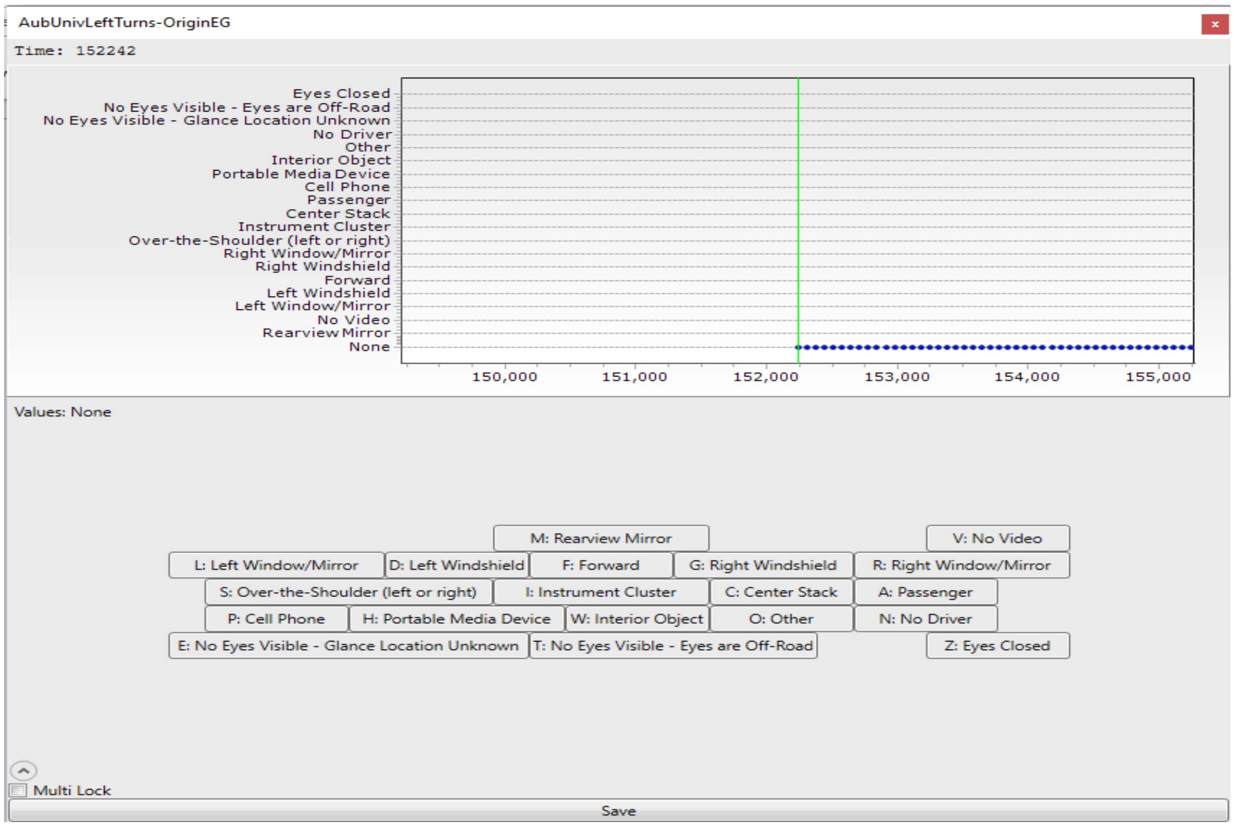


Figure 12 An example of the driver eyeglance annotation tool interface

Step 3: Collecting stopping behavior data

Using the time series reports and forward view videos, the stopping condition of the vehicle was recorded as a value of 1 for a stopped condition or a value of 0 for not stopping condition. While recording the stopping behavior, it was observed that at the minor road, some vehicles stopped in a queue and then started to move following the preceding vehicle without stopping at the stop sign. **Figure 13** illustrates this phenomenon. In this study, only stopping behaviors by the drivers not impacted by a preceding vehicle in a queue was studied. Finally, a total of 428 trips taken by 53 participants were recorded for stopping behavior analysis. All the participant-related data were merged with the stopping behavior data. The participant-related data contains information of driver age, gender, education, driving experiences, etc. The drivers/participants' characteristics were broken down into socioeconomic factors and driving-related factors. The detailed data description is contained in APPENDIX A. Overall, the sample data has a 50/50 distribution in gender and reasonable distributions in age groups and other categories.

Figure 13 Time frames of a forward view video showing the effect of a preceding vehicle

3.2 Freeway Diverge Areas

For freeway diverge areas, the diverge area of diamond interchanges with relatively straight off-ramps were targeted because diamond interchanges are the most widely used service interchange, which consists of 79% of all interchanges in the United States (Missouri DOT 2017). The number of trips and drivers available at each potential site was checked from "Traversal Density Data" on the Insight Website (https://insight.shrp2nds.us/) to ensure a relatively large sample size on the off-ramp and the freeway mainline. The sites were checked to collect their geometric features, such as taper length, deceleration lane length and type, off-ramp length, divergence angle, and off-ramp controlling feature (e.g., advisory speed limit, curvature, type of control at ramp terminals, etc.). The sites with large number of trips in the same state were selected to minimize the design differences and other impact factors (weather and population factor). In this study, ten freeway diverge areas were selected from Florida.

The original dataset contained 971 trips from 10 locations. The video of each trip was reviewed to ensure that it is a complete traversal beginning before the deceleration lane and ending after the off-ramp terminal. The trips with incomplete time-series reports and trips began after the deceleration lane or ended before the terminal were excluded from the further analysis. Finally, 709 complete trips driven by 272 unique drivers were used for analysis in this study.

4 METHODOLOGY

This section provides detailed information on data analysis methods for driver behavior analysis at both the unsignalized intersections and the freeway diverge areas. First, the study sites description was provided for two types of locations. For the speed analysis, the polynomial regression and the critical speed changepoint detection were introduced. Then, for the unsignalized intersections study, the methods of driver visual workload analysis (the descriptive data analysis and the entropy rate) and stopping behavior analysis were included. For the freeway diverge area study, the method of determining the length of the freeway deceleration lane was introduced.

4.1 Site Description

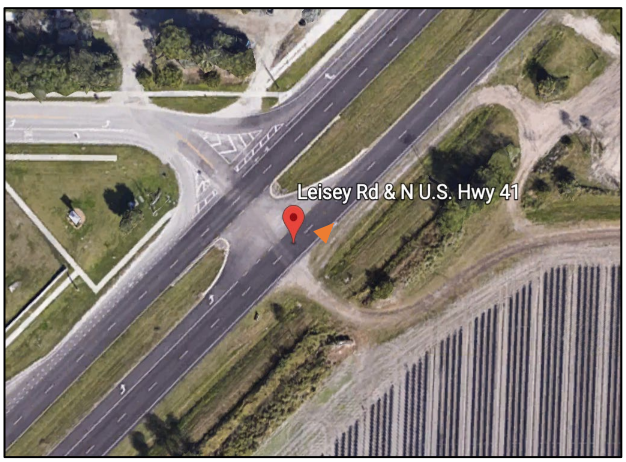
4.1.1 Unsignalized Intersection Study Sites

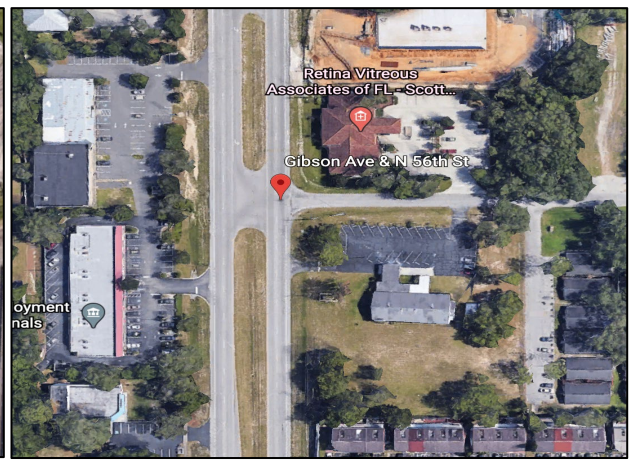
Table 6 contains the information of the six selected conventional intersections in Florida**,** including location, median width, speed limit, number of trips and drivers**.** The aerial photos of the selected conventional intersections are presented in **Figure 14**. **Table 7** lists the information on RCUT trips at eight study sites in Florida and North Carolina, including location, median width, speed limit, number of trips and drivers.

Table 6 Information on the selected conventional intersections in Florida

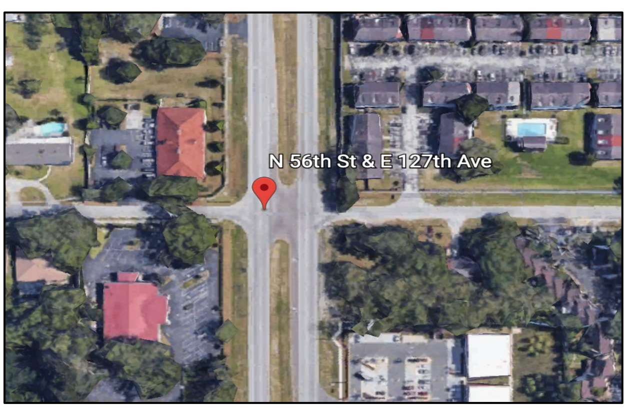
Location	Major Rd	Minor Rd	Median Width (ft)	Major Rd Speed Limit (mph)	Number of Trips	Number of Drivers
FL 1	US 41	Flamingo Dr	40	55	96	21
FL 2	US 41	Miller Mac Rd	40	55	161	6
FL 3	US 41	Leisey Rd	40	55	29	10
FL 4	FL 583	Gibson Ave	40	45	41	10
FL 5	FL 583	E 127th Ave	40	45	44	6
FL 6	E Fowler Ave	Williams Rd	44	45	66	20

(a) FL1: US Hwy 41 and Flamingo Dr. (b) FL2: US Hwy 41 and Miller Mac Rd





(c) FL3 US Hwy 41/Leisey Rd (d) FL4 N 56th St/Gibson Ave



(e) FL5 N 56th St and E 27th Ave (f) FL6 E Fowler Ave and Williams Rd

Figure 14 Selected conventional intersections

Table 7 Information on the selected RCUT intersections in Florida and North Carolina

Location	Major Rd	Minor Rd	Median Width (ft)	Major Rd Speed Limit (mph)	Number of Trip	Number of Drivers
FL RCUT1	E Fowler Ave	N 46th St	30	50	5	5
FL RCUT2	E Fowler Ave	N 52nd St	25	50	4	4
NC RCUT1	New Bern Ave	Lord Ashley Rd	30	45	1	1
NC RCUT2	US 70	Cannon Blvd	35	55	7	5
NC RCUT3	NW Maynard Rd	Mall Access	25	45	13	3
NC RCUT4	US 441S	Webster Rd	30	40	6	5
NC RCUT5	Knightdale Blvd	Marks Creek Rd	35	45	1	1
FL RCUT6	Andrew Jackson Hwy	Walker Rd	20	55	3	3

4.1.2 Freeway Diverge Area Study Sites

Table 8 lists ten study sites information, including the type of interchange design, the type of deceleration lane design, the divergence angle, the length of every section (taper, deceleration lane, and off-ramp) in the diverge area, the minimum length determined by the method in the *Green Book*, the number of trips, and the number of unique drivers. The minimum deceleration lane lengths determined by 2018 *Green Book* design criteria were compared with the actual lengths at the study locations to determine if they met the minimum requirements. Five of the ten locations had actual deceleration lane lengths less than the minimum length.

Figure 15 illustrates aerial photos of ten study locations located on I-75 in Florida, including five one-lane exit with parallel-design deceleration lane locations (Locations 1P through 5P), and five one-lane exit with tapered-design deceleration lane locations (Locations 1T through 5T). Eight of 10 locations are diamond interchanges with relatively straight off-ramps. Two others are partial cloverleaf interchanges (Locations 3P and 5P) where the straight off-ramps were selected for reducing the impact on the speed by horizontal curvature (as presented in **Figure 15*e*** and **Figure 15*i***). The divergence angle ranges from 2 degrees to 7 degrees for all locations. For parallel-design locations, taper lengths are from 165 to 270 ft. Taper lengths of tapered-design locations were found to be shorter (130 to 205 ft). Deceleration lane lengths are in the range of 645 to 990 ft for parallel-design locations, which are longer than lengths in tapered-design locations (320 to 445 ft). For both types, off-ramp lengths vary from 940 to 1,725 ft. Most of the locations' off-ramp terminals are signalized intersections while three of them are under yield control (Locations 1T, 3P, and 5T). The speed limit on the freeway mainline is 70 mph for all locations. Off-ramp advisory speeds of 35 mph were posted at four locations (Locations 1P, 2P, 3P, and 4T). It should be noted that limited information is available on establishing advisory speeds for off-ramps that do not have horizontal curvatures (Venglar, et al. 2008). After comparing the actual deceleration lane length of each location with *Green Book* requirements, lengths of deceleration lane from parallel-design locations are longer than the minimum length, while tapered-design locations are shorter.

Table 8 Site description, minimum deceleration lane length, and number of trips and drivers

Site Locations	**Interchange Design**	**Divergence Angle**	**Taper Length (ft)**	**Deceleration Lane Length (ft)**	**Off-Ramp Length (ft)**	***Green Book* Minimum Deceleration Length (ft)**	**Design Status Compared to *Green Book***	**# of Trips**	**# of Drivers**
Location 1P: I-75/SW Archer Rd	Diamond	4°	190	645	1475	490	**GREATER**	92	45
Location 1T: I-75/Clark Rd	Diamond	4°	200	425	1595	615	**LESS**	102	30
Location 2P: I-75/SW County Highway 484	Diamond	5°	195	735	990	490	**GREATER**	23	23
Location 2T: I-75/US 98	Diamond	7°	150	320	940	615	**LESS**	59	48
Location 3P: I-75/FL 326	Parclo	5°	165	775	1030	490	**GREATER**	46	32
Location 3T: I-75/US 98	Diamond	4°	205	420	1170	615	**LESS**	202	56
Location 4P: I-75/CR 768	Diamond	3°	200	700	1180	615	**GREATER**	28	6
Location 4T: I-75/SW College Rd	Diamond	4°	150	445	1340	490	**LESS**	16	13
Location 5P: I-75/CR 765	Parclo	2°	270	990	1690	615	**GREATER**	120	9
Location 5T: I-75/CR 769	Diamond	4°	130	365	1725	615	**LESS**	21	10

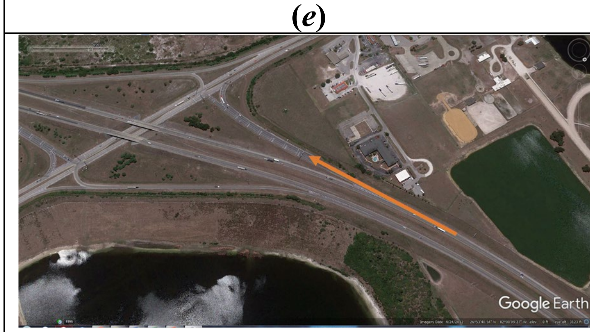
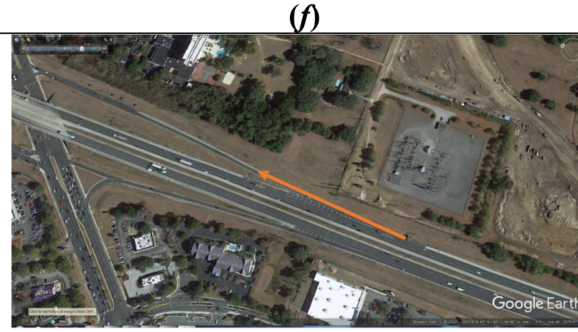
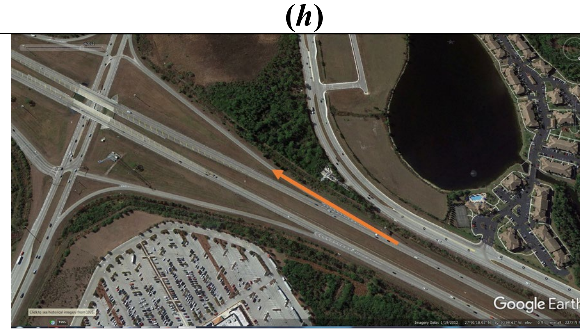
	
(a)	***(b)***
	
(c)	***(d)***
(e)	***(f)***
	
(g)	***(h)***
	
(i)	***(j)***

Figure 15 Aerial photos of study locations: (*a*) Location 1P; (*b*) Location 1T; (*c*) Location 2P; (*d*) Location 2T; (*e*) Location 3P; (*f*) Location 3T; (*g*) Location 4P; (*h*) Location 4T; (*i*) Location 5P; and (*j*) Location 5T (Imagery © 2020 Google, Map data © 2020 Google)

4.2 Data Analysis Method

In this study, the same method was used to analyze the speed data for trips at both intersection and deceleration lane. A polynomial regression method was used to model the speed distribution at different phases of two-stage left turns and different segments of freeway diverge area.

4.2.1 Polynomial Regression

The speed distributions were calculated by applying polynomial regression models, which were estimated using the NDS trips and speed data at 0.1-second intervals. The polynomial regression method minimizes the sum-of-squared residuals between measured and simulated quantities. The least squares method is used to estimate unknown parameters (Gill, Murray and Wright 2019):

$$v = \beta_0 + \beta_1 L + \beta_2 L^2 + \beta_3 L^3 + \cdots + \beta_n L^n + \varepsilon \tag{7}$$

Where, L = The distance from the starting point of the taper along the deceleration lane and off-ramp (ft)
v = Vehicle speed (mph)
β_n = Estimated parameters
ε = The error of the specification

Four best fitted models using NDS speed data, maximum speed, 85$^{\text{th}}$ percentile speed, mean speed, and minimum speed distributions, were developed for each study location by using the statistical computing software R. R software provides a variety of statistical (linear and nonlinear modeling, classical statistical tests, time-series analysis, classification, clustering, etc.) and graphical techniques (R Core Team 1993). The residual standard error was used as a measure of goodness-of-fit to evaluate and determine the quality of the fitted model.

4.2.2 Critical Speed Changepoint Detection

The changepoint detection estimates the point at which the statistical properties of a sequence of observations change (Killick and Eckley 2014). It has been widely used in various application areas, including climatology, bioinformatic applications, finance, oceanography, and medical imaging (Reeves, et al. 2007, Erdman and Emerson 2008, Zeileis, Shah and Patnaik 2010, R. Killick, et al. 2010, Nam, Aston and Johansen 2012). By applying this method, speed time series data is defined as: $V_{1:n} = (V_1, V_2, \dots, V_n)$. A changepoint may occur within this set when there exists a time, $\tau \in \{1, \dots, n-1\}$, where the statistical properties of $\{V_1, \dots, V_\tau\}$ and $\{V_{\tau+1}, \dots, V_n\}$ are different in some ways (R Core Team 1993). The aim of the analysis is to estimate the location of the changepoint efficiently and accurately by minimizing the following equation:

$$\sum_{i=1}^{m+1}\left[C\left(V_{(\tau_{i_1}+1):\tau_i}\right)\right]+\beta f(m) \tag{8}$$

Where, C = A cost function for a segment (e.g., negative log-likelihood)
m = The number of changepoints
$\beta f(m)$ = A penalty to guard against over fitting

This method is used to identify the driver critical speed change position on minor road, the deceleration lane, and off-ramp.

Different methods were used to study driver behaviors at unsignalized intersections and freeway diverge areas. For unsignalized intersections, driver behavior was quantified by visual workload and stopping behaviors. Visual workload was measured using eye-glance data and stopping behavior was quantified by the speed data. For freeway diverge areas, driver behavior was measured by the brake pedal usage and deceleration rates at taper, deceleration lane and off-ramp areas.

4.2.3 Driver Behavior Analysis at Unsignalized Intersection

The analysis of driver behavior was conducted for three different phases for direct left turns: 1 - Deceleration; 2 - Intersection Entry; and 3 - Execute Turn. **Table 9** shows the driving tasks and speed characteristics corresponding to each of the three phases. **Figure 16** shows the details of the study scenario for a typical left turn movement from a minor-road stop-controlled intersection. During the deceleration phase, the main driving task is to stop/ rolling stopped at the intersection and wait for a safe gap to enter the intersection. The second analysis phase is defined as intersection entry. During this phase, drivers get into position to turn and wait for a safe gap of major road oncoming traffic. The last phase is the executed turn, where drivers will find a safety gap to make the left turn and merge into the major road traffic.

Table 9 Driving tasks and speed characteristics of the three phases

Phase	Driving Tasks	Speed Characteristics
1. Deceleration	Stop/ rolling stopped at the intersection, wait for a safe gap to enter the intersection	Start controlled decelerating until stopped/ rolling stopped
2. Intersection Entry	Get into position to turn, wait for a safe gap of major road oncoming traffic	Slowly advance into the median opening and stopped/ rolling stopped at the median opening
3. Executing Turn	Make the turn, merge into the major road traffic	Slowly move to the road edge, start to turn and accelerate up to the speed limit

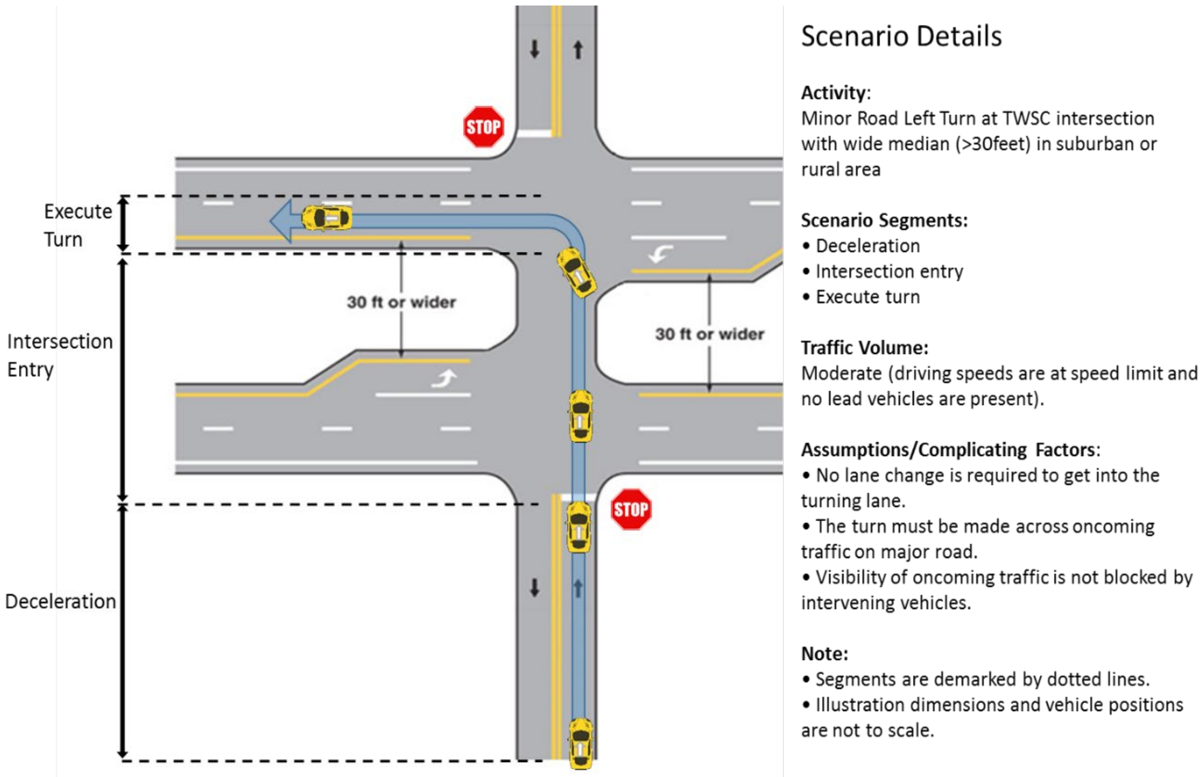


Figure 16 Scenario of left turn movement diagram at TWSC Intersections

4.2.4 Driver Visual Workload Analysis

Descriptive data analysis and entropy rate methods were used to quantify driver visual workload for both direct left turns at conventional intersections and right-turn followed by U-turn movements at RCUT intersections.

Descriptive Data Analysis

The percentage of time spent on each eye glance location was calculated for each trip. Average percentage of time spent on eye-glance locations of all the trips were calculated as follows:

$$\text{Percentage of time spent on each eye glance location} = \frac{\textit{Time spent on glancing at location X}}{\textit{Total Trip Time}} \quad (3)$$

Figure 17 illustrates the eight eye-glancing locations defined for this study, including forward, left windshield, right windshield, rearview mirror, left window/mirror, right window/mirror, over the right shoulder and others (eyes closed, no eyes visible, etc.).

Entropy Rate

The entropy rate of each trip for the study was calculated using **Equation (4)**.

$$\text{Entropy Rate} = \sum_{i=1}^{D} \frac{\left(E/E_{max}\right)}{D \cdot T_{xi}} \quad (4)$$

D, number of variables in the visual scanning sequence

$$D=M \times (M-1)^{N-1} \qquad (5)$$

M, the defined visual scanning area of interests
N, the sequence length of interest

E, Shannon Entropy (Shannon, 1948)

$$E = \sum_{i=1}^{D} Px_i \log_2 \frac{I}{Px_i} \qquad (6)$$

Px_i, probability of occurrence of x_i

$E_{max} = \log_2 D$

Px_i, average fixation duration in the visual scanning sequence (per second)

In this study, each frame individual scan is of interest, so N=1, and the number of variables in the visual scanning sequence (D) is equal to the number of defined visual scanning area of interests (M). There are eight defined visual scanning area of interests for the study, as shown in **Figure 17**. When each eye-glancing location has a same probability of scanning, the E, Shannon Entropy, is the maximum value. So, here $E_{max} = \log_2 8$. Video was recorded at 14 frames per second, therefore, the duration for each frame is 1/14 second. The eye-glancing information was coded for each frame.

Figure 17 Eight eye-glancing locations

4.2.5 Stopping Behavior Analysis

For stopping behavior analysis, the average value for each participant was taken into consideration since the number of trips taken varies by participant. For example, if a participant has taken two trips and stopped only once at the stop sign, then it was considered that the participant has stopped on 50% of the trips. If the same participant has taken trips for more than one location, then the trips were considered by different participants considering the impact on the behavior by the location. Finally, a total of 428 trips by 65 participants were evaluated for stopping behavior analysis. **Table 10** shows the details of participants and trips considered for this analysis.

Table 10 Description of trips and participants for stopping behavior analysis

Location	No. of trips	% of trips	No. of participants
FL1	94	22%	17
FL2	161	38%	6
FL3	29	7%	8
FL4	40	9%	8
FL5	40	9%	5
FL6	64	15%	21
Total	428	100%	65

For the analysis purpose, recorded stopping behavior was divided into four different categories for a single two-stage left-turn maneuver, including (1) drivers stopped at the minor road stop sign or not; (2) drivers stopped at both minor roads and the median; (3) drivers stopped at median only; and (4) drivers did not stop at all. Stopping behavior analysis was conducted at three levels: individual intersection, a subgroup of similar intersections, and all intersections. Also, the impact of demographic and driver-related factors on driving behavior was examined. Gender, age, and education variables were taken as demographic factors while driving experience, training, number of previous crashes, and violation charges were used as driver-related factors.

4.2.6 Method to Determine the Length of Freeway Deceleration Lane

Freeway deceleration data analysis was performed from three aspects: (1) speed distributions on deceleration lanes and off-ramps; (2) driver behaviors in terms of the brake pedal usage and the deceleration rates, and (3) methods on estimating minimum deceleration lane length for naturalistic driving speeds and deceleration rates.

Reviewing videos was the first step in the data analysis necessitated. Observers recorded the video frame number (the time stamp) at critical points in the video. Taper start point, deceleration lane start point, deceleration lane endpoint (physical gore), and off-ramp endpoint (stop bar at the terminal) on each location were considered critical points for this analysis. The frame number allowed for correlation to the data in the time-series report (speed, acceleration/deceleration rate, brake pedal status, etc.). Thus, the time stamp of each critical point in the time-series table was tagged to help determine the speed distribution (i.e., maximum, 85th percentile, mean, and minimum speed; and their standard deviations) of every section on the deceleration lane and off-ramp. Driver behavior was identified by brake pedal usage and deceleration rate. Brake pedal status was coded as 0 or 1 in the time-series reports. The value of 0 indicates that the driver did not apply the brake at the certain 0.1 seconds, while 1 means he or she did. To find where drivers applied brakes most often, brake pedal usage was evaluated by the percentage of the drivers' applying brakes in certain sections.

The time-series reports provided deceleration rates which can be used to calculate the mean and 85th percentile deceleration rates on the taper, deceleration lane, and off-ramp sections. The rates can also be determined by converting the distance-based speed model to the time-based one. The deceleration rate distribution was executed to find out the section where drivers mostly reduce their speeds so that the effective decelerating section could be found. When calculating deceleration rates, the *Green Book* recommended two methods (AASHTO 2018): one is based on a two-step process of deceleration, coasting (assumed 3 seconds) and braking; the other is based on a constant decelerating behavior on the deceleration lane which was validated by El-Basha et al. (El-Basha, Hassan and Sayed 2007). In this study, the deceleration rate was compared with the *Green Book* rates based on a constant decelerating behavior over the entire deceleration process.

$$D = \frac{v_i^2 - v_f^2}{2d} \tag{9}$$

Where, D = Deceleration distance (ft)
v_i = Initial speed (ft/s)
v_f = Final speed (ft/s)
d = Deceleration rate (ft/s^2)

After determining the deceleration rates, the minimum deceleration lane length can then be estimated based on the deceleration rate from NDS data and polynomial regression models by using Equation 5.

5 ANALYSIS RESULTS

This section contains the NDS data analysis results at both the unsignalized intersections and the freeway diverge areas. Analysis results at the unsignalized intersection include the driver visual workload study, driver speed change, brake behavior, and stop behavior study. For the driver visual workload study part, descriptive data analysis results, visual workload comparison between two types of intersections, and the visual workload analysis results at conventional were provided. Driver speed change and brake behavior study results were separated into three different phases. Analysis at the freeway diverge areas covers the speed distribution, driver brake pedal usage, deceleration rate distribution, and the determination of the minimum length of deceleration lane.

5.1 Analysis Results on NDS Data at Unsignalized Intersection

5.1.1 Driver Visual Workload

Average Percentage of Time Spent on Eye-glance Locations

Figure 18 showed the average percentage of time that direct or indirect left turn drivers spent on glancing at 430 conventional intersection trips and 40 RCUT intersection trips. For RCUT intersections, drivers spent the most proportion of time glancing forward (72%), followed by glancing the left window/ mirror (14%). For conventional intersections, beside looking forward (56%) and left window/mirror (17%), drivers also spent a large portion of time glancing the right window/ mirror (11%), and right windshield (5%). The results suggest that left turn drivers spent more time on looking left and right mirrors and less time looking forward. Additionally, the drivers at conventional intersections also spend more than 7% of time moving their body to look at the rearview mirror, and look backwards over the shoulder, which can bring more workload to the drivers while making the turning movements.

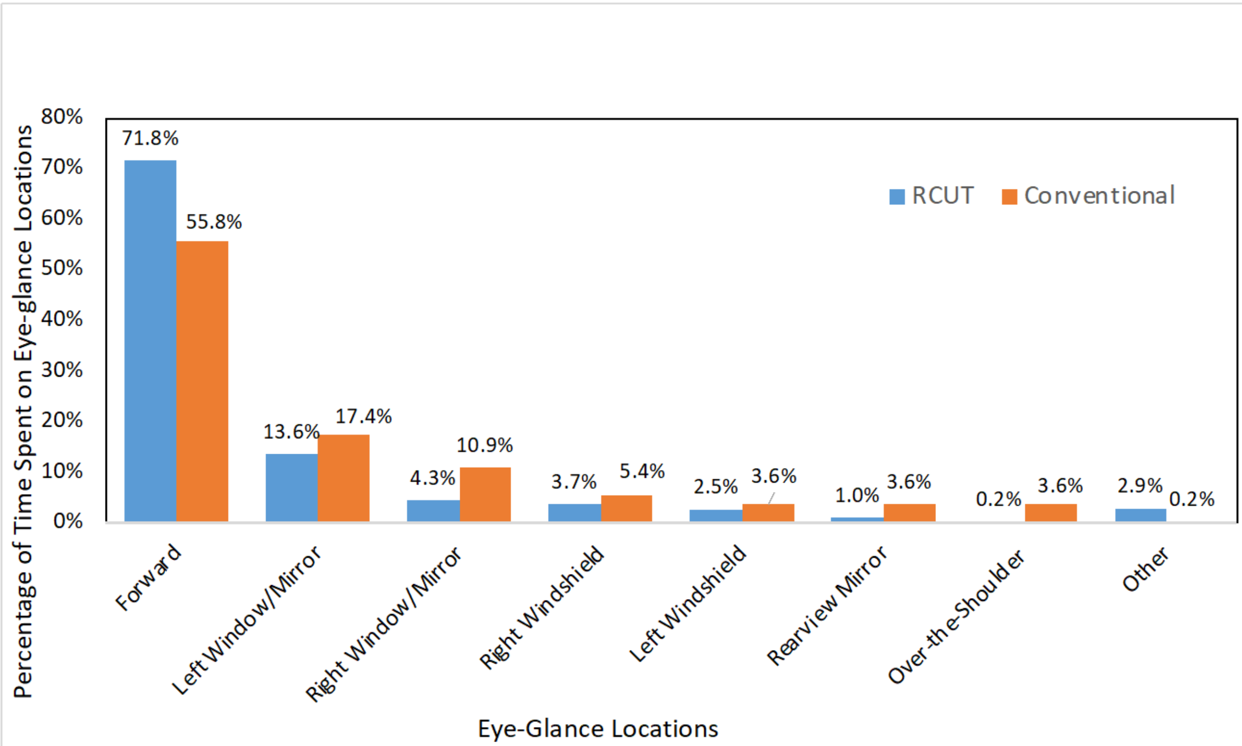


Figure 18 Average percentage of time spent on eye-glance locations

<u>Visual Workload Comparison</u>

Figure 19 showed the boxplots of entropy rates between conventional and RCUT intersections with Welch T-test results**.** The average entropy rate of drivers making left turns at conventional intersections (0.24) is more than twice higher than drivers making indirect left turns at RCUT intersection (0.09). Similarly, the median value of entropy rate of drivers making direct left turns at conventional intersections (0.22) is about four times higher than the drivers making indirect left turns at RCUT intersections (0.06).

Welch t-test was used to test the null hypothesis that the mean entropy rate of direct left turn drivers is equal to the right-turn followed by U-turn drivers. Welch's t-test is a modification of Student's t-test to determine if two sample means are significantly different and can be used for two samples with unequal variances and frequency. Since the sizes of the two samples and their variances are unequal, Welch's t-test was applied in this study. The test results show that the mean entropy rates of the two types of intersections differed significantly according to Welch's t-test, $t = 9.24$, $p < .001$, implying a significant difference between the mean entropy rates of the two types of maneuvers. The results suggest that drivers making direct left turns at conventional intersections have a higher visual workload compared to drivers making diverted left turns at RCUT intersections.

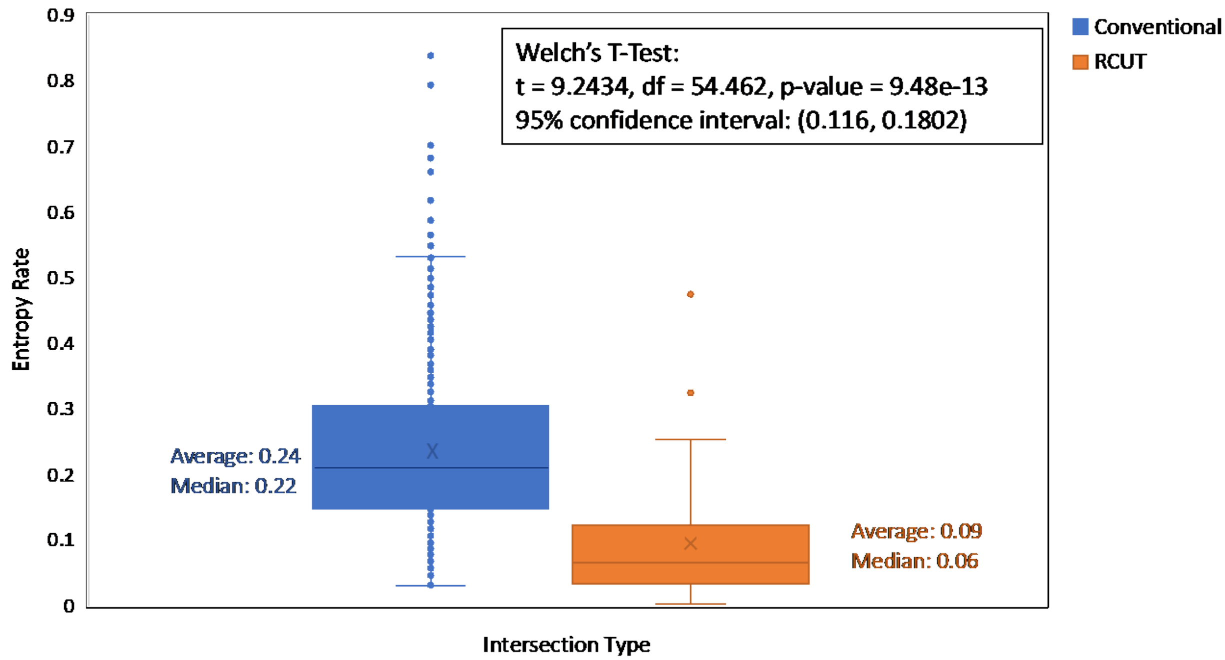


Figure 19 Welch t-test results and boxplots of entropy rates

Figure 20 shows the boxplots of the entropy rates among drivers of different gender and age. Based on the boxplots, the average entropy rate for the male drivers is higher than the female drivers at conventional intersections. For RCUT intersections, the study found there is no significant different between average entropy rates by male and female drivers. Younger drivers have a slightly higher average entropy rate than the middle-aged and older drivers at both intersection types. The results also suggest that male and younger drivers have higher randomness in scanning patterns while crossing the intersections.

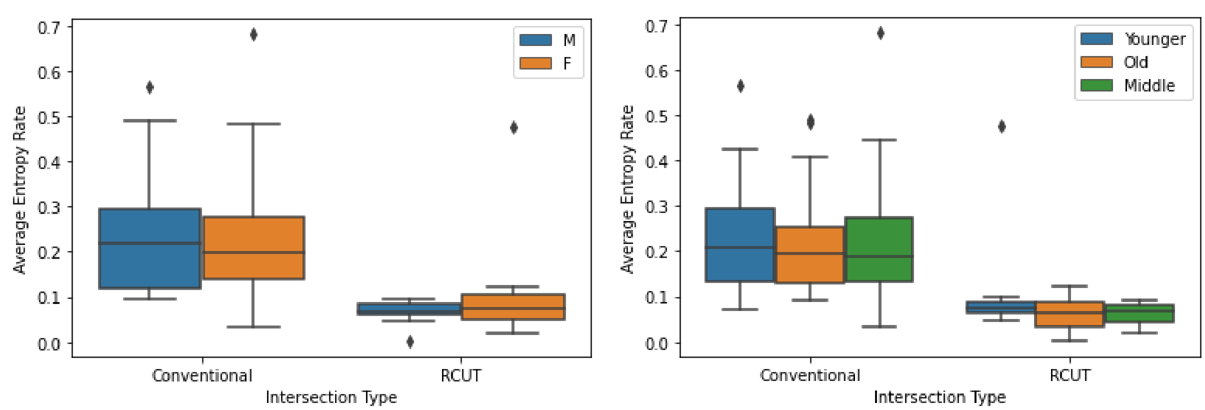


Figure 20 Boxplots of entropy rate among drivers of different gender and age

Visual Workload at Conventional Intersections

Further analysis was conducted to examine the correlation between entropy rates and driver demographic characteristics and roadway design features at conventional intersections. Due to the limited participants and trip numbers, no statistical analysis for visual workload among drivers at the RCUT intersections was conducted in this study.

Driver Demographic Feature Analysis

A two-way ANOVA was used to analyze the effects of gender and age on entropy rates at conventional intersections. **Table 11** shows the two-way ANOVA analysis results. All the p-values are larger than 0.05, indicating no significant differences at the 95% confidence interval in visual workload among drivers of different gender and age at conventional intersections.

Table 11 Two Way ANOVA Analysis of Entropy Rate at Conventional Intersections

	Df	**Sum Sq**	**Mean Sq**	**F-value**	**P-value(>F)**
Gender	1	0.0002	0.000175	0.017	0.898
Age	2	0.0077	0.00385	0.366	0.696
Gender: Age	2	0.046	0.022982	2.185	0.126
Residuals	40	0.4207	0.010517		

A Pearson correlation analysis was conducted to see if the social economic and demographic features (income level, education level, driving years, crash numbers in the past 3 years) and the driver risk perception scores were correlated with entropy rates. **Table 12** lists the attributes of driver's demographic features and risk scores. They were categorized into different levels as continuous numbers for analysis purpose.

Table 12 Driver demographic information and the average risk perception scores

Features	**Categories**	**Levels**
Education	High school or G.E.D. (24%)	1
	Beyond high school or college degree (48%)	2
	Graduate or professional school or advanced degree (28%)	3
Income Level	<50,000 (41%)	1
	50,000 - 70,000 (15%)	2
	70,000 - 100,000 (33%)	3
	>100,000 (11%)	4
Driving Years	0-70	-
Crash Records in the past 3 years	0 crash (44%)	1
	1 crash (42%)	2
	≥ 2 crashes (14%)	3
Average Risk Perception Scores	0-7	-

Figure 21 showed the Pearson correlation analysis results. The darker color indicates higher correlation. With the exception for income levels, increases in education level, years of driving,

crash numbers, and drivers risk perception scores resulted in the decreased average entropy rates. The correlation between the entropy and the risk perception score. One of the reasons is caused by the accuracy of the driver survey results. Authors noticed some extreme answers in the NDS risk perception score survey.

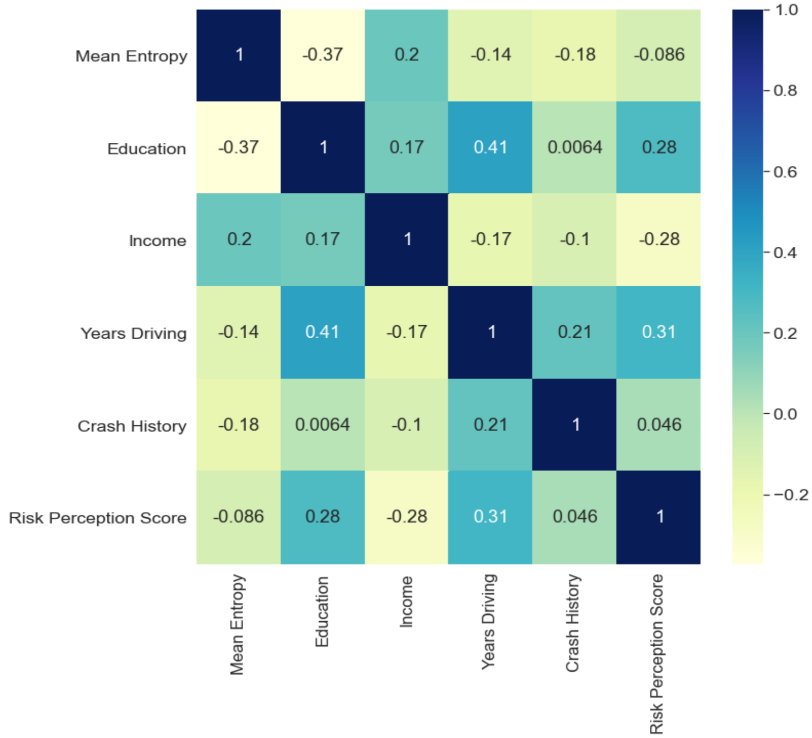


Figure 21 Pearson correlation between driver demographic features

Roadway Feature Analysis

The impact of roadway features on entropy rates were also analyzed. The roadway features include channelization island at minor road; 3-leg or 4-leg intersection; major-road AADT ≤ 20,000 or ≥ 30,000; major-road speed limit 45 mph or 55 mph. Welch t-tests were used to determine whether these roadway features had a significant impact on driver visual workload. **Table 13** showed the Welch t-test results of impacts of different roadway features on entropy rates. Among the four features, only the p-value of AADT is less than 0.05, which implies that AADT has statistically significant impact on the visual workload. The results indicate that driver visual workload at the intersections with major road AADT ≥ 30,000 is much higher than the intersections with AADT ≤ 20,000.

Table 13 Welch t-test results of entropy rate between different roadway features

95% Confidence Interval						
	Mean Difference	**Lower**	**Upper**	**t**	**df**	**P-value**
No Channelization (66.9%) vs. Channelization on Minor Road (33.1%)	-0.006	-0.035	0.023	-0.403	217.640	0.687
3_Intersection (75.8%) vs. 4-leg Intersection (24.2%)	0.015	-0.015	0.045	1.000	152.650	0.319
Major AADT≤20,000 (50.6%) vs. Major AADT≥30,000 (49.4%)	-0.069	0.045	0.092	5.677	407.860	<0.001***
45 mph (66.9%) vs. 55 mph (33.1%)	0.006	-0.023	0.035	0.403	217.640	0.687

5.1.2 Driver Speed Change Behavior Analysis

Driver speed change behavior was analyzed for 430 direct left turn movements in three phases: deceleration, intersection entry, and executing turn.

Phase 1 – Deceleration

Figure 21 shows the vehicle speed trajectory in phase one among the different age group drivers. Polynomial regression models were developed to predict 85th percentile speed for the different types of the drivers. It was found that the 85th percentile speed of younger drivers is much higher than the middle age and order driver.

Figure 22 shows the critical change points of the 85th percentile speed of all trips. The critical change points suggest that most of the drivers tend to decelerate sharply when they are about 115 ft or 50 ft away from the minor-road stop bar (**Figure 23**). **Table 14** shows the Tukey Honestly Significant Difference (HSD) test results on if different age drivers had different average speed at different distances from the stop bar. The results show that the average speed of the younger driver is significant higher (p-value smaller than 0.05) than the older and middle-aged drivers when the distance is smaller than 100 ft.

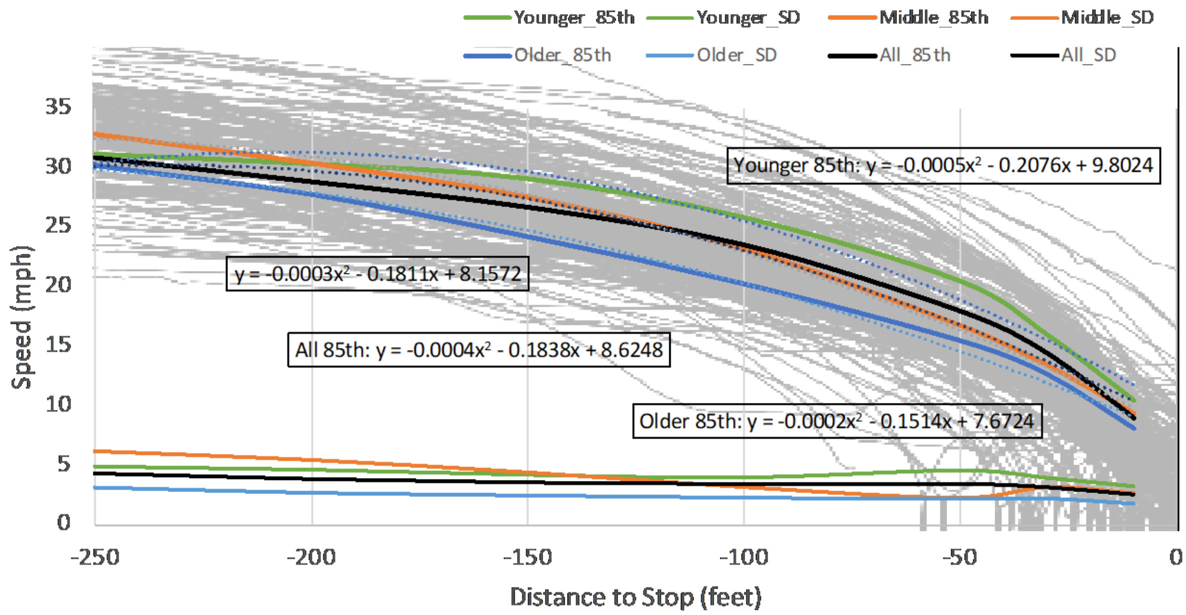


Figure 22 Vehicle speed trajectory on minor road by different age drivers

Table 14 Tukey HSD test results on average speed by different age group drivers

Distance to stop(ft)	Tukey HSD Test Results				
		Diff	lower	upper	p-adj
250	Old-Middle	1.054	-1.239	3.348	0.524
	Young-Middle	-0.291	-2.664	2.082	0.955
	Young-Old	-1.346	-2.838	0.147	0.087
200	Old-Middle	0.103	-1.974	2.180	0.993
	Young-Middle	0.436	-1.713	2.585	0.881
	Young-Old	0.334	-1.018	1.685	0.830
150	Old-Middle	-0.609	-2.462	1.245	0.719
	Young-Middle	1.572	-0.346	3.490	0.132
	Young-Old	2.181	0.974	3.387	<0.001
100	Old-Middle	-1.450	-3.126	0.227	0.105
	Young-Middle	1.836	0.102	3.571	<0.05
	Young-Old	3.286	2.195	4.377	<0.001
50	Old-Middle	-0.755	-2.529	1.019	0.575
	Young-Middle	1.663	-0.173	3.499	<0.05
	Young-Old	2.418	1.264	3.573	<0.001

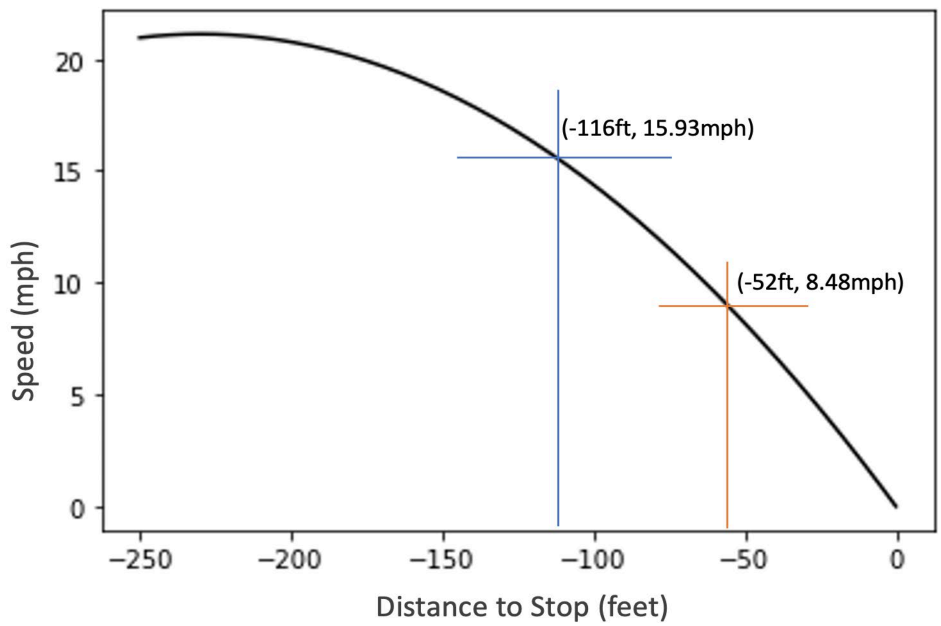


Figure 23 Critical change point of 85th percentile speed of all trips on minor road

Figure 24 showed the brake pedal use condition in phase 1 of left turn movement on the minor road. At the first critical change point, when the vehicles were about 115 ft away from the stop line, about 35% of the vehicles used the brake pedal; and at the second critical change, when the vehicles were about 50 ft away from the stop line, about 50% of the vehicles used the brake pedal. The results are consistent with the speed trajectory analysis.

The NDS data analysis results suggest that advanced intersection warning sign should be located at least 120 ft+2.5 * Speed Limit before the intersection, assuming 2.5 seconds perception and reaction time.

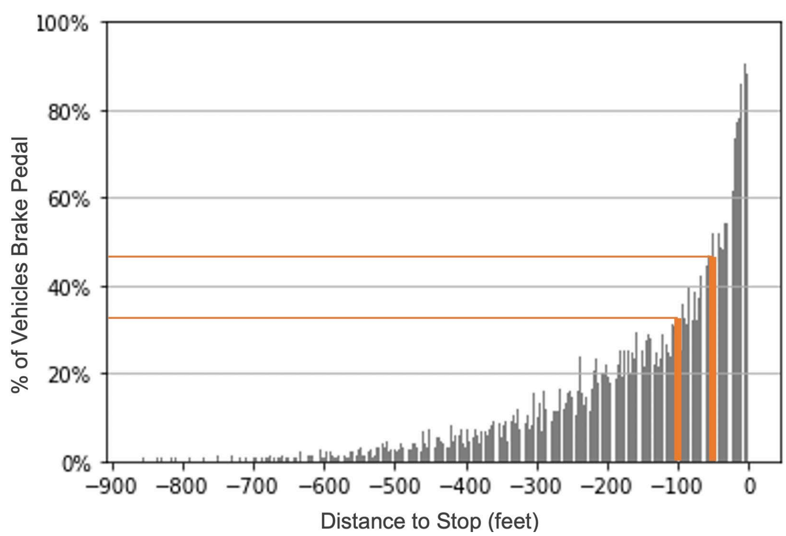


Figure 24 Brake pedal usage on minor roads

Phase 2 Intersection Entry

Figure 25 showed the brake pedal use conditions during intersection entry phase. Approximately 70% of vehicles used brake pedal at the distance of 0 (near the minor road stop sign), while only 20% of the drivers braked at the median openings (distance between 80 to 100 ft). **Figure 26** showed the vehicle median (speed equals to 0 when the distance is around 80 ft), and other parts of the vehicles did not top at the median, and their speed can be 15 to 20 mph while at the median openings.

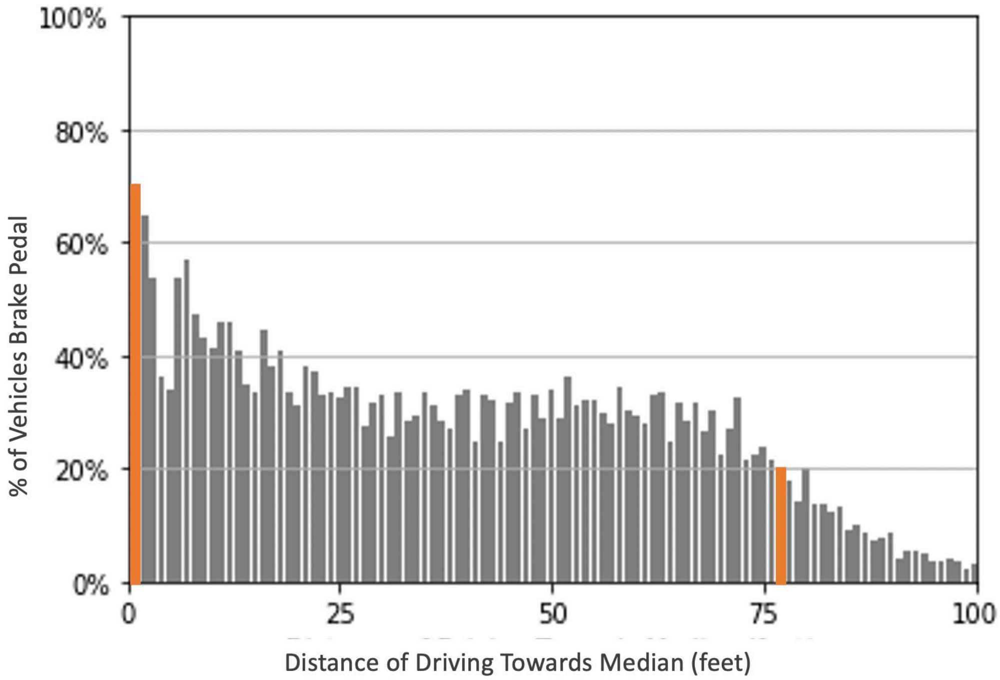


Figure 25 Brake pedal usage of intersection entry

Figure 26 shows the vehicle speed trajectory during the intersection entry. The average distance from the minor-road stop bar to the median opening is approximately 80 ft. The trajectory data showed that a large portion of vehicle speed went through a cycle of from acceleration to 10 mph to deceleration to near zero. Some vehicles did not decelerate to stop at the median, and their speed can be up to 20 mph while passing through the median openings.

Figure 27 shows the vehicle stop conditions at the median openings. The completed stop conditions are defined as the spot speed is less than 3 mph; slow down means the minimum speed is larger than 3 mph but less than 10 mph; none means the vehicle neither stop nor slow down at the median openings. The data shows that almost half of the vehicles did not stop or slow down.

Figure 28 shows the vehicle speed trajectory of intersection entry for those drivers who did not stop or slow down at the median openings. Polynomial regression models were developed to predict 85th percentile speed for the different age groups of the drivers. It was found that the 85th percentile speed for younger drivers is significantly higher than the middle age and order driver.

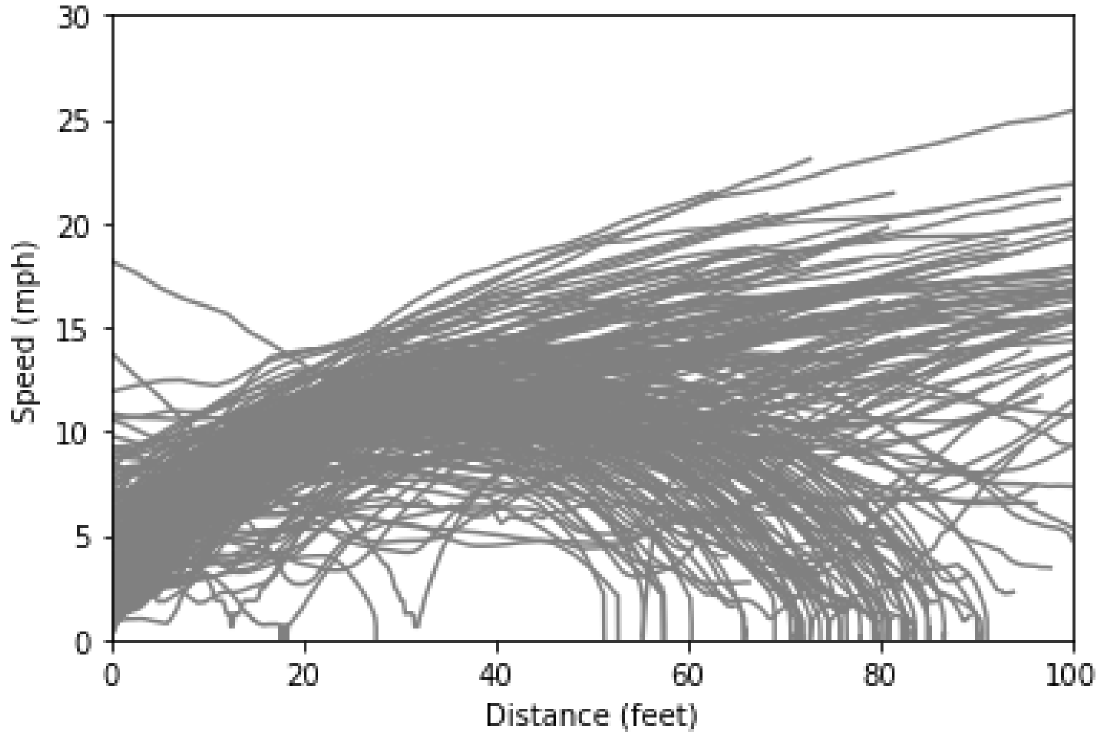


Figure 26 Vehicle speed trajectory during intersection entry

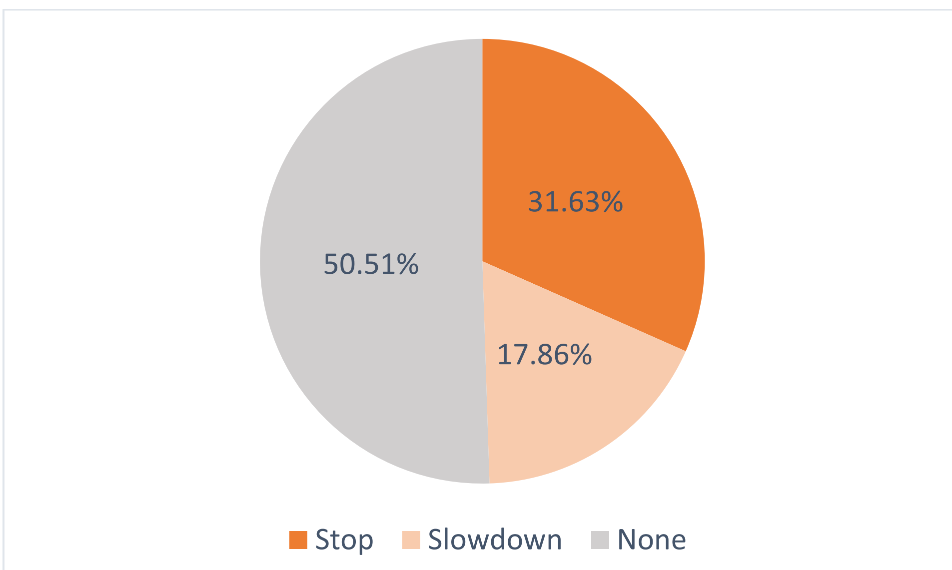


Figure 27 Vehicle stop conditions at the median openings

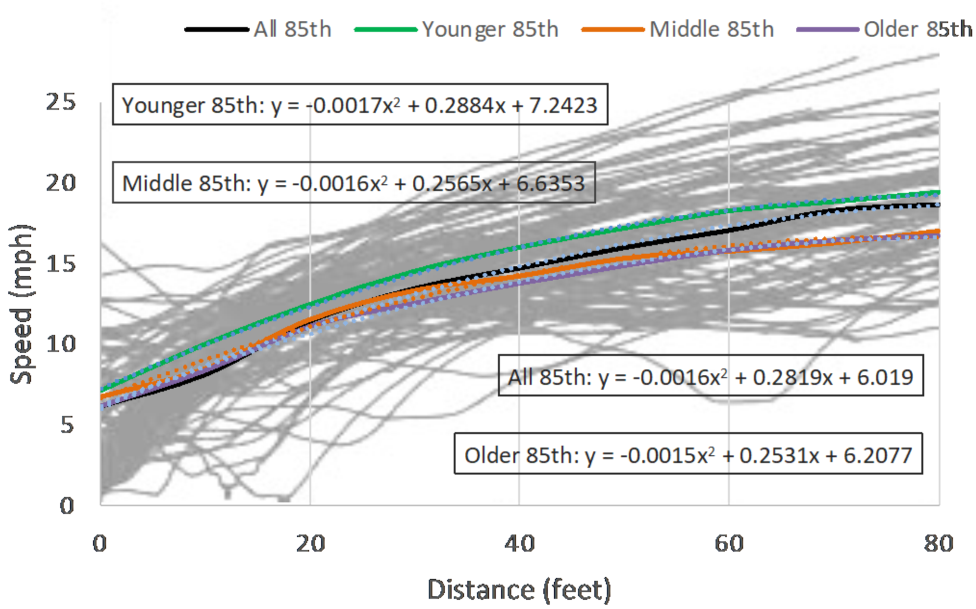


Figure 28 Vehicle speed trajectory of intersection entry for different age drivers who did not stop or slow down at the median openings

Table 15 shows the Tukey HSD test results on average speed by different age group drivers (who did not stop). The results show that the speed of the younger driver is significantly different with the older and middle-aged drivers at the confidence level of 90% at the distances of 20 ft, 40 ft, 60 ft and 80 ft from the stop bar.

Table 15 Tukey HSD test results of different age drivers (who did not stop) at different distance of phase 2

Distance to Stop Bar (ft)	Tukey HSD Test Results				
		Diff	Lower	Upper	p-adj
20	Old-Middle	-0.755	-2.529	1.019	0.575
	Young-Middle	1.663	-0.173	3.499	<0.1
	Young-Old	2.418	1.264	3.573	<0.001
40	Old-Middle	-0.755	-2.529	1.019	0.575
	Young-Middle	1.663	-0.173	3.499	<0.1
	Young-Old	2.418	1.264	3.573	<0.001
60	Old-Middle	-0.755	-2.529	1.019	0.575
	Young-Middle	1.663	-0.173	3.499	<0.1
	Young-Old	2.418	1.264	3.573	<0.001
80	Old-Middle	-0.755	-2.529	1.019	0.575
	Young-Middle	1.663	-0.173	3.499	<0.1
	Young-Old	2.418	1.264	3.573	<0.001

Phase 3 Execute Turn

Figure 29 shows the speed trajectory when drivers accelerating on the major road. To accelerate to the speed of 45 mph, 85% of all the NDS drivers used less than 650 ft. The average acceleration distance of all the trips is 480 ft. Though there are three locations with major road speed limit of 55 mph, less than half of the drivers accelerate to 55 mph based on the NDS speed data. **Figure 30** shows the brake pedal use condition on major road. It suggests that brake pedal was used by average of 16% drivers to merge onto major road in the first 100 ft on major road.

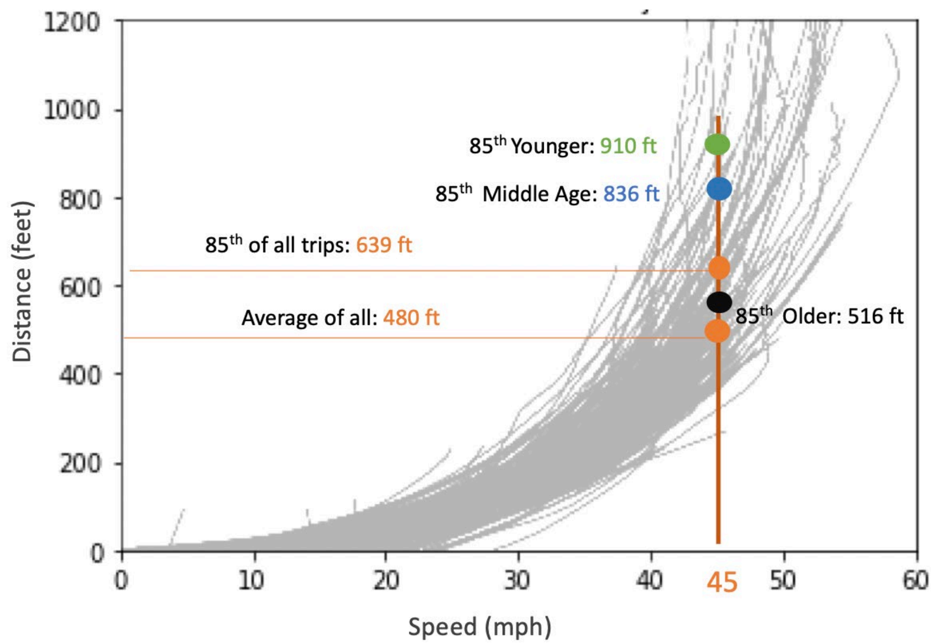


Figure 29 Vehicle accelerating trajectory on major road

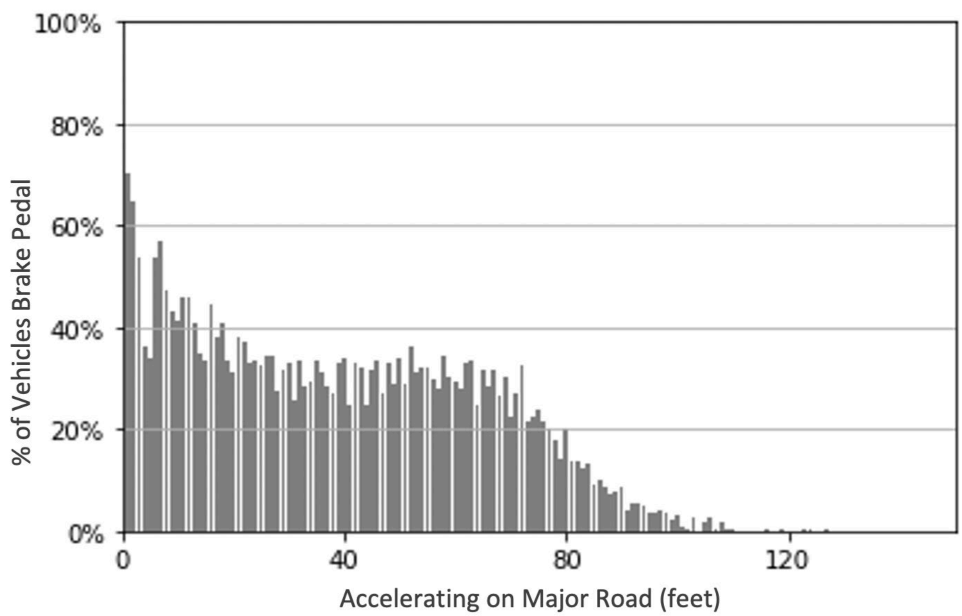


Figure 30 Brake pedal usage on major road

5.1.3 Driver Stopping Behavior Analysis

Driver stopping behavior was analyzed for two-stage left turn movements at the six study locations. The analysis results indicated that intersection geometry and traffic characteristics had an impact on stopping behaviors of the two-stage left-turn movements. To better understand how geometry and traffic characteristics affect stopping behavior, the six study locations were categorized into four types (**Table 16**). Each type has similar speed limit, AADT, intersection type (3-leg or 4-leg). Types 1 and 2 are 3-leg intersections. Type 1 location has higher major and minor-road speed limits. Types 3 and 4 are 4-leg intersections. Type 4 has higher major and minor-road speed limits. Type 1 location includes FL1, FL2 and FL3. All of them are T-intersections with the similar major road AADT (33,000), speed limit (55 mph), and median width (40 ft).

Table 16 Category of locations

Location Type	**Type 1**	**Type 2**	**Type 3**	**Type 4**
Major Rd Speed limit (mph)	55	40	40	55
Minor Rd Speed Limit (mph)	25	20	20	40
Major Rd AADT	33,000	27,000	27,000	26,000
Intersection Type	3-leg	3-leg	4-leg	4-leg
Site ID	FL1, FL2, FL3	FL4	FL5	FL6
# of Participants	15	7	4	16
# of Trips	158	37	37	41

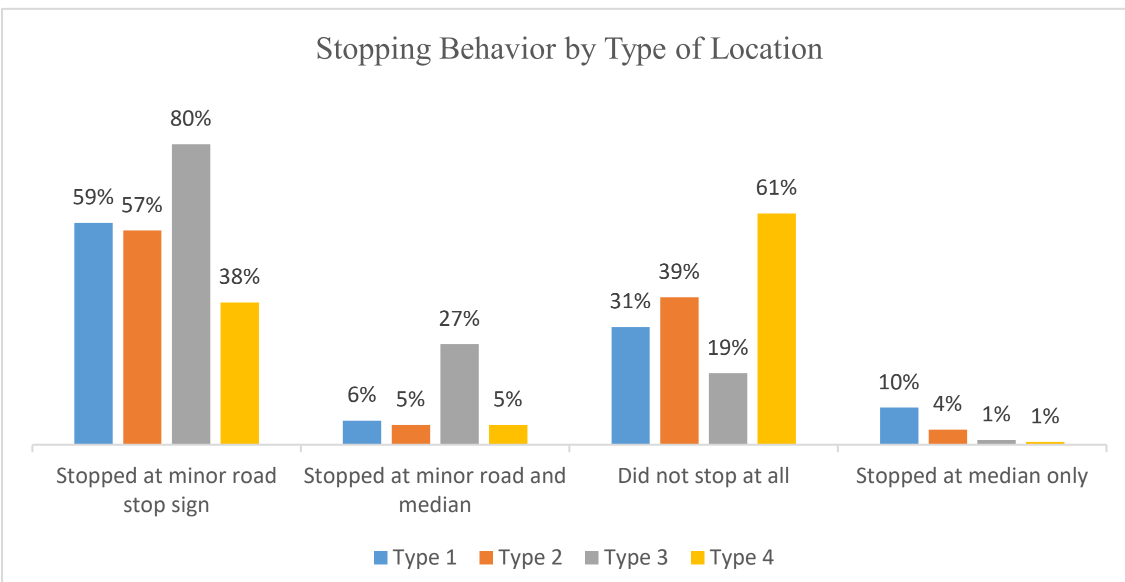


Figure 31 Driver stopping behavior by types of locations

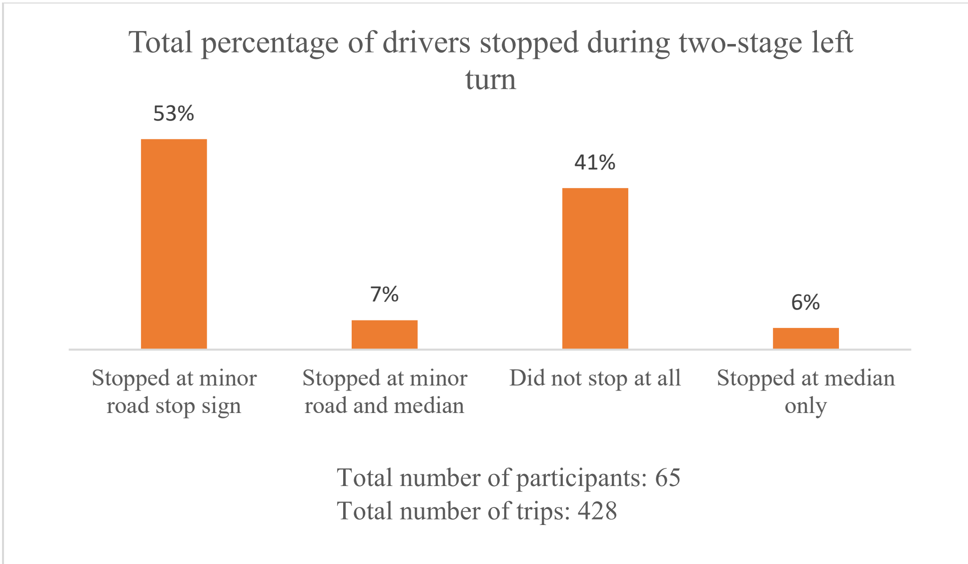
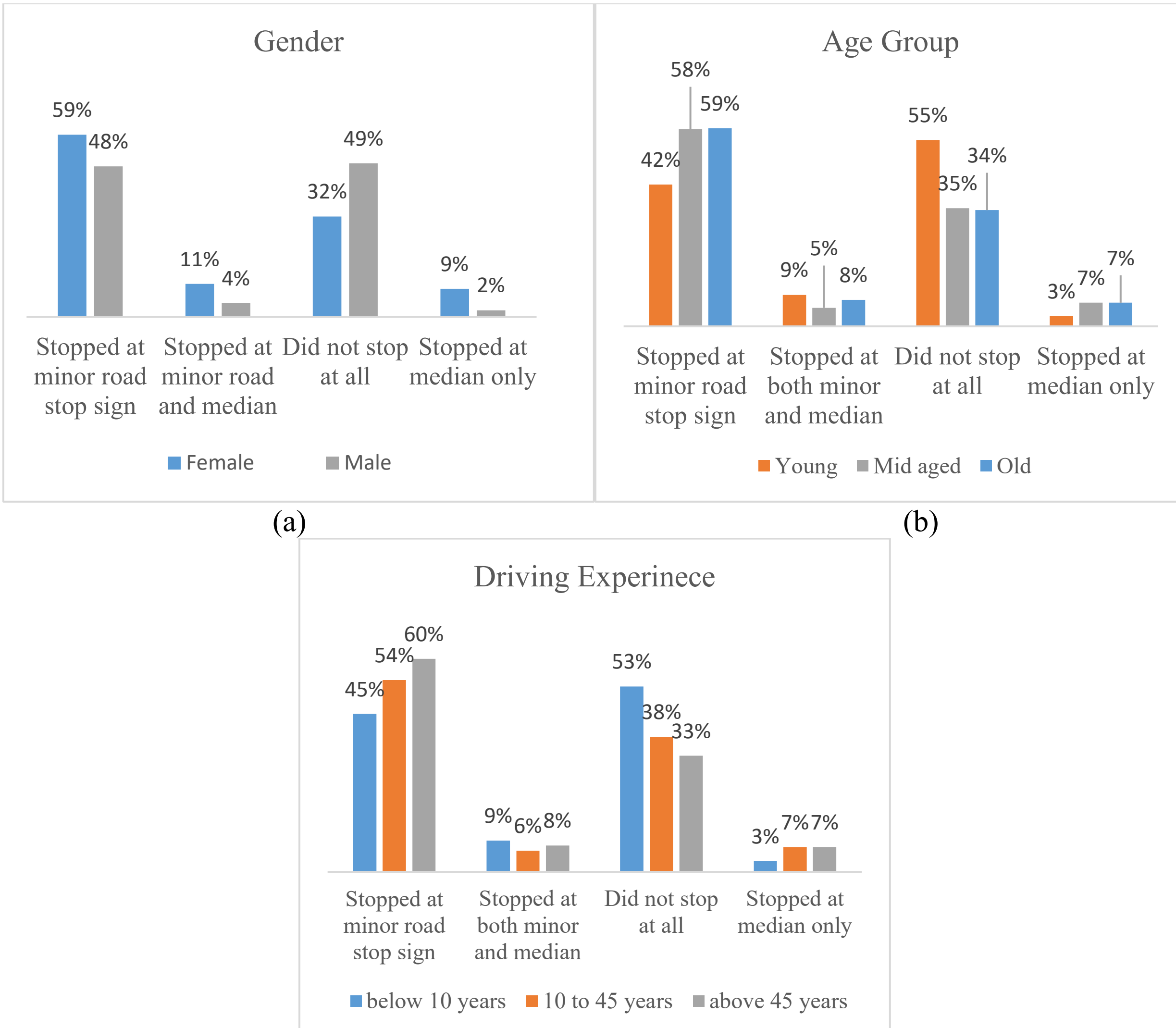


Figure 32 Driver stopping behavior at six study locations

Figure 31 showed the results of stopping behavior with respect to type of location. In general, the type of intersection, speed limit and AADT were found to have an impact on stopping behaviors. A higher percentage of drivers did not stop at type 2 locations (39%) than type 1 locations (31%). This indicates that lower AADT and major road speed at location 2 may contribute to the higher non stopping percentage. The percentage of not stopping was found to be much higher at type 4 location (61%) than the type 3 location (19%). The results suggest that the high minor road speed limit 40 mph had a significant impact on stopping behaviors. **Figure 32** showed the results of the assessment of stopping behavior made by 65 drivers on 428 trips at all the intersections. It indicates that only 7% of drivers stopped at both minor road and median, 41% of drivers did not stop at both locations. The detailed analysis of stopping behavior for each individual location is contained in APPENDIX A.

Impact of Demographic and Driver-Related Factors

Figure 33 showed the results of the impact of two demographic factors (gender, age) and one driver related factor (driving experience) on stopping behavior. It showed that a larger percent of female drivers stopped at minor-road and median opening, a larger percent of young drivers (55%) did not stop at all when making a two-stage left turn movements. It is also evident that drivers with higher percentage of driving experience are more likely to stop (66%). The detailed results on the impact of various driver-related factors on stopping behavior at each location can be found in APPENDIX A.



(a) (b)

(c)

Figure 33 Driving behavior analysis: (a) Gender; (b) Age; (c) Driving Experience

5.2 Analysis Results on NDS Data at Freeway Diverge Areas

The analysis results on freeway diverge areas are categorized into three parts: (1) polynomial regression of speed distribution on the deceleration lanes and off-ramps; (2) driver behavior in terms of brake pedal usage, deceleration rates, and a comparison with the *Green Book* assumptions; and (3) minimum lengths of deceleration lanes based on naturalistic driving speed and deceleration rates.

5.2.1 Speed Distribution

Four fitted speed distribution profiles by polynomial regression are presented in **Figure 33**, which shows speed distribution on the deceleration lane and the off-ramp in Locations 1P and 1T. The x-axis is the length (ft) and the y-axis is the speed (mph). The light blue lines are the speed data from

NDS time-series reports, one trace coming from one traversal. The other four lines in the figure are fitted polynomial regression models, including the maximum speed distribution (Maroon), the 85th percentile speed distribution (Red), the mean speed distribution (Orange), and the minimum speed distribution (Pink). The critical points are also marked with estimated speeds.

For example, the 85th percentile speed distribution in Location 1P (**Figure 34*a***), the speed at the beginning of the taper was 74.02 mph. It was reduced to 72.67 mph when the vehicle entered the deceleration lane. The speed was further reduced to 63.39 mph after driving through the 645 ft deceleration lane, resulting in a 9.28 mph speed reduction on the deceleration lane. However, it was found that a great speed reduction occurred on the off-ramp, especially close to the off-ramp terminal where a signalized intersection exists. Finally, the 85th percentile speed was reduced to 23.88 mph. As for Location 1T as shown in **Figure 34*b***, the speed distribution was slightly different from Location 1P. Before the taper in Location 1T, an extra 210-ft segment before the taper section was counted to make the length equal to the total length of taper and deceleration lane in Location 1P. It was found that, in Location 1T, drivers decelerated on the mainline before entering the taper section. The 85th percentile speed at the taper start point was 69.64 mph, which is nearly 5 mph lower than that in Location 1P. When entering the deceleration lane, the speed was 68 mph. The 425-ft deceleration lane only helps reduce 3 mph considering the speed at the off-ramp start point being 64.49 mph. Similar to Location 1P, a significant speed reduction of 33.58 mph was observed on the off-ramp. The speed distribution of other study locations can be found in Appendix B.

Polynomial regression models of 85th percentile speed and mean speed distributions for Locations 1P and 1T are summarized as follows:

For Location 1P:

$$v_{1P-85^{th}} = -9.767 \times 10^{-12} {L_{1P}}^4 + 3.38 \times 10^{-8} {L_{1P}}^3 - 3.462 \times 10^{-5} {L_{1P}}^2 - 1.703 \times 10^{-3} L_{1P} + 74.02 \tag{6}$$

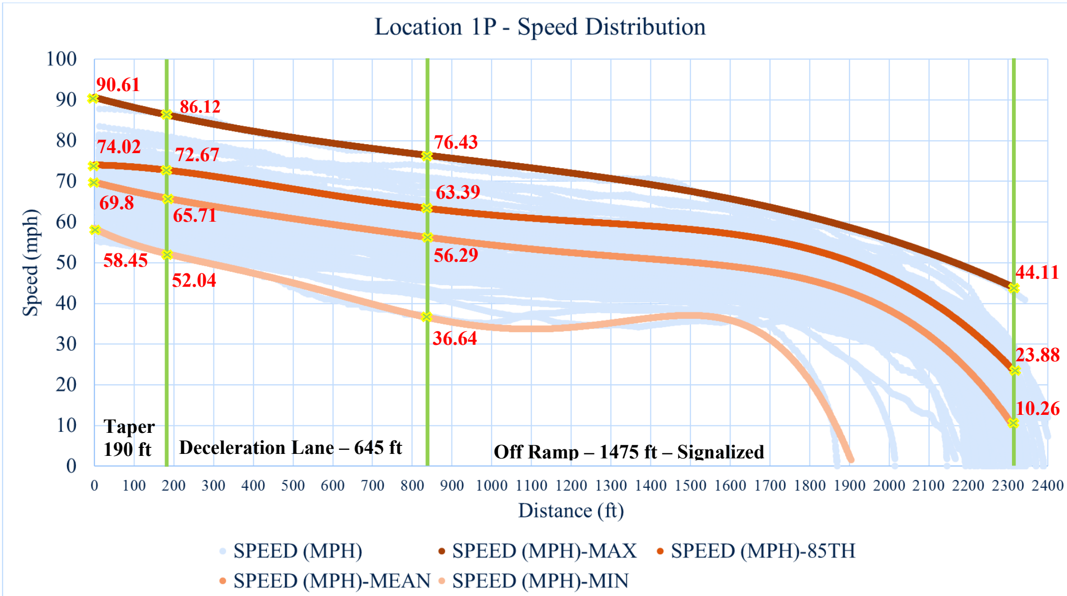
$$v_{1P-Mean} = -6.646 \times 10^{-15} {L_{1P}}^5 + 2.697 \times 10^{-11} {L_{1P}}^4 - 3.978 \times 10^{-8} {L_{1P}}^3 + 2.997 \times 10^{-5} {L_{1P}}^2 - 2.594 \times 10^{-2} L_{1P} + 69.80 \tag{7}$$

For Location 1T:

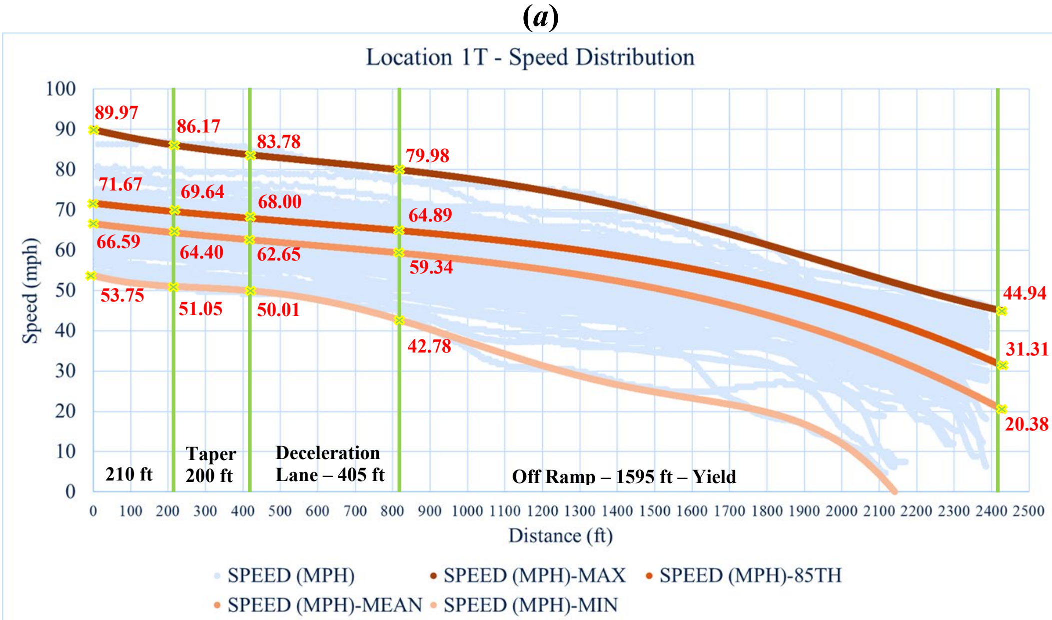
$$v_{1T-85^{th}} = -3.32 \times 10^{-9} {L_{1T}}^3 + 5.64 \times 10^{-6} {L_{1T}}^2 - 1.071 \times 10^{-2} L_{1T} + 71.67 \tag{8}$$

$$v_{1T-Mean} = -3.968 \times 10^{-9} {L_{1T}}^3 + 6.61 \times 10^{-6} {L_{1T}}^2 - 1.165 \times 10^{-2} L_{1T} + 66.59 \tag{9}$$

All study locations performed four regressions. The equations can be found in Appendix C. It should be noted that all estimated parameters are statistically significant at the 99% confidence level. L is defined as the distance from the starting point of the taper to any points on the taper, deceleration lane or off-ramp. v is the speed downstream from the taper start point.



(*a*)



(*b*)

Figure 34 Speed distributions: (*a*) Location 1P; and (*b*) Location 1T

From the models developed, only 85th percentile speeds and mean speeds at the taper start point, deceleration lane start points, deceleration lane endpoints, and off-ramp endpoint were summarized in **Table 17**. The speeds at parallel-design locations were 1-2 mph higher than that at tapered-design locations in taper and deceleration lane sections. However, the speeds upon vehicles entering the off-ramp for Locations 1T to 5T were typically 3 mph higher than parallel-design locations. When an advisory speed was posted on the off-ramp, the operating speeds were not significantly affected by the advisory speed which is 35 mph for Locations 1P, 2P, 3P, and 4T. The mean speed for a 35-mph advisory speed location was approximately 55 mph, and the approximate speed was 58 mph without the advisory speed sign.

Table 17 A comparison of speed distribution and speed reduction percentage on the deceleration lane and off-ramp: (a) parallel-design locations; and (b) tapered-design locations

(a)

Site		Speed (mph)				Speed Reduction Percentage*		
		Taper Start	Deceleration Lane Start	Deceleration Lane End	Off-Ramp End	Taper	Deceleration Lane	Off-Ramp
Location 1P 645 ft	85th	74	73	63	24	2.7%	18.5%	78.8%
	Mean	70	66	56	10	6.9%	15.8%	77.3%
Location 2P 735 ft	85th	73	70	60	32	5.8%	24.6%	69.6%
	Mean	65	66	53	18	-2.4%	26.8%	75.6%
Location 3P 775 ft	85th	68	65	56	19	6.1%	18.9%	75.0%
	Mean	62	59	47	12	6.2%	23.2%	70.6%
Location 4P 700 ft	85th	70	71	63	19	-2.5%	14.8%	87.7%
	Mean	63	65	56	14	-3.1%	17.5%	85.6%
Location 5P 990 ft	85th	75	74	70	29	2.6%	8.8%	88.6%
	Mean	69	68	62	23	1.9%	13.2%	84.9%
*Note: Speed reduction percentage=speed reduction/total speed reduction from deceleration lane start point to the off-ramp end point								

(b)

Site		Speed (mph)				Speed Reduction Percentage*		
		Taper Start	Deceleration Lane Start	Deceleration Lane End	Off-Ramp End	Taper	Deceleration Lane	Off-Ramp
Location 1T 425 ft	85th	70	68	65	31	4.3%	8.1%	87.6%
	Mean	64	63	59	20	4.0%	7.5%	88.5%
Location 2T 320 ft	85th	73	71	66	21	3.7%	10.0%	86.3%
	Mean	64	63	57	16	3.8%	11.3%	84.9%
Location 3T 420 ft	85th	67	66	62	28	4.3%	10.0%	85.7%
	Mean	61	60	55	20	4.4%	11.7%	83.9%
Location 4T 445 ft	85th	68	68	65	8	-0.3%	5.8%	94.5%
	Mean	64	64	59	0	1.1%	7.6%	91.3%
Location 5T 365 ft	85th	73	72	68	37	3.4%	10.3%	86.3%
	Mean	67	66	63	28	2.2%	8.3%	89.5%
*Note: Speed reduction percentage=speed reduction/total speed reduction from deceleration lane start point to the off-ramp end point								

The speed reduction percentage is the percentage of speed reduced at the taper, deceleration lane, and off-ramp section. As shown in **Table 17**, the high percentage of the speed reduction occurred on off-ramps, which revealed that speed reduction was more significant on off-ramps than deceleration lanes. However, NCHRP Report 730 made a different indication (Torbic, et al. 2012). It should be mentioned that NCHRP Report 730 did not include the speed and deceleration along the entire deceleration lane and off-ramp but only several points (Torbic, et al. 2012) . The authors indicated that drivers were completing much of the required deceleration in the freeway lane upstream of the beginning of the taper when they found field-measured deceleration rates were less than the *Green Book* assumptions (Torbic, et al. 2012). This indication is very different from our results. For example, our results showed that Location 1P only had a 16% speed reduction in mean speed distribution on deceleration lanes and approximately 77% on off-ramps. When comparing parallel-design locations with tapered-design locations, it was found that tapered-design locations have higher speed reduction percentages on off-ramps in the range of 84% to 95%, while parallel-design locations have 70% to 88% speed reduction. The speed- reduction percentage on the deceleration lane and off-ramp indicated that drivers decelerated more on an off-ramp than on the deceleration lane. Also, some negative speed reduction percentages were observed, which implied that drivers may have accelerated on the taper section at three out of five parallel-design locations.

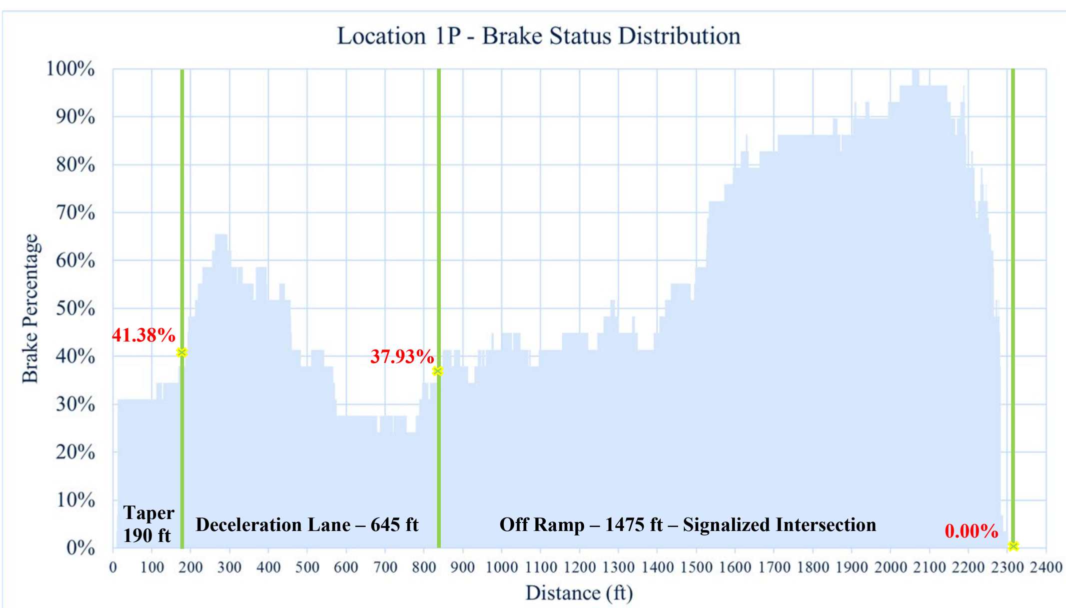
Moreover, longer deceleration lanes may not lead to higher speed reduction percentages. For both types of locations, the study sites with the longest deceleration lanes have the lowest speed reduction percentages. Location 5P with a 990-ft deceleration lane only had 8.81% speed reduction on it. Location 4T with a 445-ft deceleration lane only had 5.80% speed reduction on it. However, shorter deceleration lanes do not result in higher speed reduction percentages either. The locations with highest speed reduction percentages are median lengths – Location 2P (735-ft deceleration lane) and Location 3T (420-ft deceleration lane).

5.2.2 Driver Braking Behavior

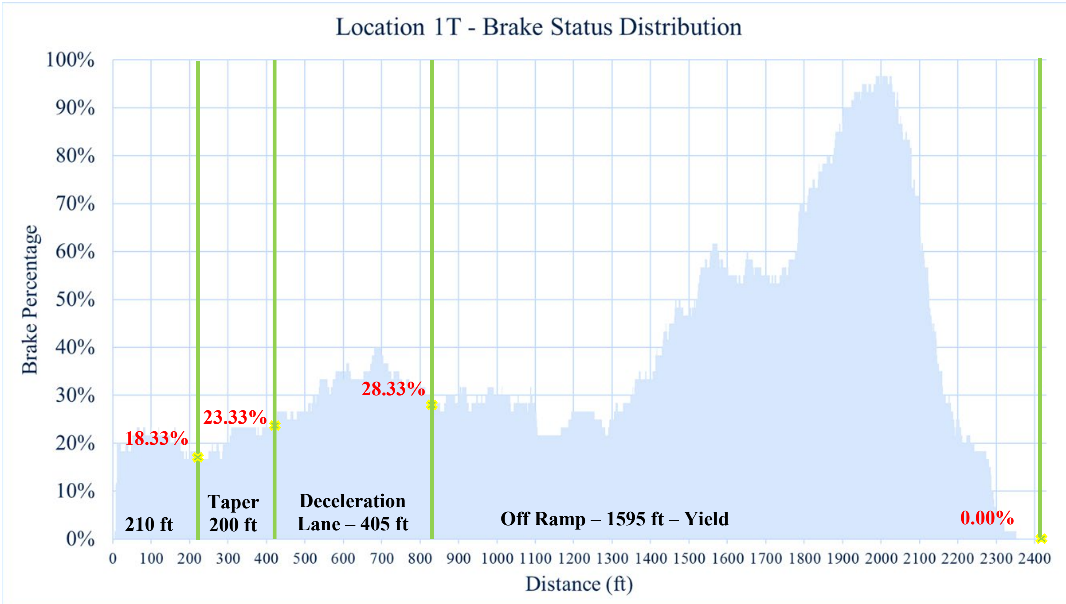
Driver braking behavior was interpreted by the brake pedal usage and deceleration rate distribution on the deceleration lane and off-ramp.

5.2.2.1 Brake Pedal Usage

The brake status (0 or 1) indicates whether the driver was applying the brake at the certain 0.1 seconds. The brake status distribution was performed based on the percentage of drivers who applied brakes at certain sections on the deceleration lane and off-ramp. **Figure 35*a*** shows brake status distribution at Location 1P and Location 1T. At Location 1P, only 30% of drivers applied brakes when entering the taper section. An increase to 60% of drivers applied brakes when entering the deceleration lane while a decrease back to 30% happened after traversing the first half of the deceleration lane. More braking behavior was observed after the vehicle approached the off-ramp terminals. Similar results from Location 1T were presented in **Figure 35*b***. From ten study locations, the average brake percentages for taper, deceleration lane, and off-ramp sections are 21.42%, 30.30%, and 63.67% in parallel-design locations, respectively, and 25.23%, 32.51%, and 57.69% in tapered-design locations, respectively. The brake pedal usage was also performed in other study locations, which can be found in Appendix D.



(a)

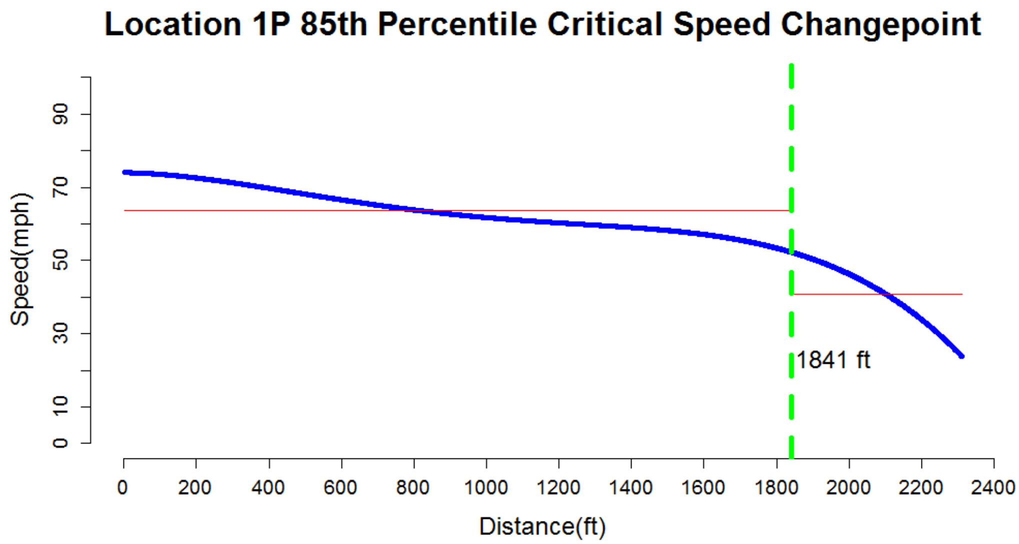


(*b*)

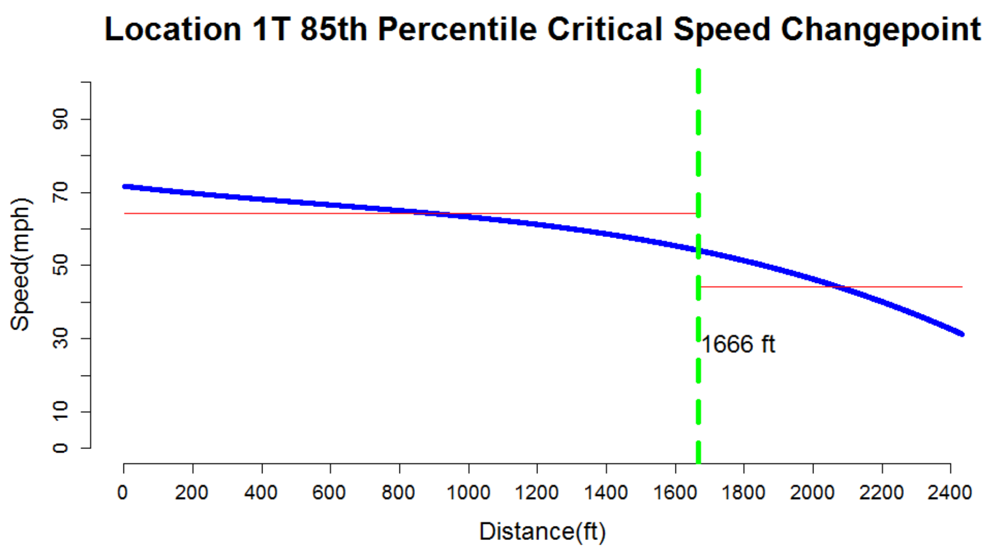
Figure 35 Brake status distribution: (*a*) Location 1P; and **(*b*) Location 1T**

5.2.2.2 Deceleration Rate Distribution

To calculate the deceleration rates, the speed-distance-based model, for example, Equations 6 to 9, was first converted to the speed-time-based model as time can be calculated from the distance and speed. Then, the first derivative of this speed-time-based model was determined. This first derivative is the deceleration rate from speed regression. The mean and 85th percentile deceleration rates will be summarized. An extra step was taken to identify the critical speed changepoint on the off-ramp. As greater speed reductions and higher brake percentages were observed upstream from the off-ramp terminal, change point models were used to identify driver reaction point where most drivers decelerate very hard when approaching the ramp terminal. Two examples of critical speed changepoint analysis are presented in **Figure 36.** In Location 1P, drivers adjusted their speed 469-ft upstream of the off-ramp terminal (1,841 ft after the taper start point) from the 85th percentile speed distribution. For Location 1T as shown in **Figure 36*b***, this number was increased to 764 ft (1,666 ft after the taper start point). The average reaction points for parallel-design locations are 540 ft in 85th percentile speed and 542 ft in mean speed. For tapered-design locations, the critical speed changepoints are 647 ft in 85th percentile speed and 652 ft in mean speed upstream from the ramp terminal. The 85th percentile critical speed changepoint was calculated for all locations, which can be found in Appendix E.



(a)



(b)

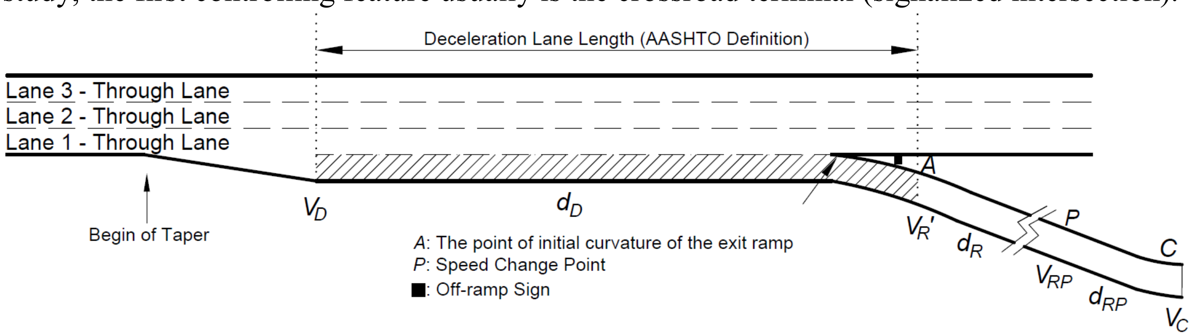
Figure 36 Critical speed changepoint: (*a*) Location 1P; and **(*b*) Location 1T**

The R statistical package of changepoint was utilized for critical speed changepoint detection based on binary segmentation algorithms. After the changepoints are detected, the deceleration rate before and after the changepoint on the off-ramps can also be obtained. The mean and 85th percentile deceleration rates were compared with the *Green Book* criterion which assumes a constant deceleration (AASHTO 2018). The *Green Book* deceleration rates were derived from recommended minimum deceleration lane lengths as summarized in NCHRP Report 730 (Torbic, et al. 2012). As shown in **Table 18**, most of the naturalistic driving deceleration rates were lower than the design deceleration rates in the *Green Book*. However, the deceleration rates after the changepoint on the off-ramp were relatively higher than other sections, and some of them were even greater than the design rates. For parallel-design deceleration lanes, the deceleration rates on the deceleration lane were slightly higher than that on the tapered-design locations. In NCHRP

Report 730, however, the authors observed that parallel deceleration lanes had a substantially higher deceleration rate of more than twice than tapered-design ones especially on straight ramps (Torbic, et al. 2012). All deceleration rates on the deceleration lane were much smaller than the *Green Book* criterion. The mean deceleration rates on certain sections of parallel-design and tapered-design locations were summarized in the last four rows in **Table 18**. It can be found that the *Green Book* assumes that drivers are exiting the freeway with a constant deceleration rate, while the results of this study indicate that drivers' braking behavior on the taper section, deceleration lane section, and off-ramp section are different with different deceleration rates.

5.2.3 Determination of the Minimum Length of Deceleration Lane

Equations 10 to 12 were developed to estimate the minimum deceleration lane length. The general idea of determining the minimum length for deceleration lane is to calculate the deceleration distance needed to decelerate from mainline speeds to ramp terminal speeds and subtract the certain off-ramp length. In other words, the minimum deceleration lane length is equal to the deceleration distance less the off-ramp length. The minimum deceleration length can be determined by plugging in the deceleration rates from the deceleration lanes (d_D) and off-ramps (d_R and d_{RP}) sections in **Table 18**, entering speed for the deceleration lane (V_D), and estimating entering speed for the exit ramp (V_R'), the changepoint on the off-ramp (V_{RP}), and the first controlling feature on off-ramp (V_C) from regression models. The controlling feature represents whether ramp curvature or the crossroad terminal is the design element that controls vehicle deceleration (Torbic, et al. 2012). On the relatively straight ramps at locations described in this study, the first controlling feature usually is the crossroad terminal (signalized intersection).



$$\begin{cases} L_{Decel} = L_Q, & V_D \leq V_R' \\ L_{Decel} = \dfrac{(1.47V_R')^2 - (1.47V_D)^2}{2d_D} + L_Q, & V_D > V_R' \end{cases} \tag{10}$$

$$V_R' = \frac{\sqrt{(1.47V_{RP})^2 - 2d_R L_R}}{1.47} \tag{11}$$

$$V_{RP} = \frac{\sqrt{(1.47V_C)^2 - 2d_{RP} L_{RP}}}{1.47} \tag{12}$$

Where, L_{Decel} = Deceleration lane length, ft
L_Q = Queue length at the off-ramp terminal, ft

L_R = Length from deceleration lane endpoint to the critical speed changepoint upstream from the first controlling feature on the off-ramp, ft
L_{RP} = Length from the critical speed changepoint to the off-ramp terminal, ft
V_C = Speed at the first controlling feature on the off-ramp, mi/h
V_D = Entering speed for deceleration lane, mi/h
V_R' = Estimated entering speed for the off-ramp, assuming drivers decelerate on L_R with a constant deceleration rate on exit ramps (d_R), mi/h
V_{RP} = Speed at the changepoint on the off-ramp, mi/h
d_D = Deceleration rate on deceleration lane, ft/s^2
d_R = Deceleration rate on exit ramp, ft/s^2
d_{RP} = Deceleration rate after the critical speed changepoint on the off-ramp, mi/h

To determine the deceleration lane length, the key parameters are summarized in **Table 19**. For example, at parallel-design locations, the speed at stop bar of the off-ramp terminal (V_C) should be 0 mph and the deceleration rate (d_{RP}) is estimated to be -5.25 ft/s^2 on the off-ramp after the changepoint. The distance between the stop bar and the changepoint (L_{RP}) is 540 ft as mentioned previously. By applying Equation 12, the speed at the changepoint (V_{RP}) is 51.22 mph. When the total length of the off-ramp is 1,550 ft ($L_R = 1550 - 540 = 1010\ ft$), drivers would be able to comfortably reduce all the required speed on the off-ramp ($V_R' = 70mph = V_D$). For tapered-design locations, the final speed should also be 0 mph ($V_C = 0\ mph$) and the deceleration rate is -4.58 ft/s^2 after the changepoint ($d_{RP} = -4.58\ ft/s^2$). Following the same steps, it can be determined that no deceleration lane will be required for decelerating purpose with a 1,540 ft off-ramp. The proposed minimum deceleration lane lengths of study locations are presented in **Table 20**. As a result, Locations 1T, 5P, and 5T do not require a deceleration lane serving decelerating functions.

Table 18 Deceleration rates at study locations

Deceleration Rate (ft/s^2)		Taper	Deceleration Lane d_D	Off Ramp d_R		GB Decel Rate* (ft/s^2)
				Before Changepoint	After Changepoint	
Location 1P 645 ft	85th	-1.63	-2.34	-2.12	-5.72	-5.41
	Mean	-3.61	-2.24	-1.88	-5.19	
Location 2P 735 ft	85th	-2.47	-2.41	-2.32	-6.46	-5.41
	Mean	-0.35	-2.57	-2.55	-4.52	
Location 3P 775 ft	85th	-2.87	-1.79	-2.76	-3.53	-5.41
	Mean	-2.67	-1.91	-2.20	-2.47	
Location 4P 700 ft	85th	0.18	-1.93	-2.89	-5.09	-5.88
	Mean	0.17	-1.95	-2.20	-4.20	
Location 5P 990 ft	85th	-0.98	-0.92	-2.15	-5.45	-5.88
	Mean	-0.52	-1.01	-1.77	-4.55	
Location 1T 425 ft	85th	-1.28	-1.12	-2.06	-2.94	-5.88
	Mean	-1.26	-1.10	-2.12	-2.69	
Location 2T 320 ft	85th	-2.08	-2.53	-3.77	-5.22	-5.88
	Mean	-1.78	-2.46	-3.35	-3.61	
Location 3T 420 ft	85th	-1.23	-1.27	-2.28	-4.48	-5.88
	Mean	-1.37	-1.48	-1.90	-4.12	
Location 4T 445 ft	85th	0.08	-1.90	-2.60	-7.03	-5.41
	Mean	-0.88	-1.63	-3.08	-5.40	
Location 5T 365 ft	85th	-1.50	-1.55	-1.50	-3.24	-5.88
	Mean	-0.96	-1.36	-1.62	-2.87	
Parallel-Design	85th	-1.55	-1.88	-2.45	-5.25	Note: *GB Decel Rate is the deceleration rate recommended in the *Green Book*.
	Mean	-1.40	-1.94	-2.12	-4.19	
Tapered-Design	85th	-1.20	-1.67	-2.44	-4.58	
	Mean	-1.25	-1.61	-2.41	-3.74	

Table 19 Summary of key parameters to determine the deceleration lane length

Key Parameters		Parallel-Design		Tapered-Design	
		Minimum (85th)	Mean	Minimum (85th)	Mean
Deceleration Lane (ft/s^2) d_D		-1.88	-1.94	-1.67	-1.61
Off Ramp	Before Changepoint (ft/s^2) d_R	-2.45	-2.12	-2.44	-2.41
	After Changepoint (ft/s^2) d_{RP}	-5.25	-4.19	-4.58	-3.74
Speed Entering Dec.Lane (mph) V_D		70.00	65.00	69.00	63.00
Speed at the Changepoint on Off-Ramp (mph) V_{RP}		51.22	45.74	52.49	47.43
Speed at the 1st Controlling Feature (mph) V_C		0.00	0.00	0.00	0.00
Length from Changepoint to Off-Ramp terminal (ft) L_{RP}		540		650	

Table 20 Comparison of proposed deceleration lane length and design length

Site Locations	Proposed Minimum Length (ft) (85th) L_{Decel}	Actual Deceleration Lane Length (ft)	Green Book Minimum Deceleration Length (ft)	Off-Ramp Length (ft) L_R+L_{RP}
Location 1P: I-75/SW Archer Rd	75	645	490	1,475
Location 1T: I-75/Clark Rd	NA[1]	425	615	1,595
Location 2P: I-75/SW County Highway 484	560	735	490	990
Location 2T: I-75/US 98	600	320	615	940
Location 3P: I-75/FL 326	520	775	490	1,030
Location 3T: I-75/US 98	370	420	615	1,170
Location 4P: I-75/CR 768	370	700	615	1,180
Location 4T: I-75/SW College Rd	200	445	490	1,340
Location 5P: I-75/CR 765	NA[1]	990	615	1,690
Location 5T: I-75/CR 769	NA[1]	365	615	1,725
Note: [1]NA indicates that the deceleration lane is not required for decelerating purpose.				

6 CONCLUSIONS

This research project explored two innovative applications of SHRP 2 NDS data for studying driver behaviors at unsignalized intersections and freeway diverge areas. For the unsignalized intersection study, the driver visual workload, driver speed change, and stopping behavior were analyzed. For the freeway diverge area, the speed change distribution, driver brake behavior and the determination of the length of the freeway deceleration lane were included. Below is the summary of the conclusions and recommendations.

6.1 Unsignalized Intersection Study

For the unsignalized intersection study, three different driver behavior analyses were conducted (visual workload, speed change behavior, and stopping behavior). Various driver demographics (i.e. gender, age, income, education, driving years, crash records and risk perception levels) and roadway/traffic features (channelization on minor road, leg numbers, major-road speed limit, and AADT) were also analyzed to exam if they have an impact on the driver workload and behavior.

The key findings of these analyses are stated below:

<u>Driver Visual Workload</u>

1) Descriptive data analysis indicates that drivers making left turns at conventional intersections spent more time looking at their left and right mirrors and less time looking forward; however, drivers executing a right turn followed by a U-turn focused more on looking forward.
2) Average entropy rates of left-turn drivers at conventional intersections are significantly higher than indirect left-turn drivers at RCUT intersections. Drivers at conventional intersections have more random scanning and shorter average fixation durations during the movement. The result indicated that entropy rate can be a proper measure to quantify the cognitive demands of specific traffic movements.
3) Correlation analysis results show that some demographic features, such as education level, driving years, crash numbers in the past 3 years, and the risk perception scores are negatively correlated with the entropy rates. The results proved that the higher the education level, the longer the driving years, the more the history crash numbers, and the higher the risk perception scores, the lower the entropy rate/ driver visual workload. The analysis results suggest that younger drivers at both types of intersections have higher entropy rates than middle-aged and older drivers.
4) Statistical analysis results suggest that AADT has a significant impact on drivers' visual workload. Higher AADT ($\geq$ 30,000) on the major road will significantly increase drivers' visual workload compared with lower major road AADT ($\leq$ 20,000).

<u>Driver Speed Change Behavior</u>

1) The critical speed change points suggest that most of the drivers tend to decelerate sharply when they are about 120 ft or 50 ft away from the minor-road stop bar. To improve minor-road drivers' stopping behavior, an advance intersection warning sign can be installed at the distance of (120 ft+2.5 * minor-road speed limit) from the stop bar, assuming a 2.5 second perception and reaction time.

2) The study found that the 85th percentile of all the NDS drivers used less than 650 ft to accelerate to 45 mph from the median opening. It suggests that a 650 ft left-turn acceleration lane on the major road can accommodate 85% of drivers' need to accelerate to major road speed limit of 45 mph.
3) The analysis results found that the younger drivers drove more aggressively than middle-aged and older drivers at both minor roads and median openings.

<u>Driver Stopping Behavior</u>

1) Overall, the percentage of drivers stopping on minor road approaches is around 53%, which means a substantial percentage (47%) of drivers did not stop at minor road stop-controlled intersections. The result aligns with the past studies where 50% of the drivers were found not to stop at minor road speed stop-controlled intersections (Cody 2013).
2) Female drivers show more compliance in stopping at minor roads than male counterparts. The odds ratio for male drivers to stop at a stop sign is 0.64, which indicates that male drivers are 0.64 times less likely to stop than female drivers. Also, higher percentages of drivers not stopping at both stages are male. This result is analogous to the past study with a slight difference (Shaaban 2017).
3) Older drivers show more tendency to stop on minor road approaches than the younger drivers(16-24 years).
4) Drivers with more driving experience have higher percentages in stopping at minor road stop signs. Those with fewer than ten years of driving experience have a higher tendency to violate the rules. Similarly, drivers having previous crash experience show a better tendency to stop at minor road stop lines than their other counterparts. This indicates that they are more cautious with driving.
5) In general, the type of intersection (3-leg or 4-leg), speed limit, and AADT were found to have an impact on stopping behaviors. Further studies of more sites are needed to quantify their impacts.

Figure 37 shows the recommended placement of the advance warning sign and the minimum length of the left-turn acceleration lane on a major road at unsignalized intersections. The advance warning sign, together with the in-lane rumble strips, were recommended to be installed at approximately 120 ft +2.5* speed limit on the minor road before the intersection. At the median openings, stop sign/stop line or yield sign/yield line were recommended to improve stopping behavior at the median opening. On the major road, the minimum length (650 ft) of a left turn acceleration lane is recommended to accommodate 85% of drivers accelerating to 45 mph.

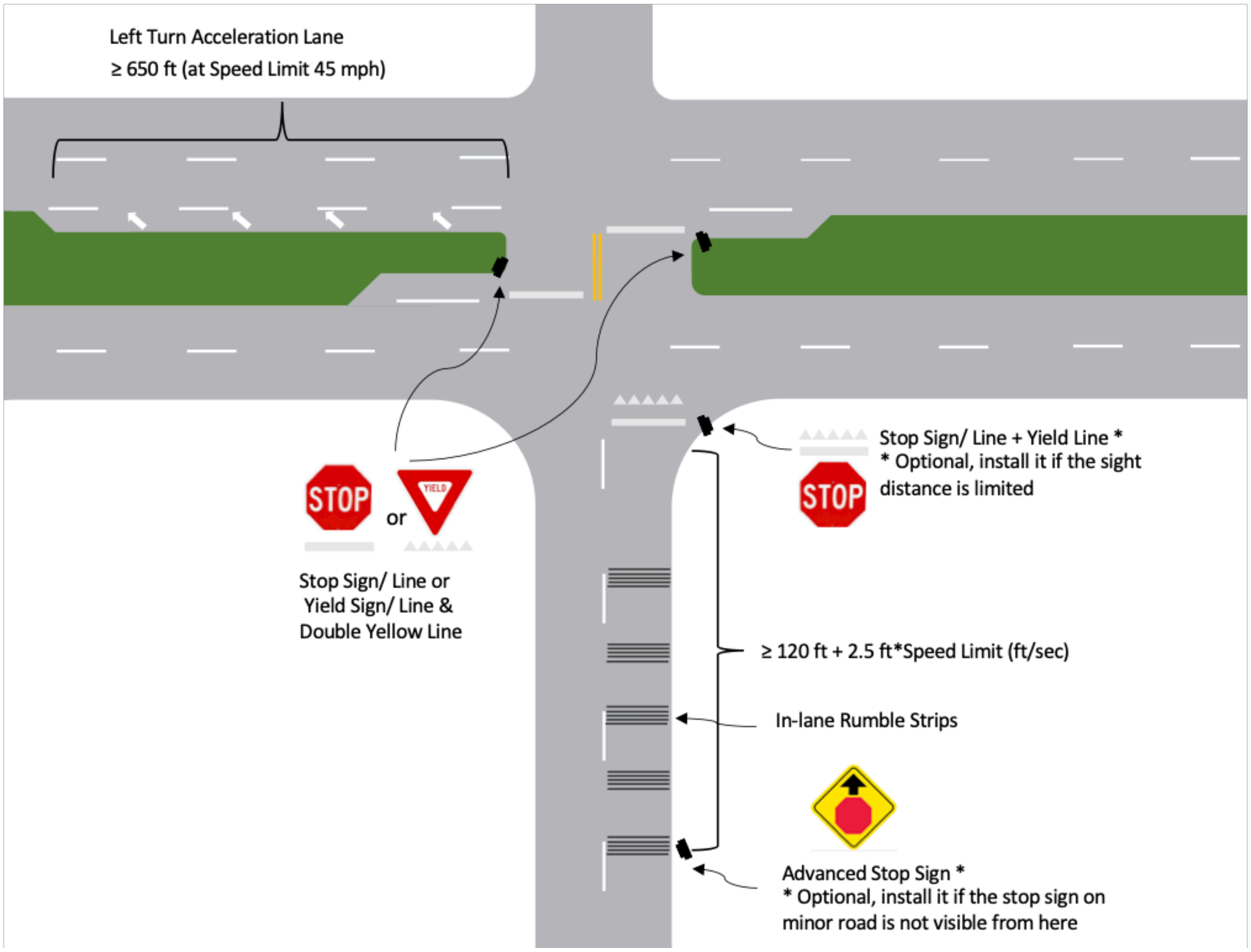


Figure 37 Recommended placement of advanced warning sign and minimum length of left-turn acceleration lane at unsignalized intersections

6.2 Freeway Diverge Area

1) The operating speeds at the freeway diverge areas were much higher than the *Green Book* assumptions. The *Green Book* states that on a freeway with a 70-mph design speed, drivers will enter the deceleration lane at 58 mph. In the five parallel-design locations, the speed distribution, however, showed that the average operating speed was 65 mph when vehicles entered the deceleration lane. The *Green Book* also assumed that the speed reached the end of the deceleration lane with a ramp of 35 mph design speed should be 30 mph, which instead was 55 mph on average based on this study.
2) Drivers were not effectively using the deceleration lane regarding the speed reduction. From speed distribution results, for parallel-design locations, the speed reduction on the deceleration lane is approximately 15% to 25%. The percentage of the speed reduced on the off-ramp is 75% to 85%, which indicates that the speed reduced much more when vehicles approached closer to the off-ramp terminal. For tapered-design locations, the speeds reduced on the deceleration lane were even lower, 10% speed reduction on deceleration lanes and 85% to 90% on off-ramps.

3) The brake status distribution further emphasized that the effective deceleration segment is on the off-ramp rather than the deceleration lane. The average brake pedal usage on off-ramps is much higher than that on deceleration lanes on average (26% for the taper section, 37% for the deceleration lane section, and 54% for the off-ramp section).
4) The results from critical speed changepoint models also implied that drivers' reaction points of sharp deceleration were on the off-ramp upstream of the ramp terminal. The average distances of critical speed changepoints from the terminal are 540 ft for parallel-design locations and 650 ft for tapered-design locations.
5) The calculated mean and 85th percentile deceleration rates were dynamic, while the *Green Book* criterion assumes constant values for the entire decelerating maneuver. It was found that the deceleration rates on the deceleration lane were much lower than those on the off-ramp after the critical speed changepoint. Most of the deceleration rates on the deceleration lane and off-ramp at study locations were lower than constants provided by the *Green Book*; however, some were higher after the critical speed changepoint.
6) Based on the speed and deceleration rate distribution, a new method was developed to determine the minimum length of the deceleration lane. The results indicated that a deceleration lane might not be required for serving a decelerating purpose on both parallel- and tapered-design deceleration lane locations when the ramp length is more than 1,550 ft. This number is specific to the diamond interchange (or interchanges with relatively straight off-ramps) with a 70 mph speed limit on the mainline with a stop controlled off-ramp terminal. For off-ramps with relatively high volume, the queue length should be added to the 1,550 ft.

In addition to the original objectives of this work, this study enabled the following observations. The advisory speeds posted on off-ramps had no significant impact on drivers' operating speeds. For locations with a 35 mph advisory speed, the average 85th percentile speed and mean speed are 63 mph and 55 mph, respectively. For those without advisory speeds, the 85th percentile speed is 65 mph, and the mean speed is 58 mph. Thus, based on the speed distribution and critical speed change points, the speed advisory sign can be installed before the critical speed change point on off-ramps for the better driver expectance.

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APPENDIX A: Driver Stopping Behavior at Unsignalized Intersections

This section summarizes the analysis of Driver Stopping Behavior:

Table A-1 Number of participants and percentages by each category

Socioeconomic and demographic factors			
Variable name	**Variable sub-category**	**Total Number**	**Percentage of each sub-category**
Gender	Female	32	49%
	Male	33	51%
Age	Young (16-24)	20	31%
	Mid Aged (25-64)	23	35%
	Old (65 and above)	22	34%
Education	High School	14	21%
	Beyond High School with no further degree	19	28%
	College	10	15%
	Graduate	14	21%
	Advanced	11	16%
Driving Related Factors			
Driving Experience (years)	Below 10 years	21	32%
	10 to 45 years	23	36%
	above 45 years	21	32%
Training	Informal	19	29%
	School	19	29%
	Informal and School	18	28%
	Other/not specified	9	14%
No. of violation in past three years	0	43	66%
	1 or more	22	34%
No. of Crash in past three years	0	45	69%
	1 or more	20	31%

Table A-2 Stopping behavior for each location

Location	**No. of participants**	**Total no. 0f trips**	**Stopped at minor road stop sign**	**Stopped at both minor and median**	**Stopped at median only**	**Did not stop at all**
FL1	17	94	51%	5%	14%	35%
FL2	6	161	65%	1%	9%	26%
FL3	8	29	70%	12%	2%	28%
FL5	8	40	57%	5%	4%	39%
FL6	5	40	80%	27%	1%	19%
FL7	21	64	38%	5%	1%	61%

Total	65	428	53%	7%	6%	41%

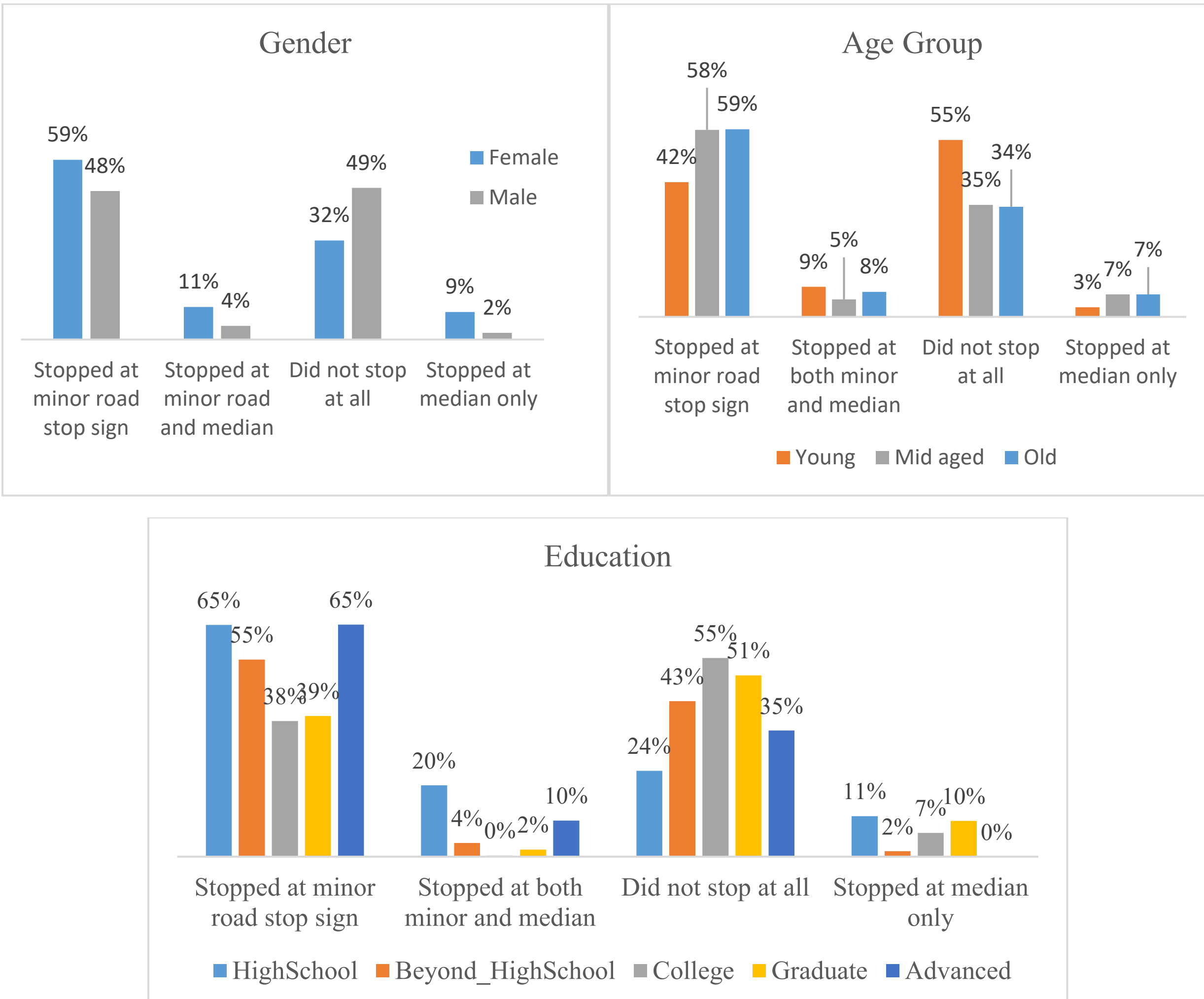


Figure A-1 Driving behavior analysis: demographic factors

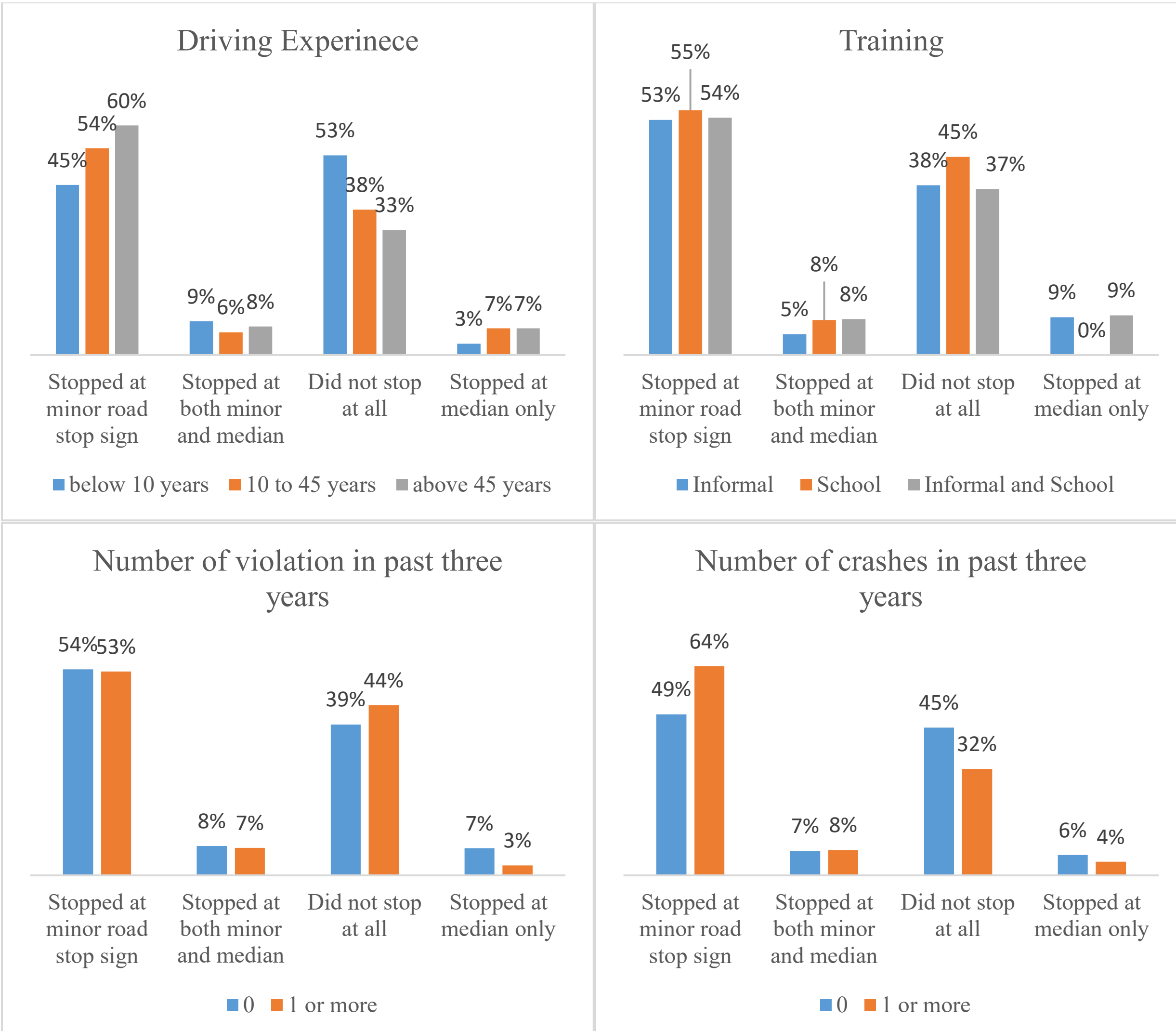
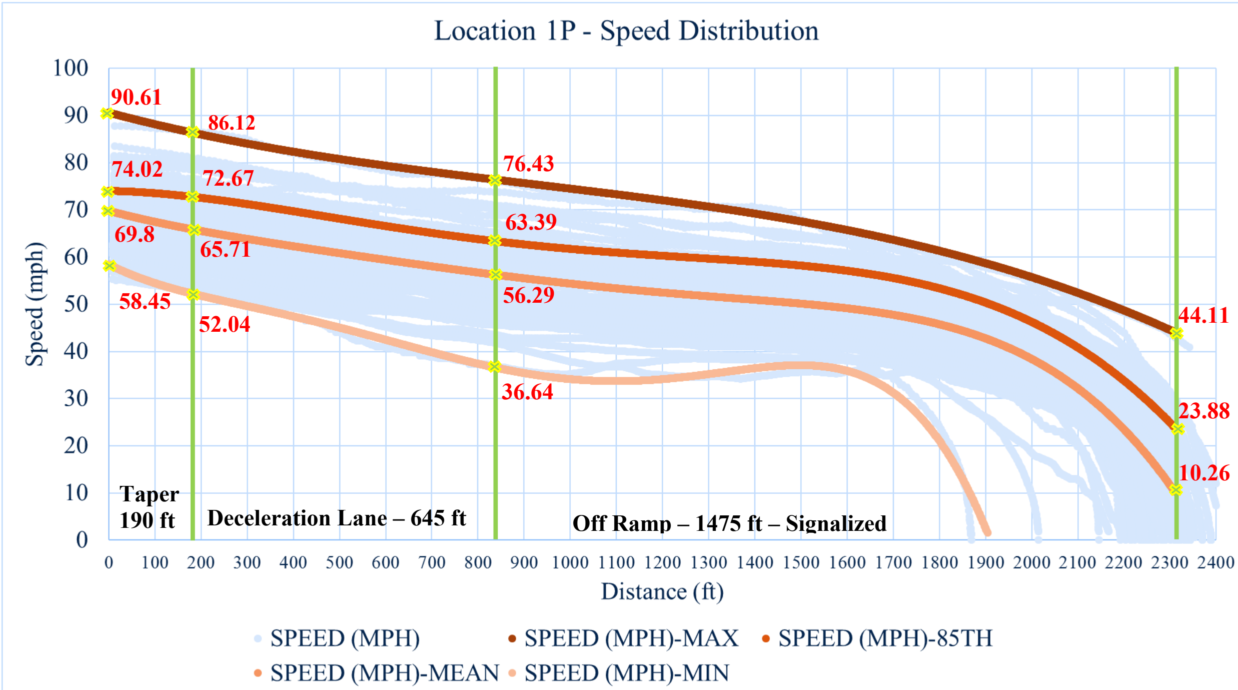
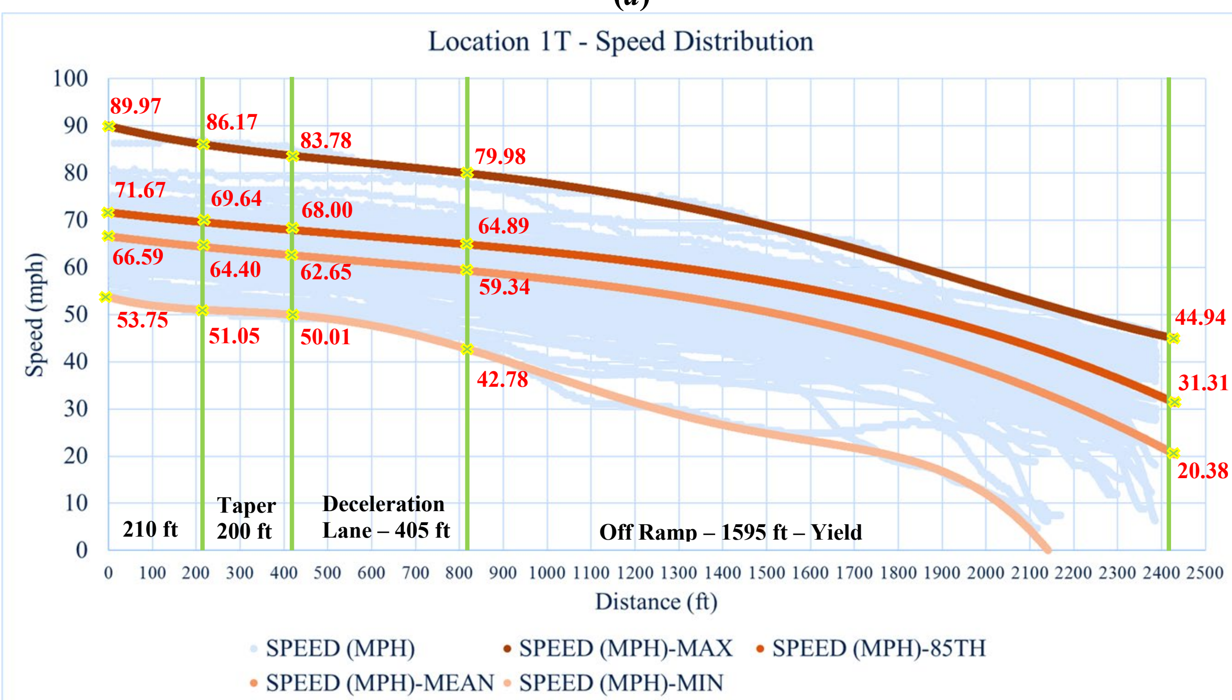


Figure A-2 Driving behavior analysis: driver-related factors

APPENDIX B: Speed Distributions on Freeway Deceleration Lane

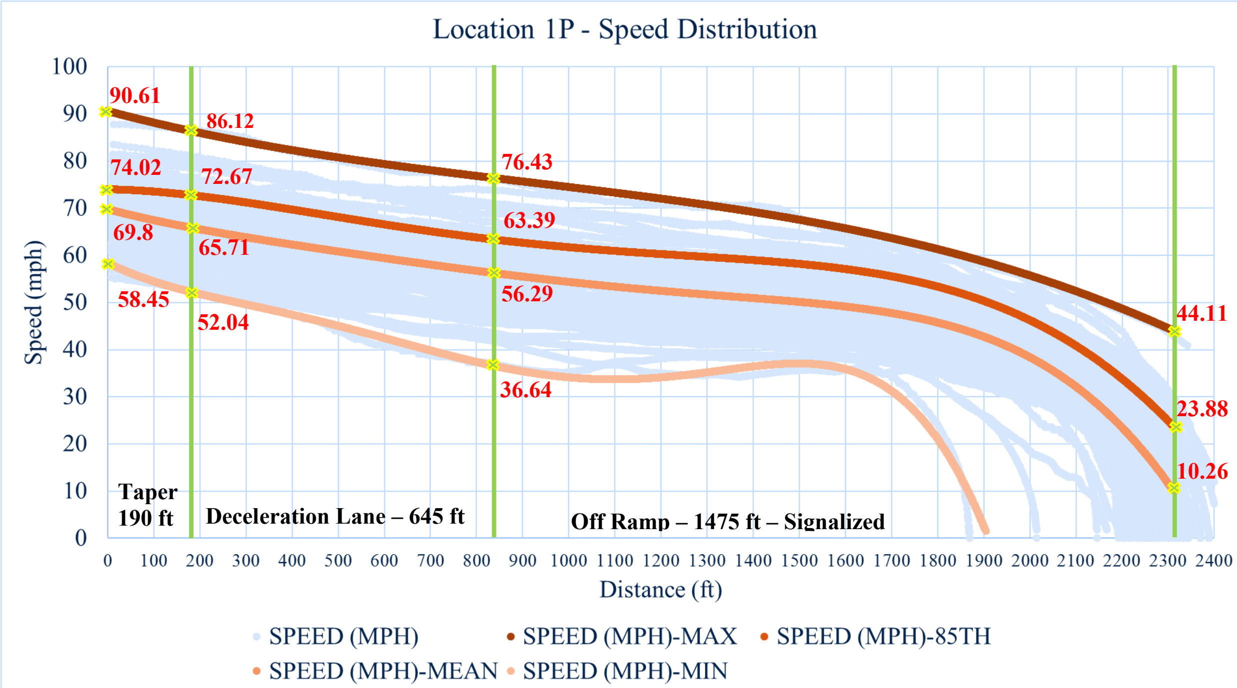


(*a*)

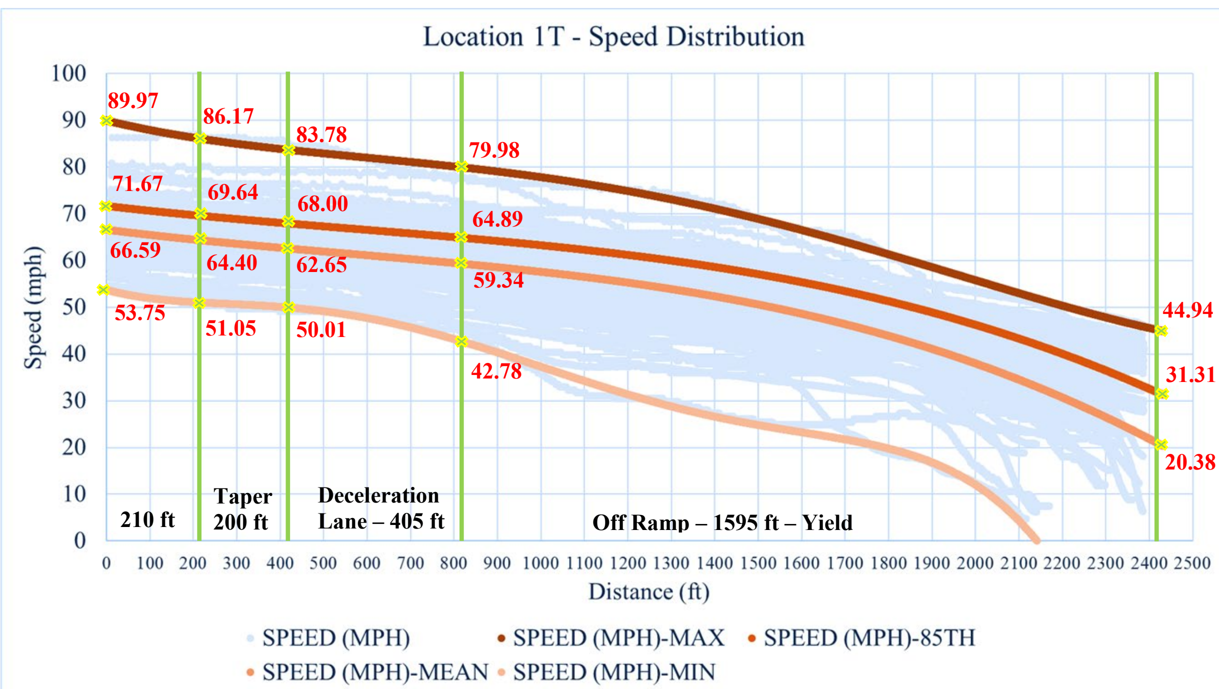


(*b*)

Figure B-1 Speed distributions: (*a*) Location 1P; and (*b*) Location 1T

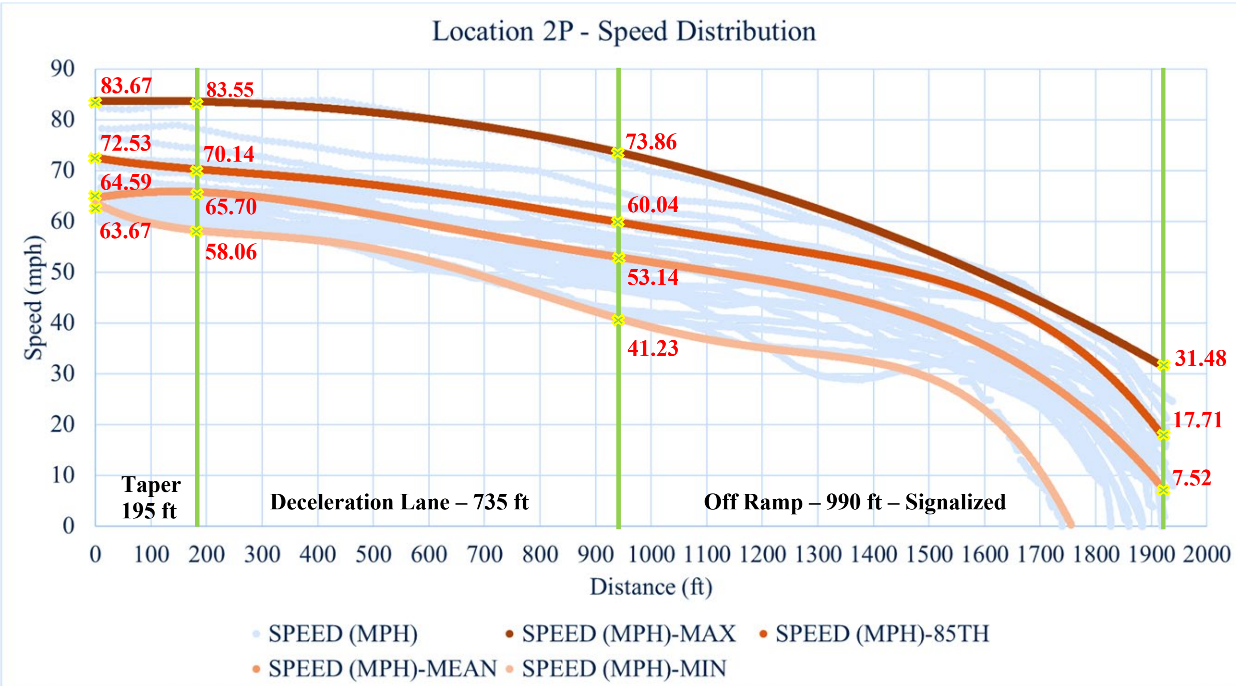


(a)

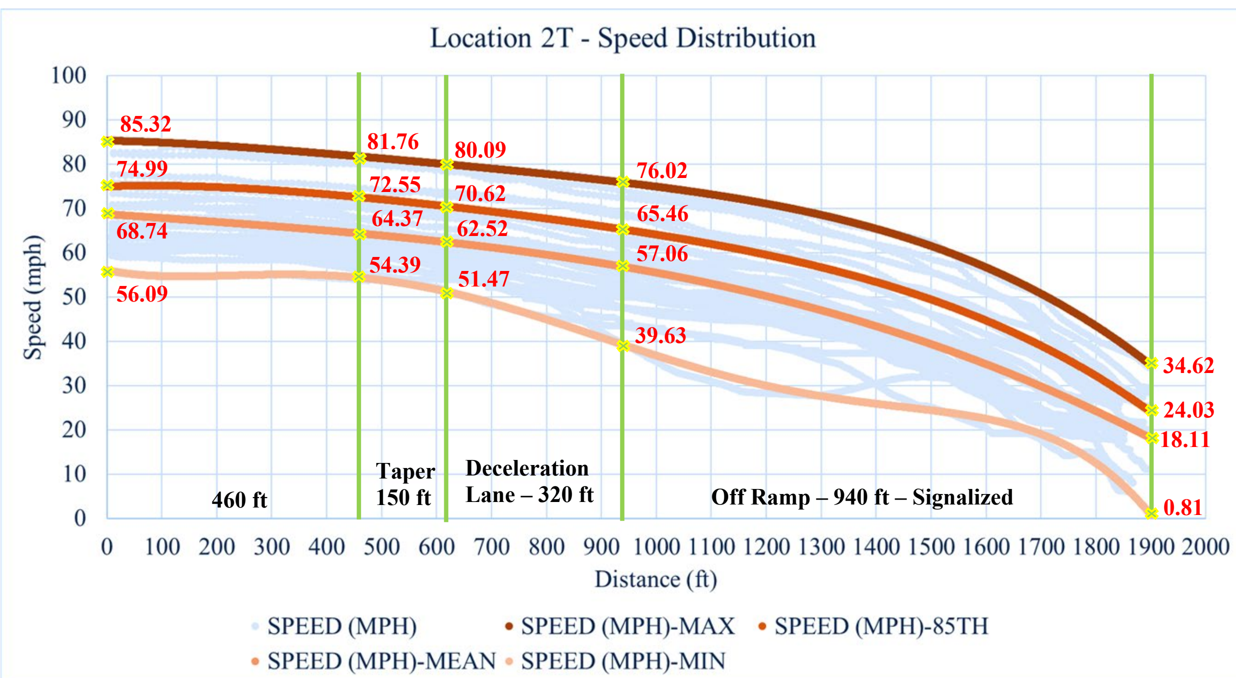


(b)

Figure B-2 Speed distributions: (*a*) Location 1P; and (*b*) Location 1T

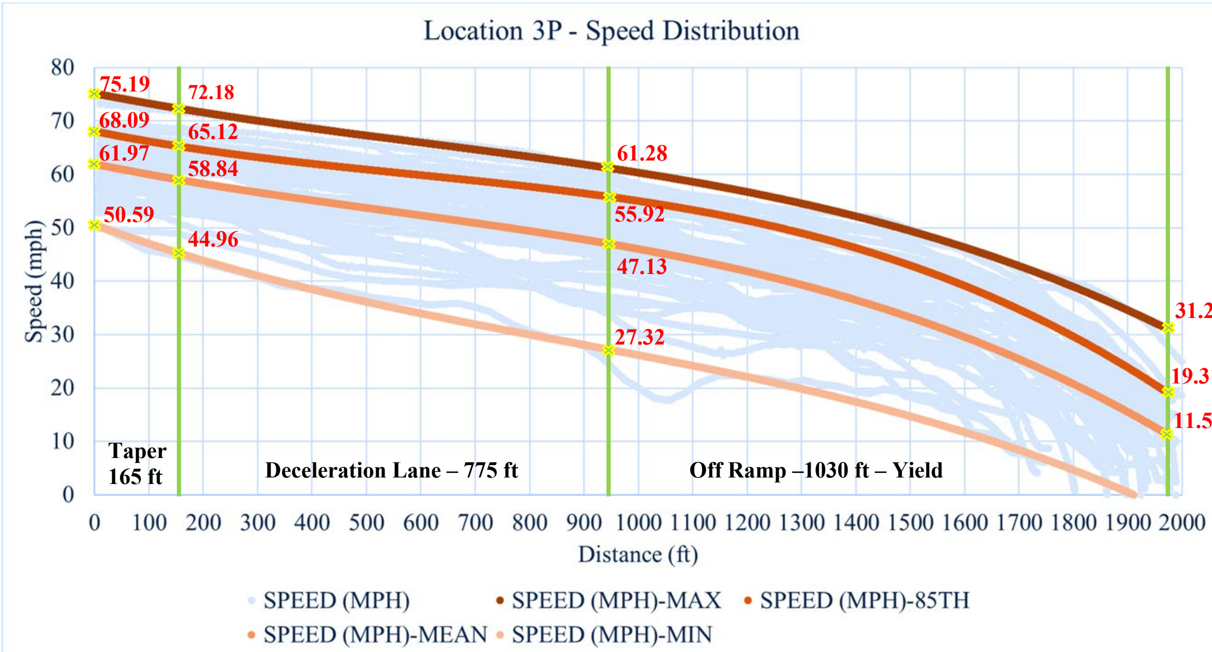


(a)

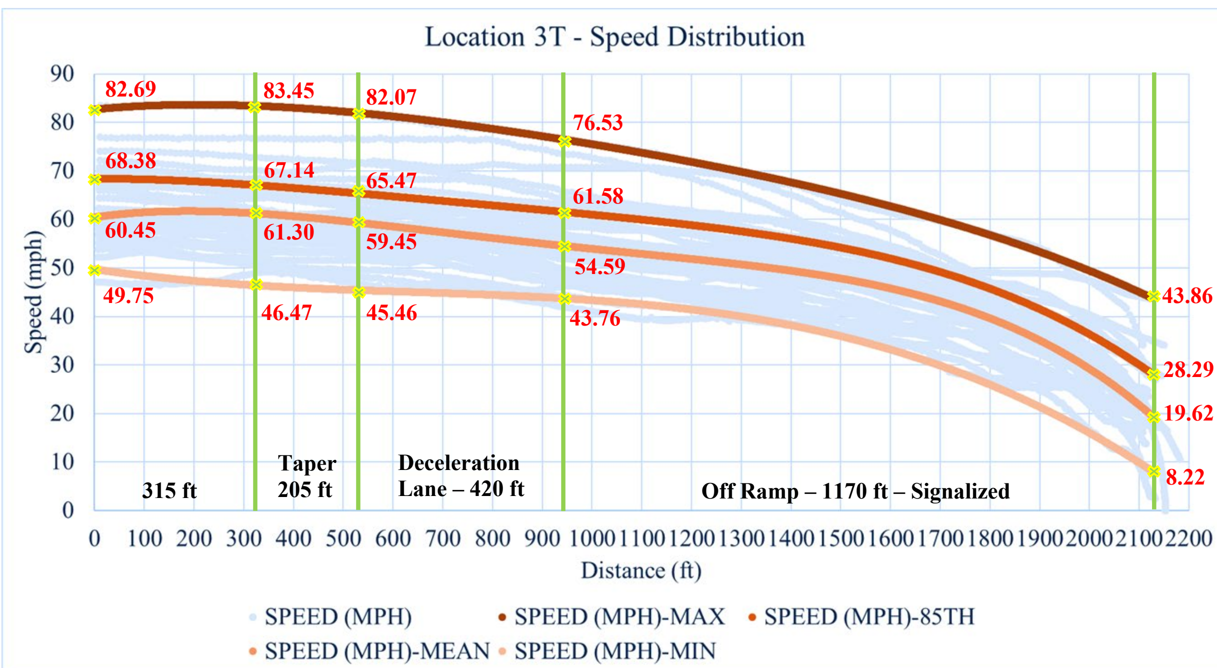


(b)

Figure B-3 Speed distributions: (*a*) Location 2P; and (*b*) Location 2T

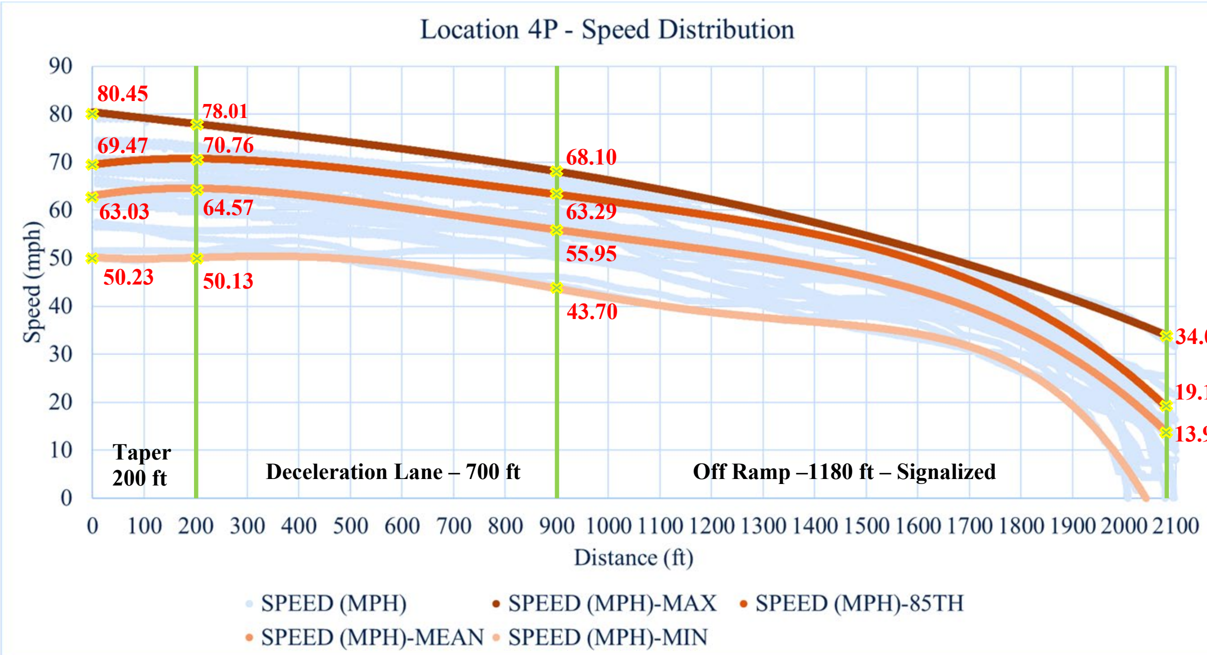


(a)

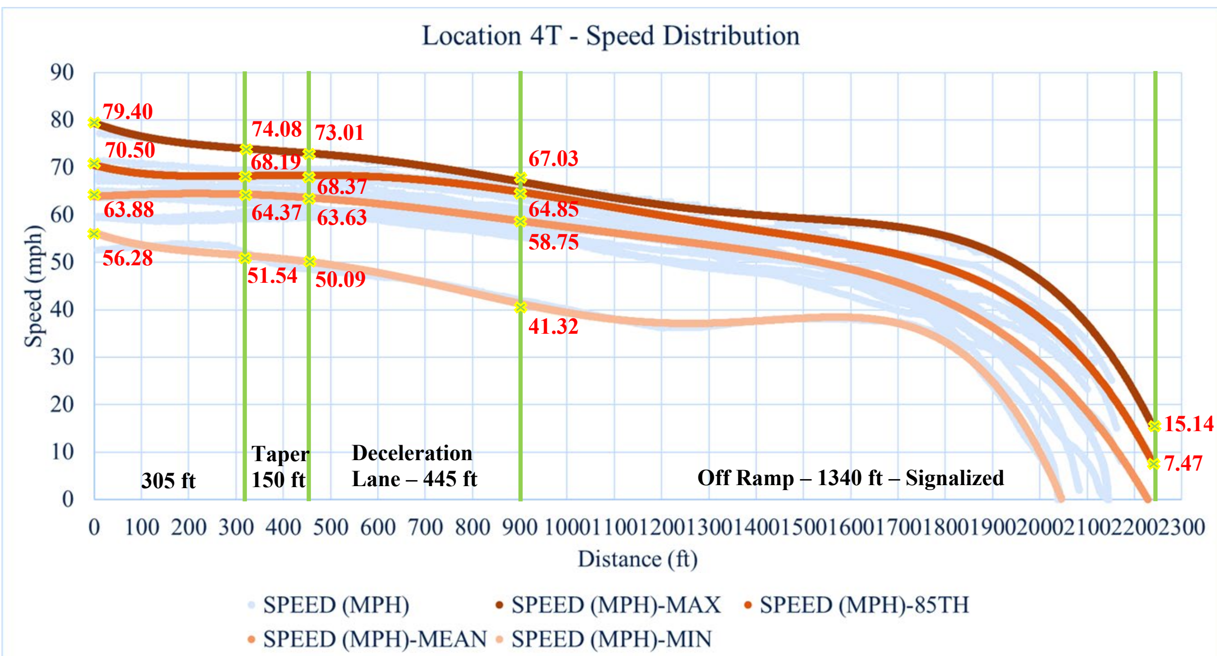


(b)

Figure B-4 Speed distributions: (*a*) Location 3P; and (*b*) Location 3T

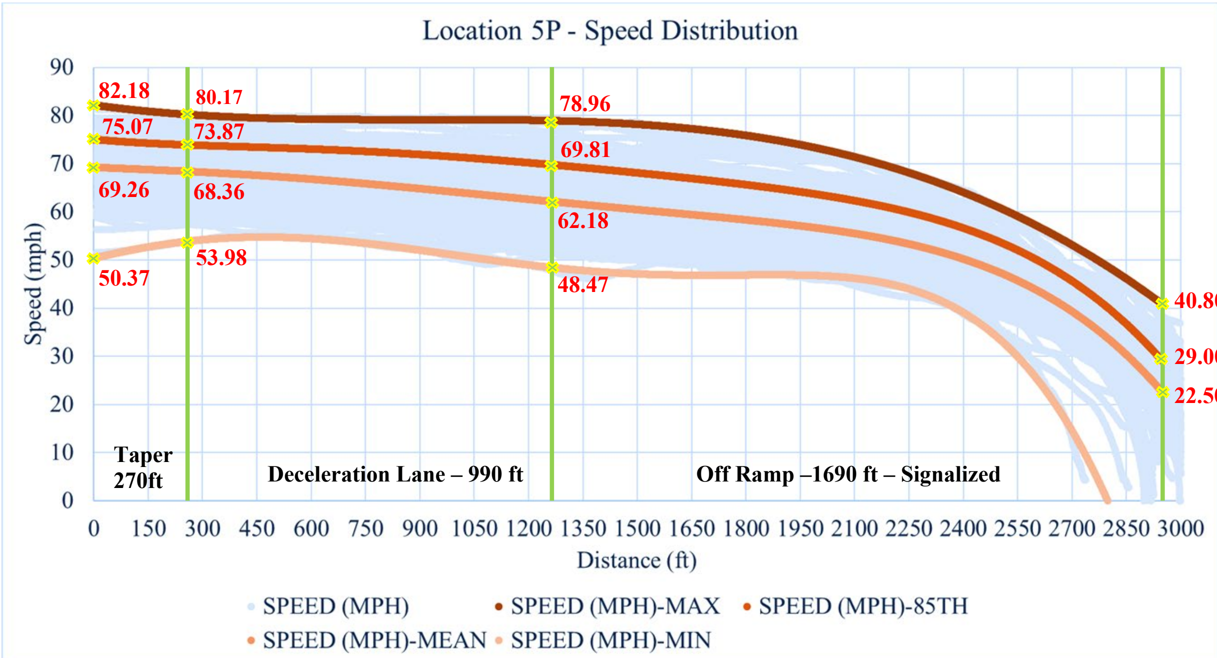


(*a*)

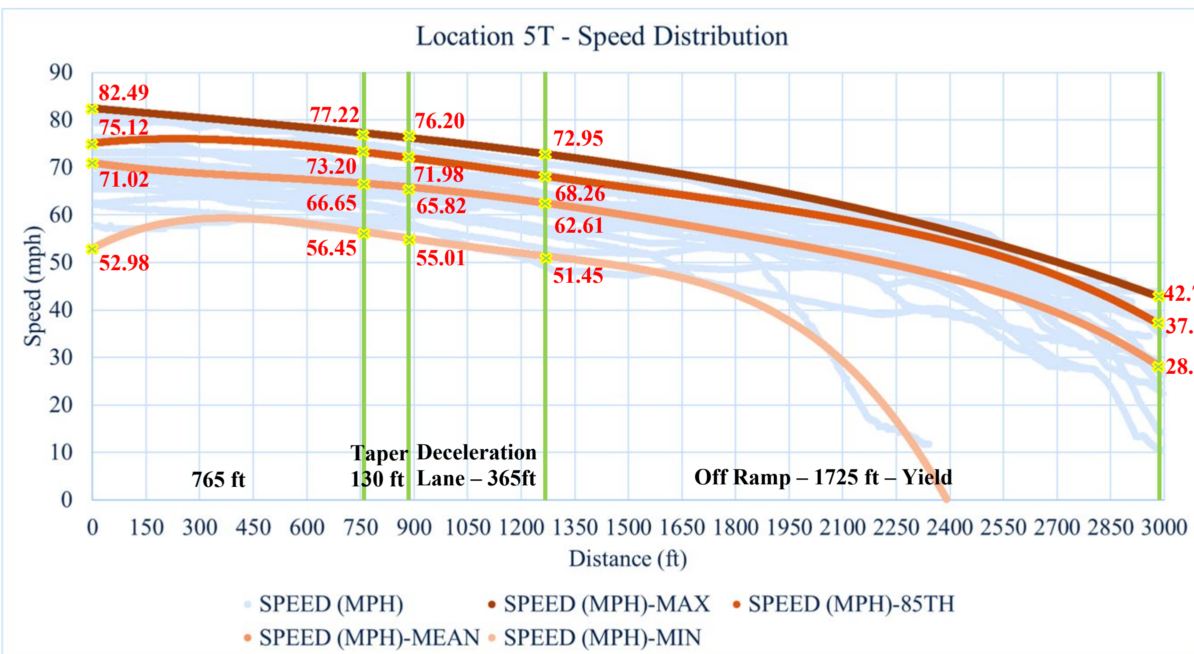


(*b*)

Figure B-5 Speed distributions: (*a*) Location 4P; and (*b*) Location 4T



(a)



(b)

Figure B-6 Speed distributions: (*a*) Location 5P; and (*b*) Location 5T

Table B-1 Max, 85th percentile, mean, and min speed at freeway diverge areas

Site	Speed (mph)					
			Taper Start	Deceleration Lane Start	Deceleration Lane End	Off-Ramp End
Location 1P	Max	-	90.61	86.18	76.43	44.11
	85th	-	74.02	72.67	63.39	23.88
	Mean	-	69.80	65.71	56.29	10.26
	Min	-	58.45	52.04	36.64	0.00
		210 ft				
Location 1T	Max	89.97	86.17	83.78	79.98	44.94
	85th	71.67	69.64	68.00	64.89	31.31
	Mean	66.59	64.40	62.65	59.34	20.38
	Min	53.75	51.05	50.01	42.78	0.00
Location 2P	Max	-	83.67	83.55	73.86	31.48
	85th	-	72.53	70.14	60.04	17.71
	Mean	-	64.59	65.70	53.14	7.52
	Min	-	63.67	58.06	41.23	0.00
		460 ft				
Location 2T	Max	85.32	81.76	80.09	76.02	34.62
	85th	74.99	72.55	70.62	65.46	24.03
	Mean	68.74	64.37	62.52	57.06	18.11
	Min	56.09	54.39	51.47	39.63	0.81
Location 3P	Max	-	75.19	72.18	61.28	31.22
	85th	-	68.09	65.12	55.92	19.31
	Mean	-	61.97	58.84	47.13	11.50

	Min	-	50.59	44.96	27.32	0.00
		315 ft				
Location 3T	Max	82.69	83.45	82.07	76.53	43.86
	85th	68.38	67.14	65.47	61.58	28.29
	Mean	60.45	61.30	59.45	54.59	19.62
	Min	49.75	46.47	45.46	43.76	8.22
Location 4P	Max	-	80.45	78.01	68.10	34.05
	85th	-	69.47	70.76	63.29	19.14
	Mean	-	63.03	64.57	55.95	13.95
	Min	-	50.23	50.13	43.70	0.00
		305 ft				
Location 4T	Max	79.40	74.08	73.01	67.03	15.14
	85th	70.50	68.19	68.37	64.85	7.47
	Mean	63.88	64.37	63.63	58.75	0.00
	Min	56.28	51.54	50.09	41.32	0.00
Location 5P	Max	-	82.18	80.17	78.96	40.80
	85th	-	75.07	73.87	69.81	29.00
	Mean	-	69.26	68.36	62.18	22.50
	Min	-	50.37	53.98	48.47	0.00
		765 ft				
Location 5T	Max	82.49	77.22	76.20	72.95	42.78
	85th	75.12	73.20	71.98	68.26	37.05
	Mean	71.02	66.65	65.82	62.61	28.18
	Min	52.98	56.45	55.01	51.45	0.00

APPENDIX C: Regression Models on Speed Distribution at Freeway Diverge Aeras

For Location 1P:

$$v_{1P-Max} = -5.633 \times 10^{-9} {L_{1P}}^3 + 1.558 \times 10^{-5} {L_{1P}}^2 - 2.606 \times 10^{-2} \times L_{1P} + 90.61$$
$$R^2 = 0.9963$$
$$v_{1P-85^{th}} = -9.767 \times 10^{-12} {L_{1P}}^4 + 3.380 \times 10^{-8} {L_{1P}}^3 - 3.462 \times 10^{-5} {L_{1P}}^2 - 1.703 \times 10^{-3} \times L_{1P} + 74.02$$
$$R^2 = 0.9981$$
$$v_{1P-Mean} = -6.646 \times 10^{-15} {L_{1P}}^5 + 2.697 \times 10^{-11} {L_{1P}}^4 - 3.978 \times 10^{-8} {L_{1P}}^3 + 2.997 \times 10^{-5} {L_{1P}}^2 - 2.594 \times 10^{-2} L_{1P} + 69.80$$
$$R^2 = 0.9981$$
$$v_{1P-Min} = -4.970 \times 10^{-14} {L_{1P}}^5 + 1.866 \times 10^{-10} {L_{1P}}^4 - 2.340 \times 10^{-7} {L_{1P}}^3 + 1.226 \times 10^{-4} {L_{1P}}^2 - 4.982 \times 10^{-2} L_{1P} + 58.45$$
$$R^2 = 0.9600$$

For Location 1T:

$$v_{1T-Max} = 3.664 \times 10^{-12} {L_{1T}}^4 - 1.877 \times 10^{-8} {L_{1T}}^3 + 2.570 \times 10^{-5} {L_{1T}}^2 - 2.272 \times 10^{-2} \times L_{1T} + 89.97$$
$$R^2 = 0.9877$$
$$v_{1T-85^{th}} = -3.320 \times 10^{-9} {L_{1T}}^3 + 5.640 \times 10^{-6} {L_{1T}}^2 - 1.071 \times 10^{-2} L_{1T} + 71.67$$
$$R^2 = 0.9980$$
$$v_{1T-Mean} = -3.968 \times 10^{-9} {L_{1T}}^3 + 6.610 \times 10^{-6} {L_{1T}}^2 - 1.165 \times 10^{-2} L_{1T} + 66.59$$
$$R^2 = 0.9956$$
$$v_{1T-Min} = -2.108 \times 10^{-14} {L_{1T}}^5 + 1.012 \times 10^{-10} {L_{1T}}^4 - 1.630 \times 10^{-7} {L_{1T}}^3 + 9.239 \times 10^{-5} {L_{1T}}^2 - 2.597 \times 10^{-2} L_{1T} + 53.75$$
$$R^2 = 0.9922$$

For Location 2P:

$$v_{2P-Max} = -1.892 \times 10^{-9} {L_{2P}}^3 - 1.141 \times 10^{-5} {L_{2P}}^2 + 1.701 \times 10^{-3} \times L_{2P} + 83.67$$
$$R^2 = 0.9932$$
$$v_{2P-85^{th}} = -1.958 \times 10^{-14} {L_{2P}}^5 + 7.472 \times 10^{-11} {L_{2P}}^4 - 9.974 \times 10^{-8} {L_{2P}}^3 + 4.947 \times 10^{-5} {L_{2P}}^2 - 1.863 \times 10^{-2} \times L_{2P} + 72.53$$
$$R^2 = 0.9968$$
$$v_{2P-Mean} = -2.261 \times 10^{-11} {L_{2P}}^4 + 7.285 \times 10^{-8} {L_{2P}}^3 - 8.193 \times 10^{-5} {L_{2P}}^2 + 1.906 \times 10^{-2} L_{2P} + 64.59$$
$$R^2 = 0.9970$$
$$v_{2P-Min} = -8.148 \times 10^{-14} {L_{2P}}^5 + 3.177 \times 10^{-10} {L_{2P}}^4 - 4.281 \times 10^{-7} {L_{2P}}^3 + 2.262 \times 10^{-4} {L_{2P}}^2 - 5.882 \times 10^{-2} L_{2P} + 63.67$$
$$R^2 = 0.9825$$

For Location 2T:

$$v_{2T-Max} = -8.369 \times 10^{-12} {L_{2T}}^4 + 1.891 \times 10^{-8} {L_{2T}}^3 - 1.848 \times 10^{-5} {L_{2T}}^2 - 2.432 \times 10^{-3} \times L_{2T} + 85.32$$
$$R^2 = 0.9971$$
$$v_{2T-85^{th}} = -8.131 \times 10^{-12} {L_{2T}}^4 + 2.220 \times 10^{-8} {L_{2T}}^3 - 2.916 \times 10^{-5} {L_{2T}}^2 + 4.214 \times 10^{-3} L_{2T} + 74.99$$
$$R^2 = 0.9962$$
$$v_{2T-Mean} = -5.557 \times 10^{-9} {L_{2T}}^3 + 1.200 \times 10^{-6} {L_{2T}}^2 - 8.866 \times 10^{-3} L_{2T} + 68.74$$
$$R^2 = 0.9962$$
$$v_{2T-Min} = -5.084 \times 10^{-14} {L_{2T}}^5 + 2.176 \times 10^{-10} {L_{2T}}^4 - 3.091 \times 10^{-7} {L_{2T}}^3 + 1.486 \times 10^{-4} {L_{2T}}^2 - 2.555 \times 10^{-2} L_{2T} + 56.09$$
$$R^2 = 0.9917$$

For Location 3P:

$$v_{3P-Max} = -6.522 \times 10^{-9} {L_{3P}}^3 + 1.167 \times 10^{-5} {L_{3P}}^2 - 2.000 \times 10^{-2} \times L_{3P} + 75.19$$
$$R^2 = 0.9906$$
$$v_{3P-85^{th}} = -9.955 \times 10^{-9} {L_{3P}}^3 + 1.750 \times 10^{-5} {L_{3P}}^2 - 2.060 \times 10^{-2} \times L_{3P} + 68.09$$
$$R^2 = 0.9932$$
$$v_{3P-Mean} = -7.536 \times 10^{-9} {L_{3P}}^3 + 1.239 \times 10^{-5} {L_{3P}}^2 - 2.078 \times 10^{-2} L_{3P} + 61.97$$
$$R^2 = 0.9965$$
$$v_{3P-Min} = -7.934 \times 10^{-9} {L_{3P}}^3 + 2.085 \times 10^{-5} {L_{3P}}^2 - 3.734 \times 10^{-2} L_{3P} + 50.59$$
$$R^2 = 0.9612$$

For Location 3T:

$$v_{3T-Max} = -3.299 \times 10^{-12} {L_{3T}}^4 + 1.362 \times 10^{-8} {L_{3T}}^3 - 2.723 \times 10^{-5} {L_{3T}}^2 + 9.745 \times 10^{-3} \times L_{3T} + 82.69$$
$$R^2 = 0.9973$$
$$v_{3T-85^{th}} = -5.794 \times 10^{-12} {L_{3T}}^4 + 1.708 \times 10^{-8} {L_{3T}}^3 - 1.931 \times 10^{-5} {L_{3T}}^2 + 6.374 \times 10^{-4} L_{3T} + 68.38$$
$$R^2 = 0.9953$$
$$v_{3T-Mean} = -1.110 \times 10^{-11} {L_{3T}}^4 + 3.937 \times 10^{-8} {L_{3T}}^3 - 4.952 \times 10^{-5} {L_{3T}}^2 + 1.475 \times 10^{-2} L_{3T} + 60.45$$
$$R^2 = 0.9977$$
$$v_{3T-Min} = -9.707 \times 10^{-9} {L_{3T}}^3 + 1.864 \times 10^{-5} {L_{3T}}^2 - 1.532 \times 10^{-2} L_{3T} + 49.75$$
$$R^2 = 0.9738$$

For Location 4P:

$$v_{4P-Max} = -2.724\times10^{-9}{L_{4P}}^3 + 8.403\times10^{-7}{L_{4P}}^2 - 1.227\times10^{-2}\times L_{4P} + 80.45$$
$$R^2 = 0.9965$$
$$v_{4P-85^{th}} = -1.144\times10^{-11}{L_{4P}}^4 + 3.867\times10^{-8}{L_{4P}}^3 - 4.975\times10^{-5}{L_{4P}}^2 + 1.493\times10^{-2}\times L_{4P} + 69.47$$
$$R^2 = 0.9976$$
$$v_{4P-Mean} = -1.379\times10^{-11}{L_{4P}}^4 + 4.858\times10^{-8}{L_{4P}}^3 - 6.145\times10^{-5}{L_{4P}}^2 + 1.814\times10^{-2}L_{4P} + 63.03$$
$$R^2 = 0.9957$$
$$v_{4P-Min} = -2.555\times10^{-14}{L_{4P}}^5 + 1.051\times10^{-10}{L_{4P}}^4 - 1.430\times10^{-7}{L_{4P}}^3 + 6.331\times10^{-5}{L_{4P}}^2 - 8.255\times10^{-3}L_{4P} + 50.23$$
$$R^2 = 0.9939$$

For Location 4T:

$$v_{4T-Max} = -2.033\times10^{-14}{L_{4T}}^5 + 9.842\times10^{-11}{L_{4T}}^4 - 1.548\times10^{-7}{L_{4T}}^3 + 1.032\times10^{-4}{L_{4T}}^2 - 3.702\times10^{-2}\times L_{4T} + 79.40$$
$$R^2 = 0.9953$$
$$v_{4T-85^{th}} = -1.844\times10^{-14}{L_{4T}}^5 + 8.800\times10^{-11}{L_{4T}}^4 - 1.487\times10^{-7}{L_{4T}}^3 + 9.779\times10^{-5}{L_{4T}}^2 - 2.589\times10^{-2}L_{4T} + 70.50$$
$$R^2 = 0.9973$$
$$v_{4T-Mean} = -7.229\times10^{-15}{L_{4T}}^5 + 2.393\times10^{-11}{L_{4T}}^4 - 2.094\times10^{-8}{L_{4T}}^3 - 7.380\times10^{-6}{L_{4T}}^2 + 5.196\times10^{-3}L_{4T} + 63.88$$
$$R^2 = 0.9955$$
$$v_{4T-Min} = -3.888\times10^{-14}{L_{4T}}^5 + 1.622\times10^{-10}{L_{4T}}^4 - 2.273\times10^{-7}{L_{4T}}^3 + 1.234\times10^{-4}{L_{4T}}^2 - 3.630\times10^{-2}L_{4T} + 56.28$$
$$R^2 = 0.9924$$

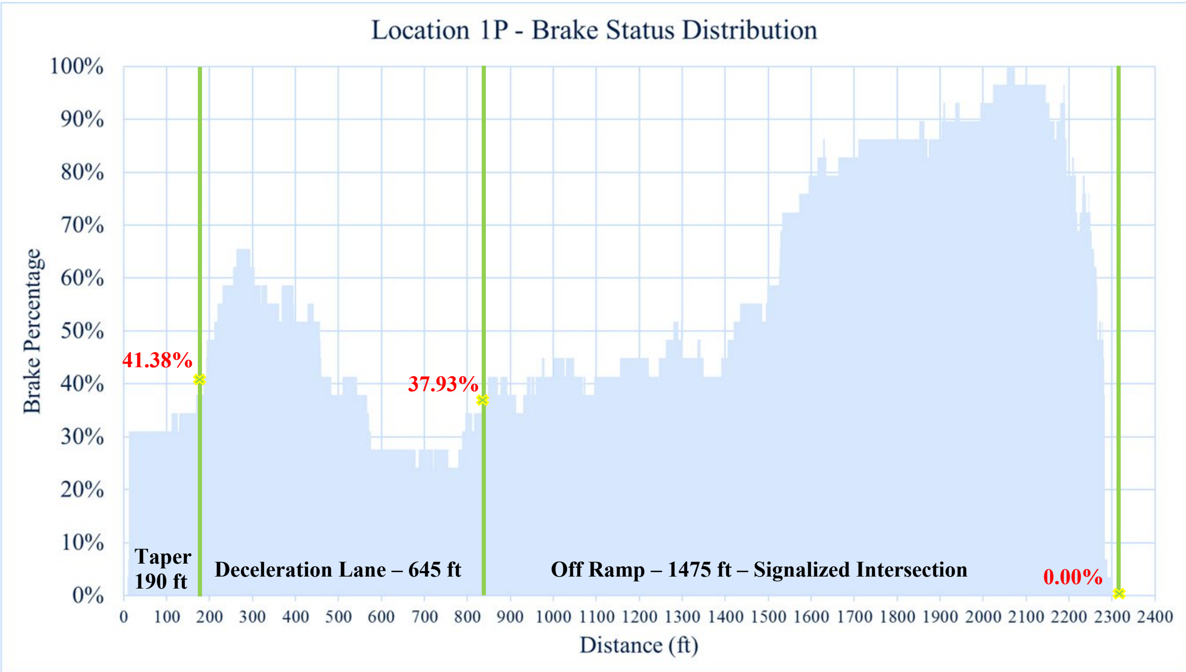
For Location 5P:

$$
\begin{aligned}
v_{5P-Max} &= -4.382 \times 10^{-9} {L_{5P}}^{3} + 1.166 \times 10^{-5} {L_{5P}}^{2} - 1.029 \times 10^{-2} \times L_{5P} + 82.18 \\
R^2 &= 0.9982 \\
v_{5P-85^{th}} &= -1.516 \times 10^{-15} {L_{5P}}^{5} + 8.509 \times 10^{-12} {L_{5P}}^{4} - 1.768 \times 10^{-8} {L_{5P}}^{3} + 1.415 \\
&\quad \times 10^{-5} {L_{5P}}^{2} - 1.732 \times 10^{-3} \times L_{5P} + 75.07 \\
R^2 &= 0.9977 \\
v_{5P-Mean} &= -1.206 \times 10^{-15} {L_{5P}}^{5} + 5.515 \times 10^{-12} {L_{5P}}^{4} - 7.754 \times 10^{-9} {L_{5P}}^{3} + 1.588 \\
&\quad \times 10^{-6} {L_{5P}}^{2} - 3.306 \times 10^{-3} L_{5P} + 69.26 \\
R^2 &= 0.9990 \\
v_{5P-Min} &= -4.204 \times 10^{-15} {L_{5P}}^{5} + 1.815 \times 10^{-11} {L_{5P}}^{4} - 1.753 \times 10^{-8} {L_{5P}}^{3} + 1.383 \\
&\quad \times 10^{-5} {L_{5P}}^{2} + 1.804 \times 10^{-2} L_{5P} + 50.37 \\
R^2 &= 0.9841
\end{aligned}
$$

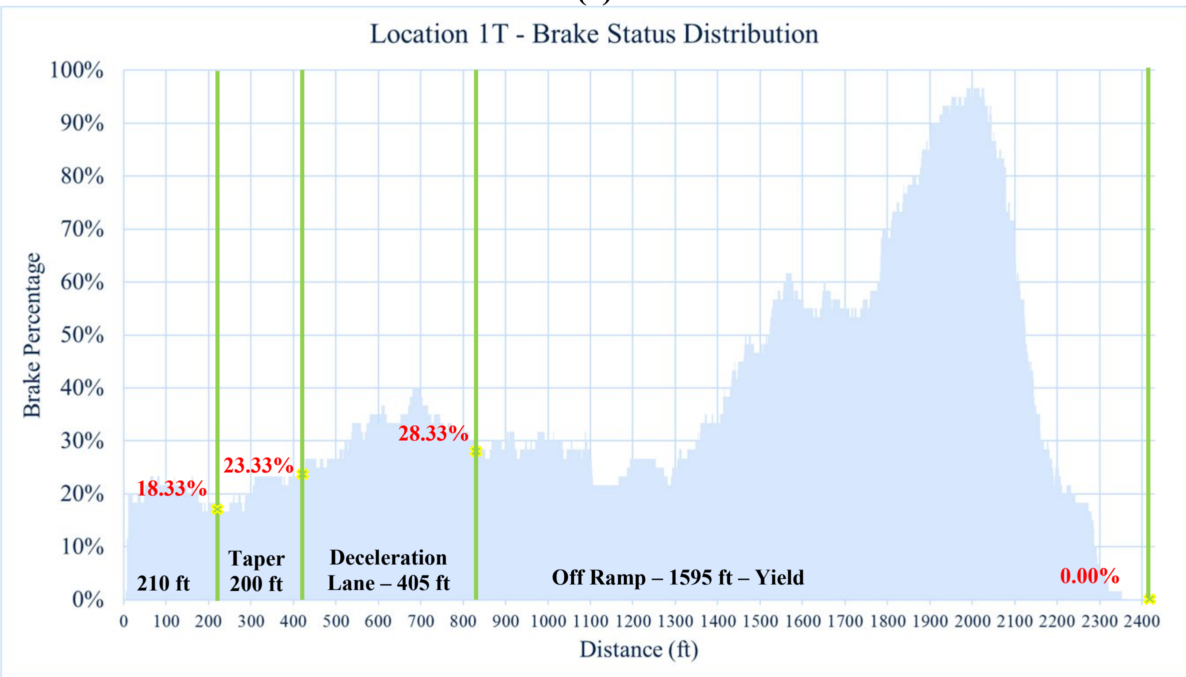
For Location 5T:

$$
\begin{aligned}
v_{5T-Max} &= -8.728 \times 10^{-10} {L_{5T}}^{3} + 3.841 \times 10^{-7} {L_{5T}}^{2} - 6.672 \times 10^{-3} \times L_{5T} + 82.49 \\
R^2 &= 0.9949 \\
v_{5T-85^{th}} &= -1.941 \times 10^{-12} {L_{5T}}^{4} + 1.048 \times 10^{-8} {L_{5T}}^{3} - 2.105 \times 10^{-5} {L_{5T}}^{2} + 8.325 \\
&\quad \times 10^{-3} L_{5T} + 75.12 \\
R^2 &= 0.9936 \\
v_{5T-Mean} &= -1.221 \times 10^{-15} {L_{5T}}^{5} + 8.262 \times 10^{-12} {L_{5T}}^{4} - 2.044 \times 10^{-8} {L_{5T}}^{3} + 1.892 \\
&\quad \times 10^{-5} {L_{5T}}^{2} - 1.151 \times 10^{-2} L_{5T} + 71.02 \\
R^2 &= 0.9895 \\
v_{5T-Min} &= -1.348 \times 10^{-11} {L_{5T}}^{4} + 5.531 \times 10^{-8} {L_{5T}}^{3} - 8.135 \times 10^{-5} {L_{5T}}^{2} + 4.044 \\
&\quad \times 10^{-2} L_{5T} + 52.98 \\
R^2 &= 0.9727
\end{aligned}
$$

APPENDIX D: Break Status Distribution at Freeway Diverge Areas

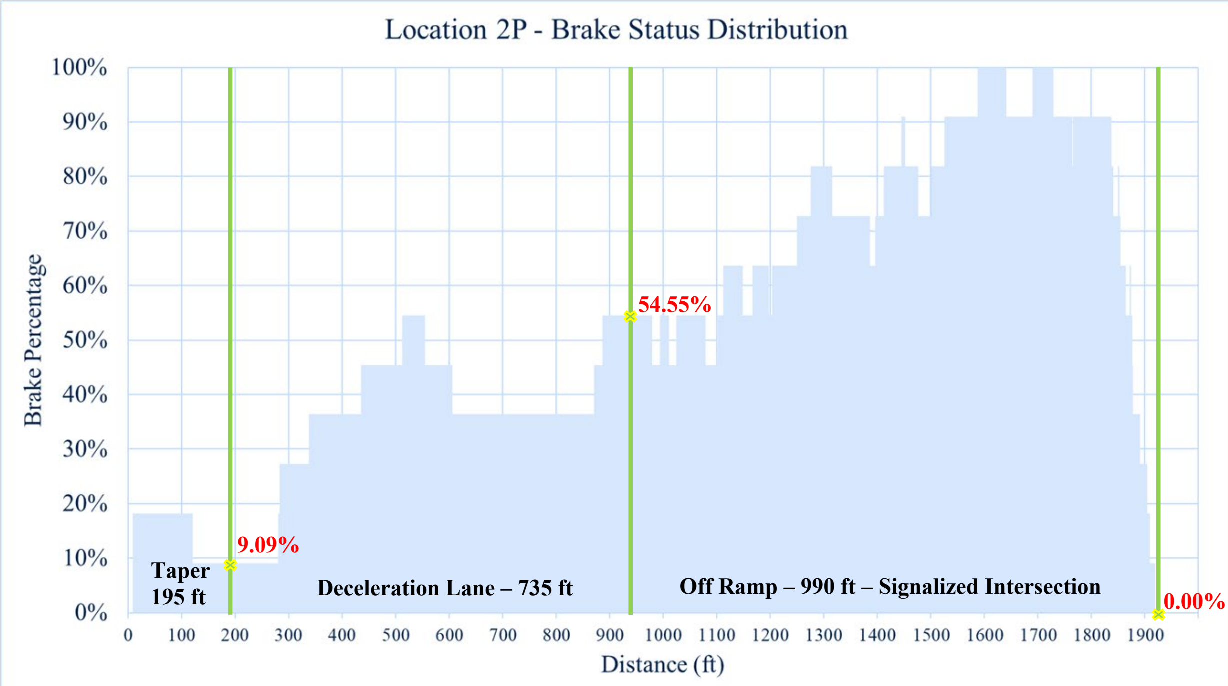


(*a*)

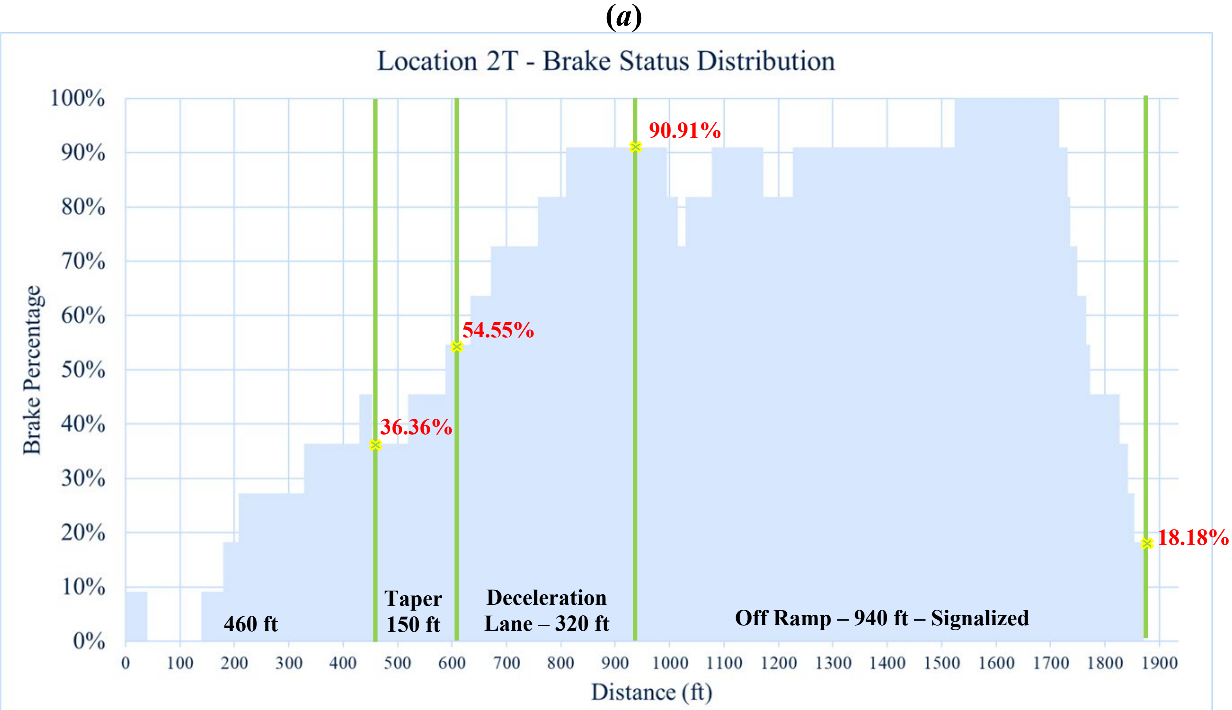


(*b*)

Figure D-1 Brake status distribution: (*a*) Location 1P; and (*b*) Location 1T

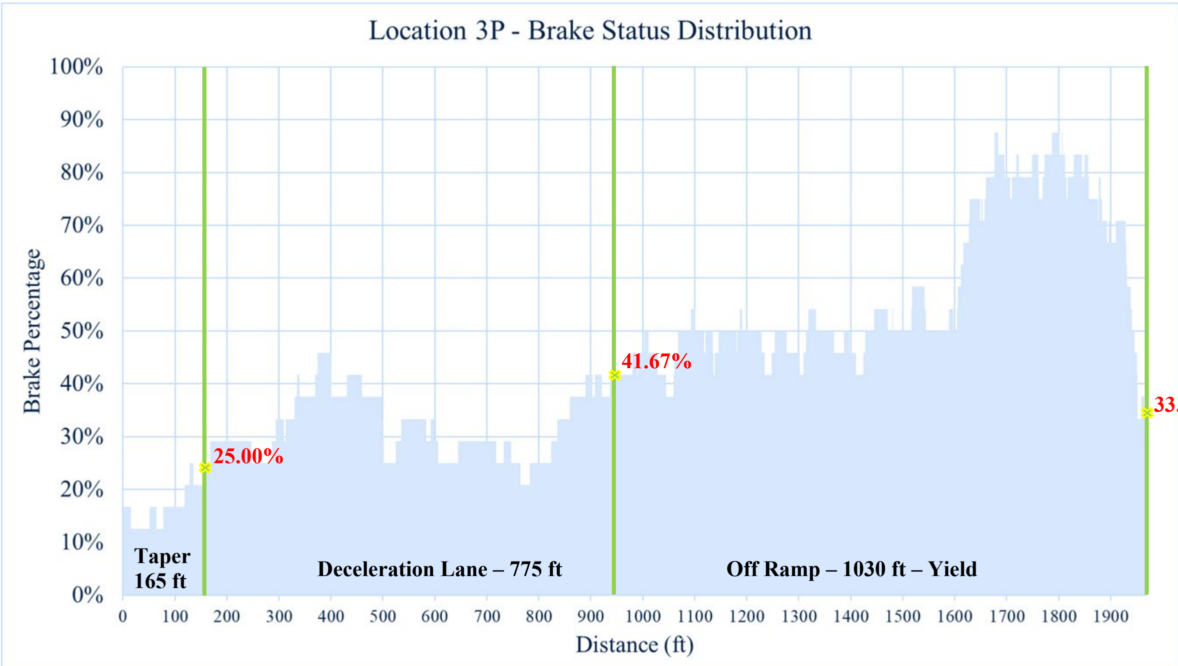


(*a*)

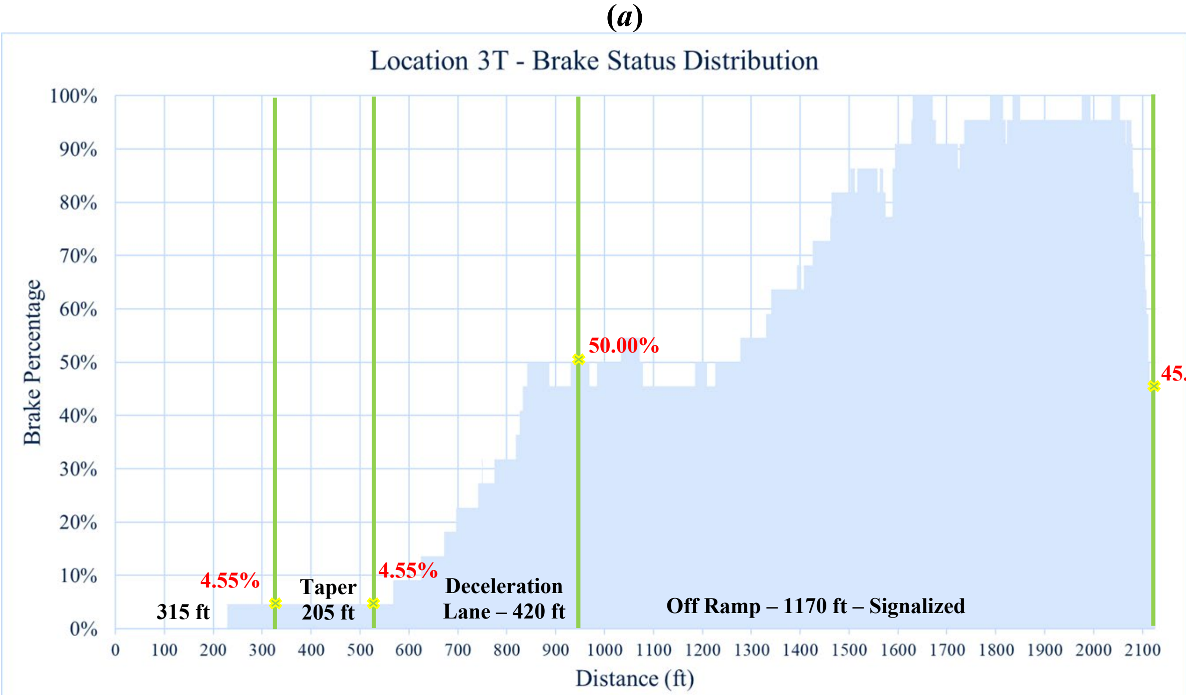


(*b*)

Figure D-2 Brake status distribution: (*a*) Location 2P; and (*b*) Location 2T

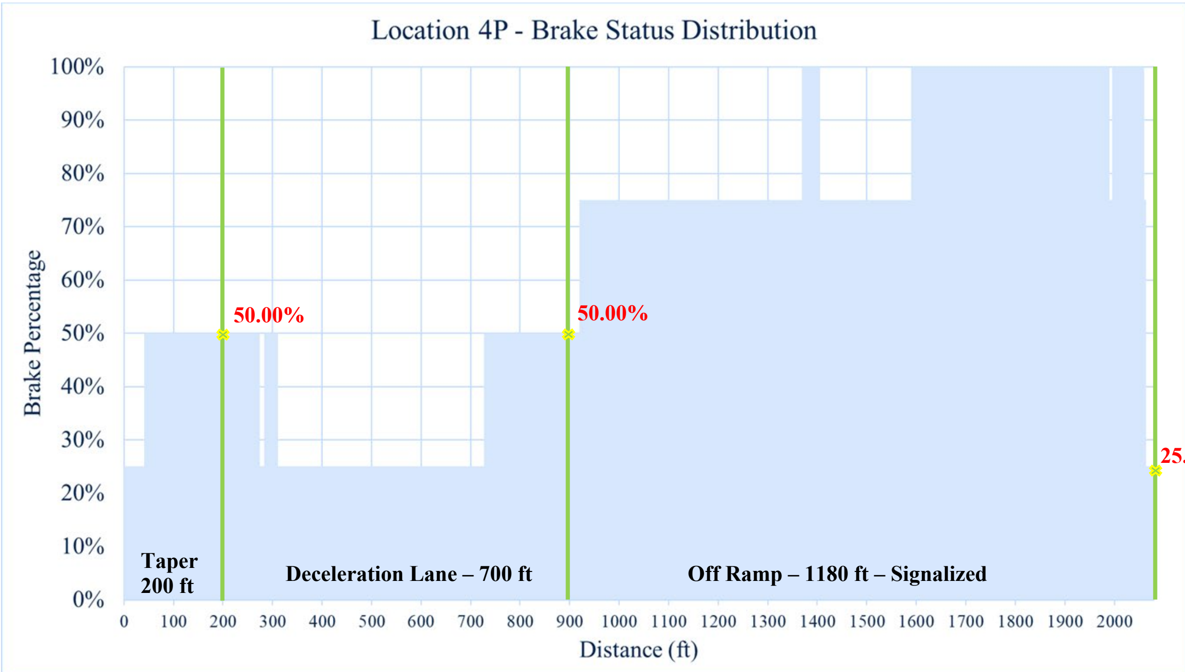


(a)

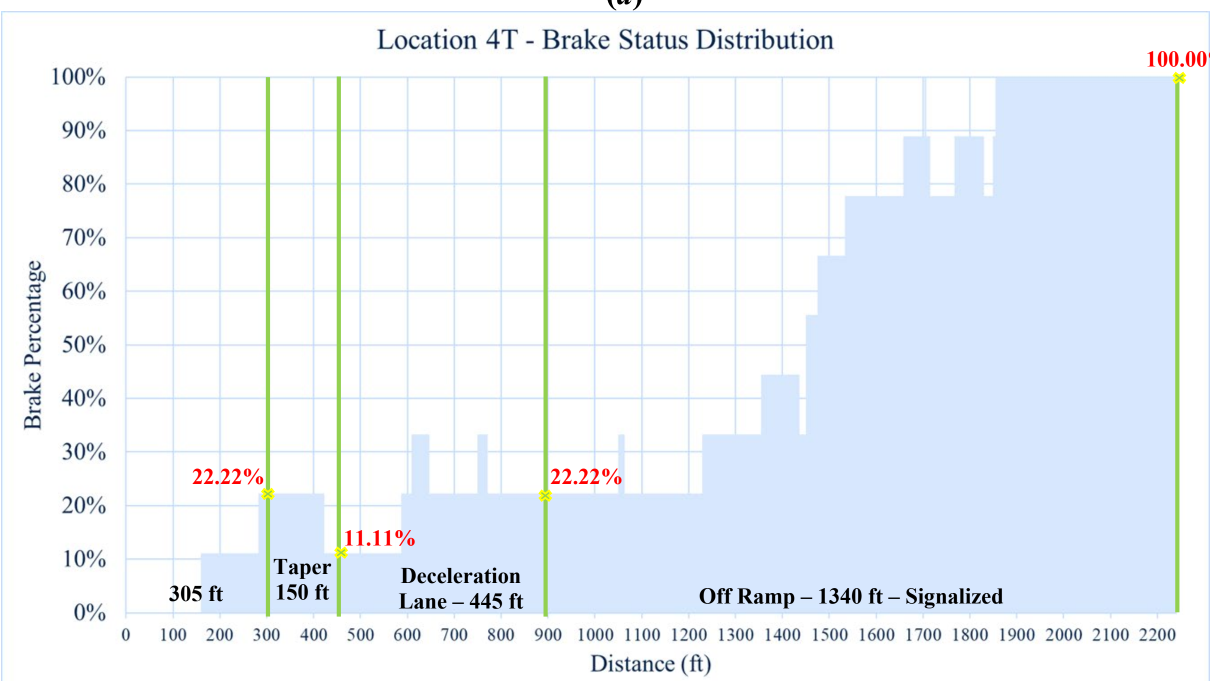


(b)

Figure D-3 Brake status distribution: (*a*) Location 3P; and (*b*) Location 3T

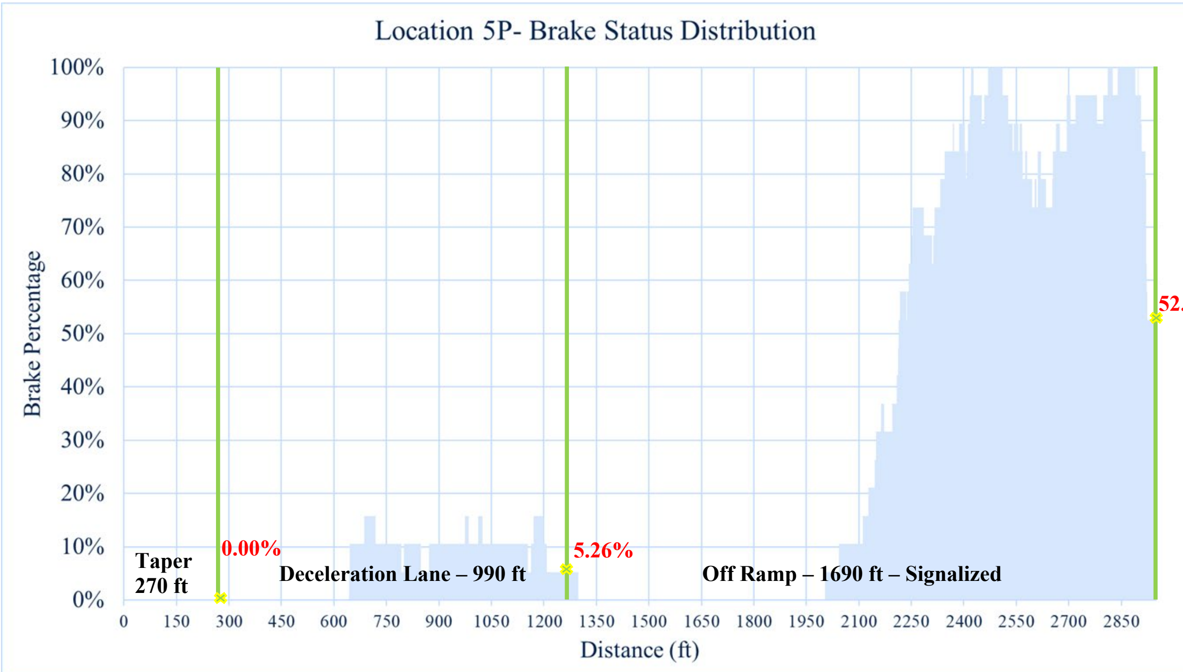


(*a*)

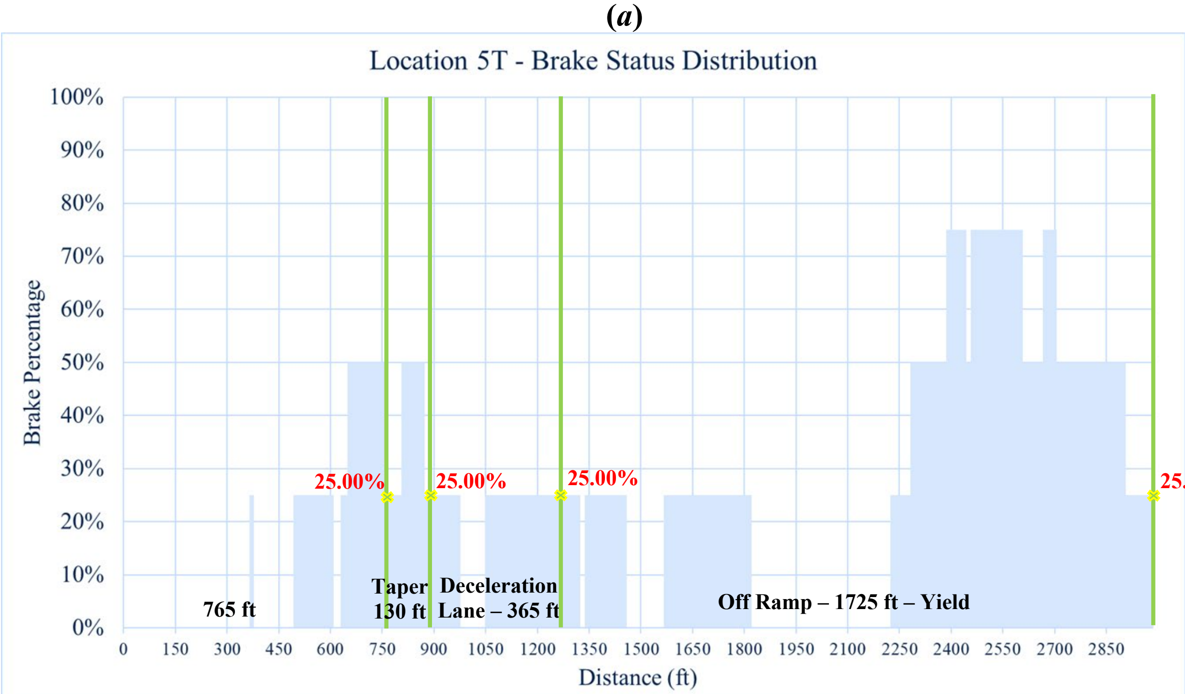


(*b*)

Figure D-4 Brake status distribution: (*a*) Location 4P; and (*b*) Location 4T



(*a*)



(*b*)

Figure D-5 Brake status distribution: (*a*) Location 5P; and (*b*) Location 5T

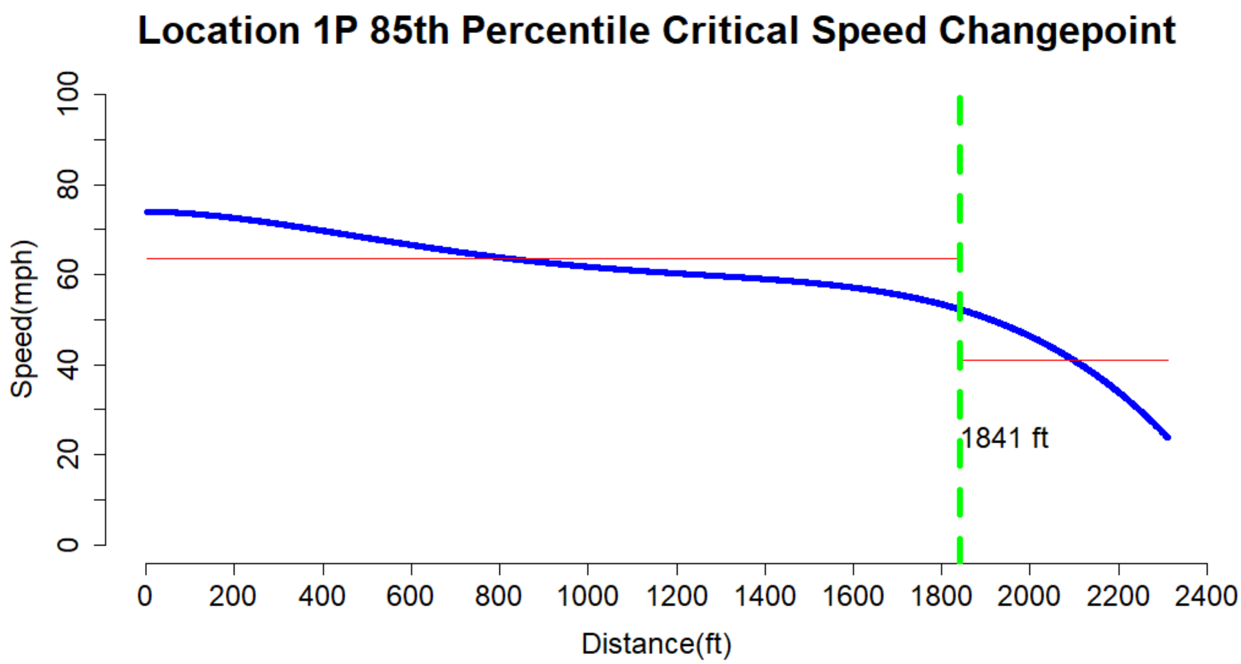
Table D-1 Brake pedal usage at critical points of freeway diverge areas

Site	Brake Usage (%)				
		Taper Start	Deceleration Lane Start	Deceleration Lane End	Off-Ramp End
Location 1P	-	0.00	41.38	37.93	0.00
	210 ft				
Location 1T	0.00	18.33	23.33	28.33	0.00
Location 2P	-	0.00	9.09	54.55	0.00
	460 ft				
Location 2T	9.09	36.36	54.55	90.91	18.18
Location 3P	-	16.67	25.00	41.67	33.33
	315 ft				
Location 3T	0.00	4.55	4.55	50.00	25.00
Location 4P	-	25.00	50.00	50.00	34.05
	305 ft				
Location 4T	0.00	22.22	11.11	22.22	100.00
Location 5P	-	0.00	0.00	5.26	52.63
	765 ft				
Location 5T	0.00	25.00	25.00	25.00	25.00

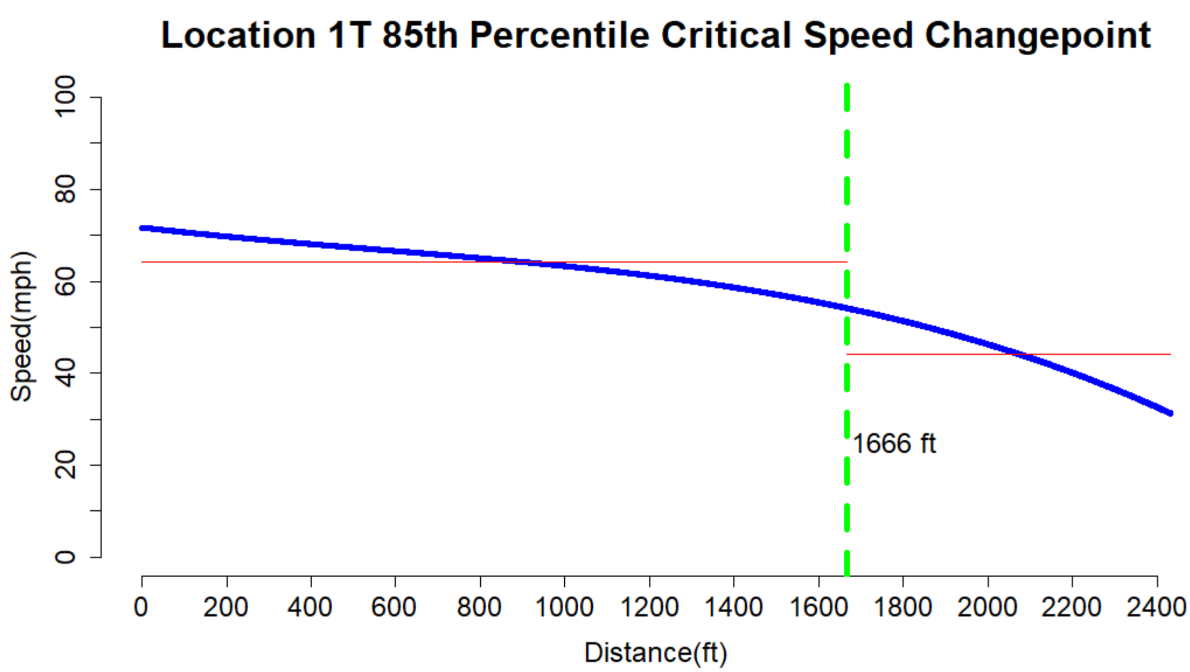
Table D-2 Average brake pedal usage on different sections of freeway diverge areas

Site	Average Brake Usage (%)			
		Taper	Deceleration Lane	Off-Ramp
Location 1P	-	31.11	41.96	64.51
	210 ft			
Location 1T	19.64	20.82	31.70	42.93
Location 2P	-	13.94	36.36	72.70
	460 ft			
Location 2T	20.75	43.27	78.66	80.33
Location 3P	-	17.02	31.94	57.25
	315 ft			
Location 3T	1.26	4.55	25.78	71.10
Location 4P	-	45.00	34.79	84.48
	305 ft			
Location 4T	6.16	19.85	20.45	64.13
Location 5P	-	0.00	6.47	39.40
	765 ft			
Location 5T	12.09	37.88	21.11	30.93
Parallel-Design	-	21.42	30.30	63.67
Tapered-Design	11.98	25.23	32.51	57.69

APPENDIX E: Critical 85th Percentile Speed Changepoint at Freeway Diverge Areas

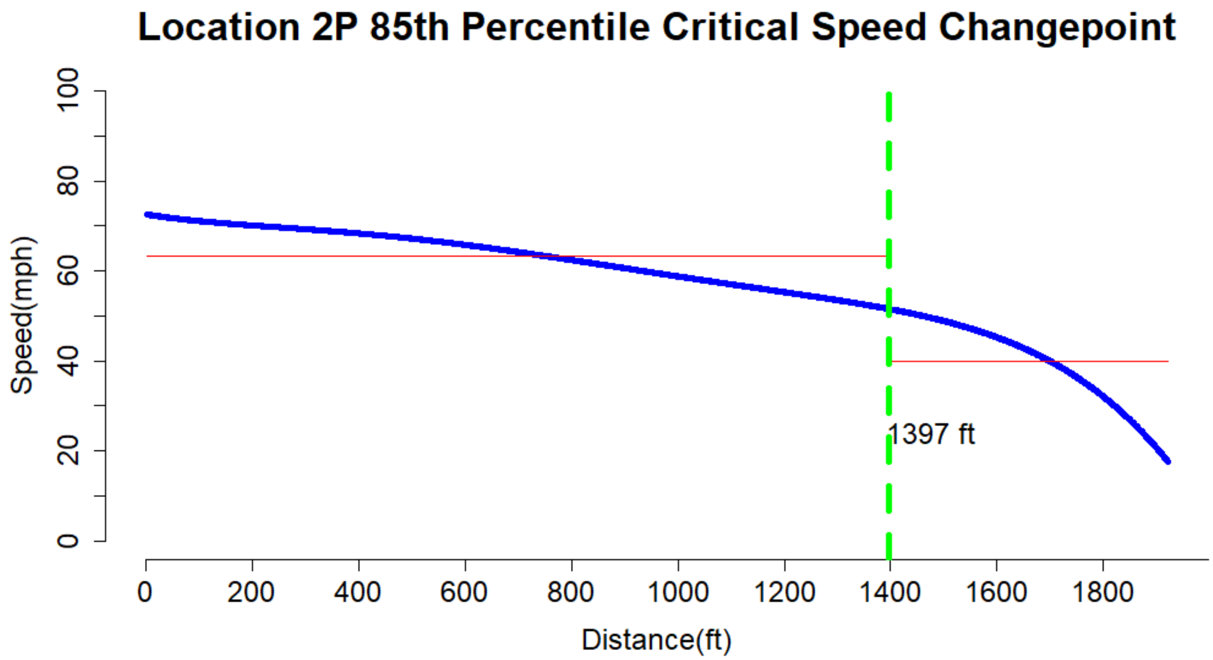


(*a*)

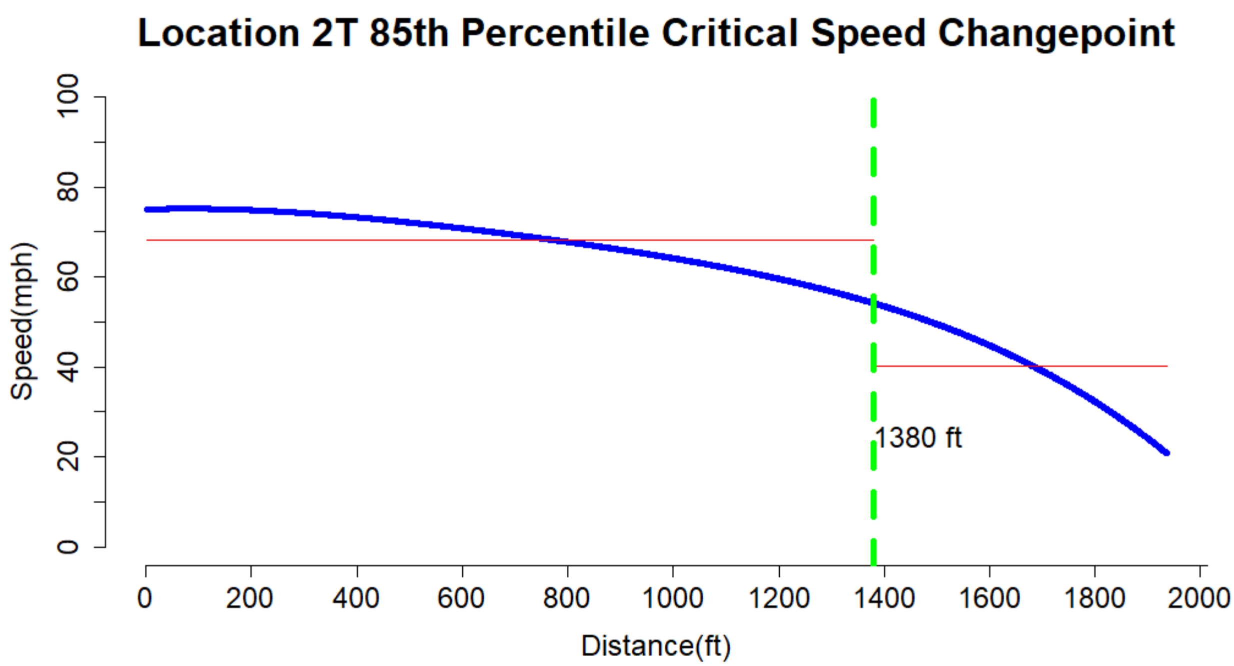


(*b*)

Figure E-1 Critical speed changepoint: (*a*) Location 1P; and (*b*) Location 1T.

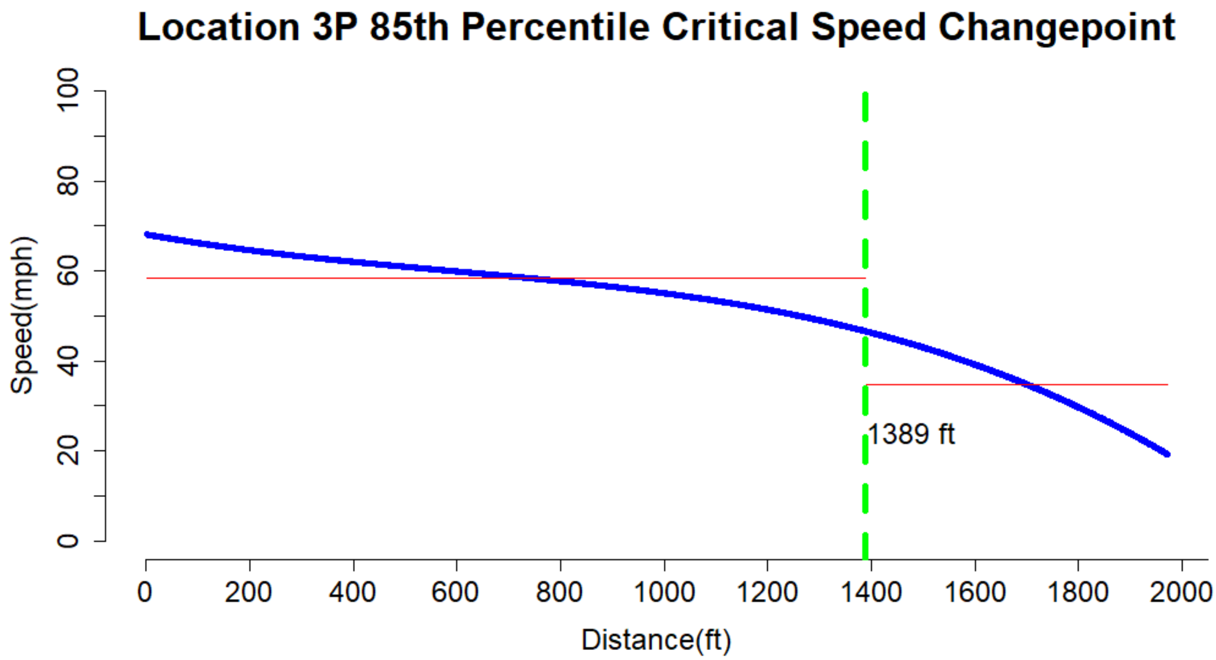


(*a*)

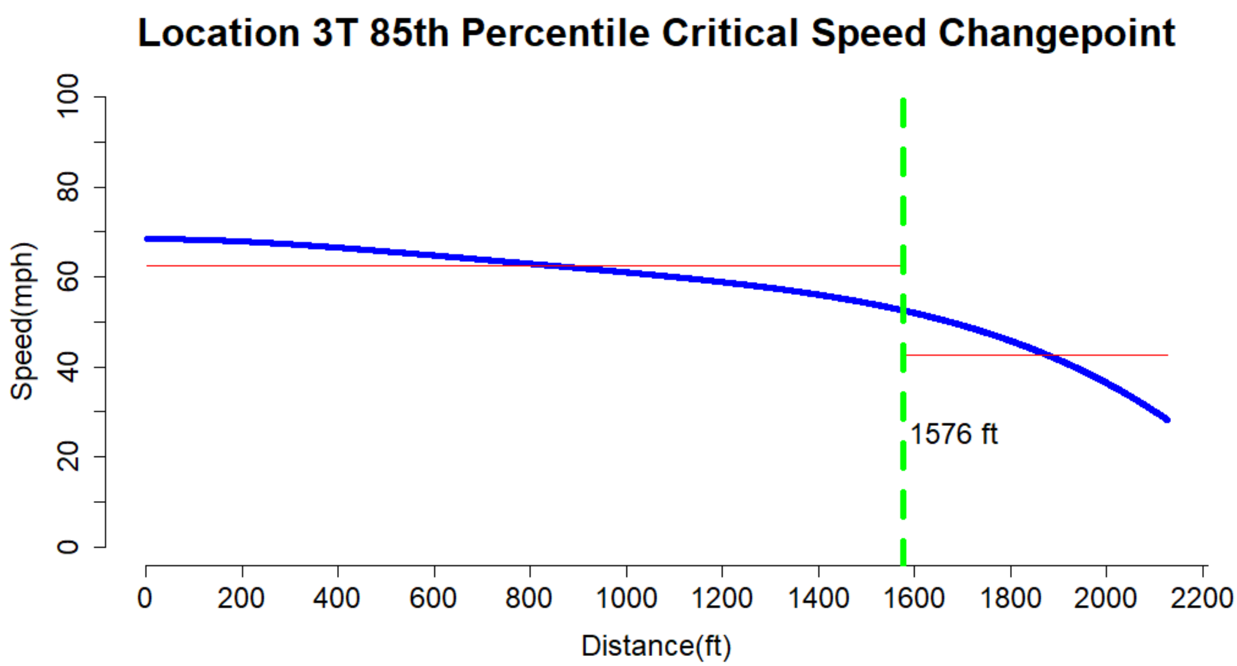


(*b*)

Figure E-2 Critical speed changepoint: (*a*) Location 2P; and (*b*) Location 2T

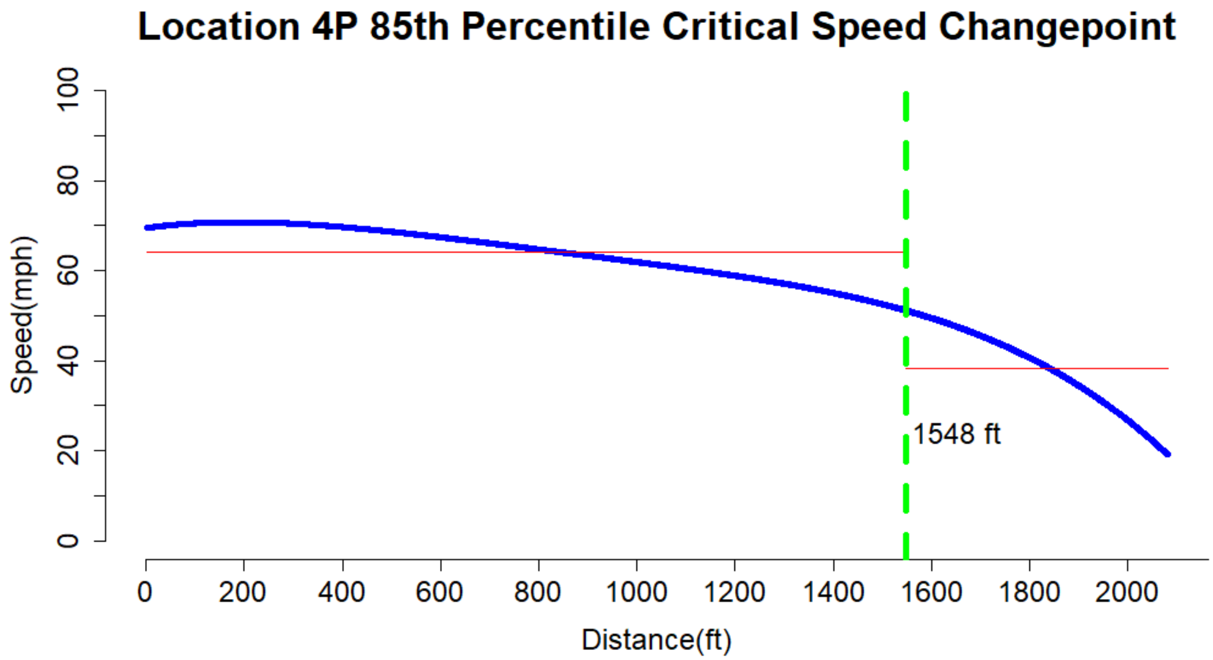


(a)

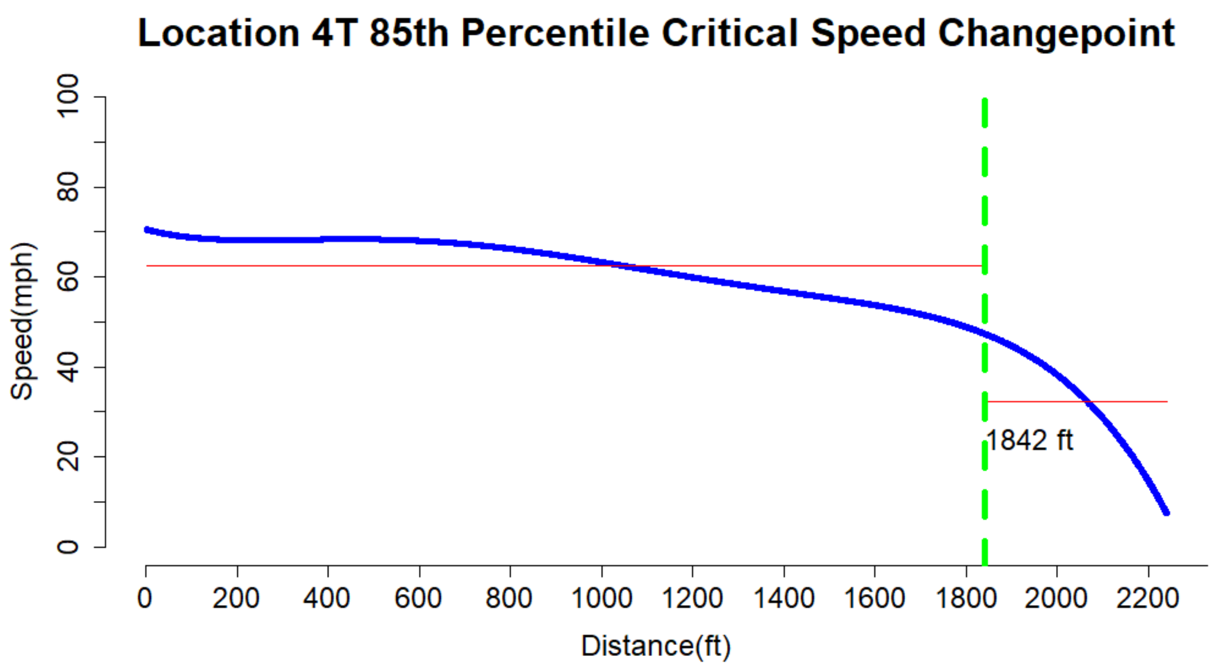


(b)

Figure E-3 Critical speed changepoint: (*a*) Location 3P; and (*b*) Location 3T

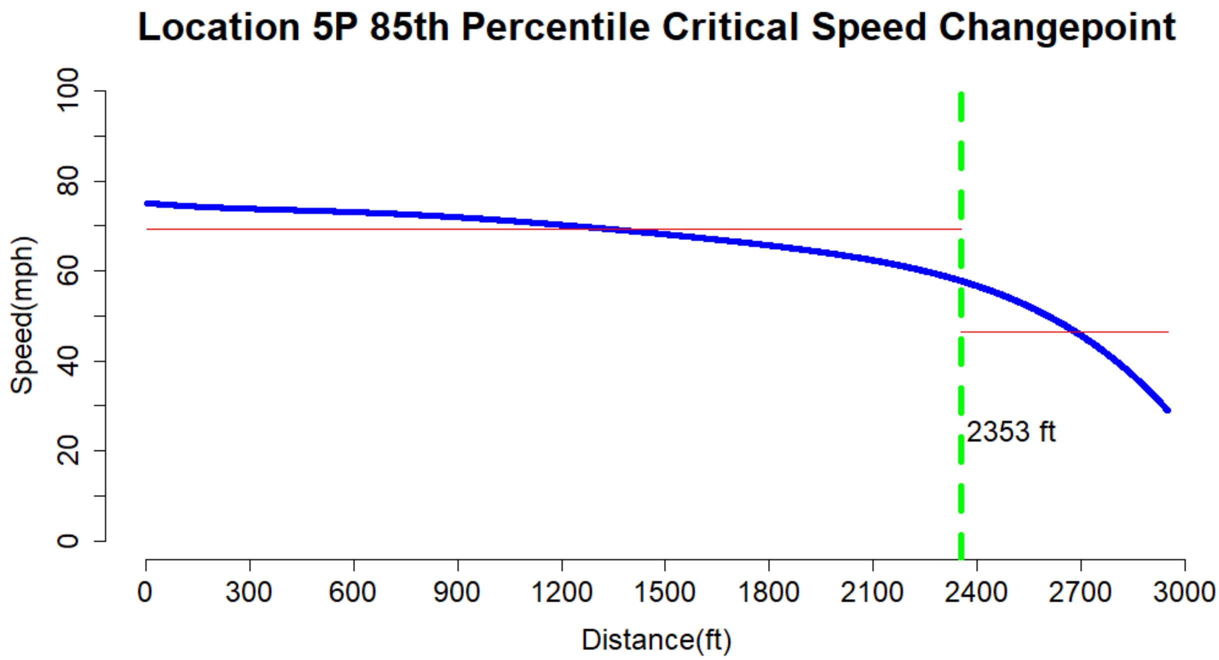


(*a*)

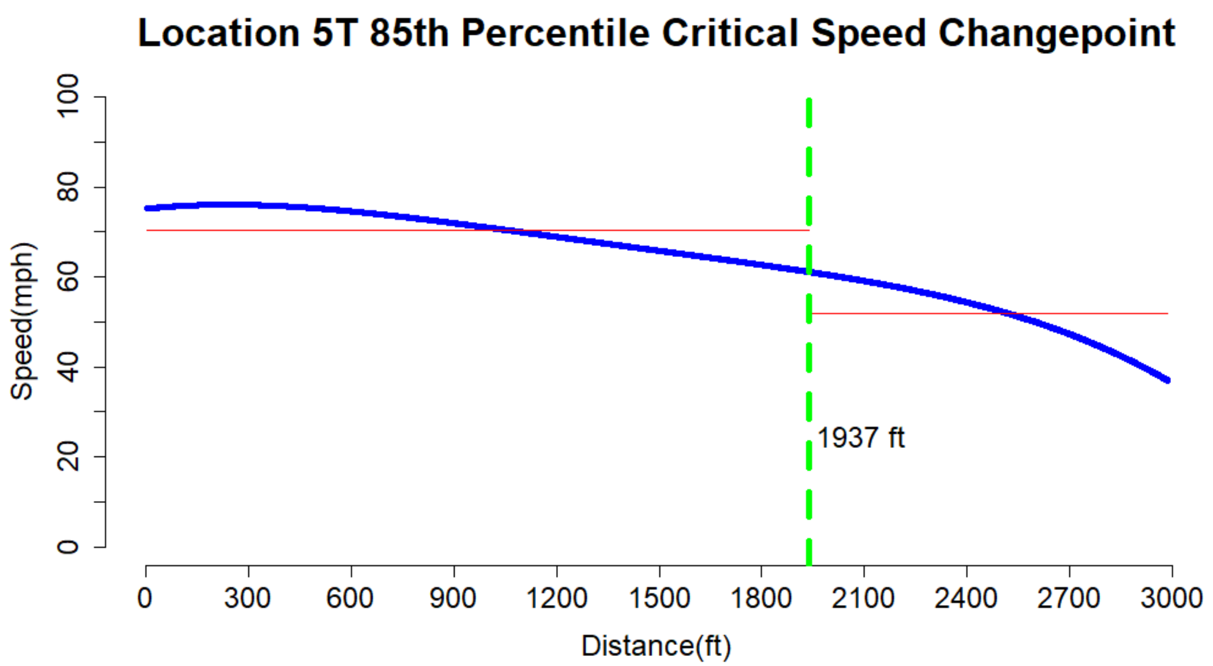


(*b*)

Figure E-4 Critical speed changepoint: (*a*) Location 4P; and (*b*) Location 4T



(a)



(b)

Figure E-5 Critical speed changepoint: (*a*) Location 5P; and (*b*) Location 5T

Table E-1 Critical speed change point summary

Site	Change Point (ft)			
	Distance from Taper		Distance from Terminal	
	85th	Mean	85th	Mean
Location 1P	1,841	1,889	469	421
Location 1T	1,666	1,685	764	745
Location 2P	1,397	1,423	523	497
Location 2T	1,380	1,309	490	561
Location 3P	1,389	1,305	581	665
Location 3T	1,576	1,615	534	495
Location 4P	1,548	1,541	532	539
Location 4T	1,842	1,817	398	423
Location 5P	2,353	2,364	597	586
Location 5T	1,937	1,948	1,048	1,037
Parallel-Design	1,421	1,420	450	451
Tapered-Design	1,400	1,396	539	544