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16. Abstract

This report presents an evaluation of the use of converging chevron pavement markings in an attempt to reduce speeds on freeway-to-freeway connector ramps. In this project, a converging chevron marking was designed and implemented on a freeway-to-freeway connector ramp in El Paso, Texas. Speed measurements were conducted at the project site in three study periods: before the implementation of chevron markings, early-after the implementation, and late after the implementation of chevron markings. Speed measurements were taken upstream of the curve, at the start of the curve, and in the middle of the curve of the connector ramp. A detailed before-after analysis of the speed data was conducted by vehicle classification, light conditions, and location along the curve. From the analysis results, it was found that chevron markings appeared to reduce speeds where the markings were in place, though the reduction in the average speeds varied based on vehicle class and curve location. Maximum reduction was observed at the upstream location of the curve with about a 4 mile per hour (mph) reduction observed for heavy trucks.

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EVALUATION OF CHEVRON MARKINGS ON FREEWAY-TO-FREEWAY CONNECTOR RAMPS IN TEXAS

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DISCLAIMER

This research was performed in cooperation with the Texas Department of Transportation (TxDOT) and the Federal Highway Administration (FHWA). The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official view or policies of the FHWA or TxDOT. This report does not constitute a standard, specification, or regulation. This report is not intended for construction, bidding, or permit purposes. The engineer in charge of the project was Anthony P. Voigt, P.E., #84845.

The United States Government and the State of Texas do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

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- Mr. Mohammad Rafipour, P.E., Texas Department of Transportation, Houston District, Project Advisor (Retired); and
- Mr. Wade Odell, P.E., Texas Department of Transportation, Research and Technology Implementation Office, Austin, Research Engineer.

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CHAPTER 1. INTRODUCTION

Freeway interchanges can be problematic for drivers who choose to drive at higher speeds on freeway connector ramps, primarily due to the restrictive geometry of many freeway-to-freeway connectors. Connector ramps may be particularly hazardous for large trucks due to a higher potential for rollover, especially for those truck drivers that tend to travel at higher speeds. Previous studies have recognized that crashes, particularly truck crashes, tend to cluster at freeway interchange off-ramps and direct connector ramps. Many of these truck crashes are single-vehicle crashes where truck and driver performance, driver expectations, and roadway geometry interact, sometimes quickly and forcefully with negative results.

This report documents an evaluation of the effectiveness of converging chevron pavement markings in reducing speed at freeway-to-freeway connector ramps. There have been very few studies, especially in the United States, that have evaluated the converging chevron marking scheme as a tool for speed reduction on freeway connector ramps. This is the first project in the state of Texas to evaluate converging chevron markings on freeway ramps. This report provides the results of this particular evaluation, as well as guidelines for the potential use of chevron markings on freeway-to-freeway connector ramps.

BACKGROUND

This research builds on a previous project that quantified the differences in vehicle operations on freeway-to-freeway connectors *(1)*. In that project, researchers found that 95 to 99 percent of non-truck driver motorists (i.e., drivers in passenger cars, light trucks, and sport-utility vehicles, etc.) typically exceeded the posted advisory speed limits on freeway-to-freeway connector curves by more than 10 miles per hour *(1)*. That same research indicated a 5 to 10 mile per hour (mph) higher difference between a passenger car driver's maximum comfortable speed on a freeway-to-freeway connector ramp compared to that of drivers of larger vehicles *(1)*. In response to the observed higher speeds (and differential speeds between trucks and cars) that may lead to unsafe operating conditions on freeway-to-freeway connector ramps, a project was conducted to address the differential advisory speed signing *(2)*. The project evaluated dual advisory signs, with differing advisory speeds for passenger vehicles and trucks, and provided guidelines to implement those signs on freeway-to-freeway connectors *(2)*.

This research project is more specifically founded on experiences in the Houston, Texas, urban area where speed-related crashes frequently occur on some freeway-to-freeway connector ramps. Freeway connector ramps usually have lower advisory speeds than the operating speeds upstream or downstream of the freeway-to-freeway connector ramps. Some motorists may fail to judge safe maneuver speeds on freeway-to-freeway connectors, which can result in crashes. More specifically, higher-speed trucks may be more prone to crashes on freeway-to-freeway connector ramps, many of which may result in truck rollovers. Truck rollovers are typically high impact and high visibility incidents that can bring traffic on freeway facilities to a halt during any time of the day. These incidents tend to require several hours for cleanup and removal, often result in injuries or fatalities, and traffic impacts from the incident can result in extraordinary traffic delays on affected freeway systems. Although advisory speed signs are customarily installed to inform drivers of appropriate speeds on freeway-to-freeway connectors, some additional innovative signing and/or pavement marking treatments may be beneficial to supplement traditional signing and pavement markings to discourage motorists from speeding.

OVERVIEW

According to the National Highway Traffic Safety Administration (NHTSA), the overall economic impact due to crashes has been estimated at around $230 billion annually in 2001. While there may be many contributing factors for fatal crashes, one of the most significant is selection of unsafe speed. Approximately 30 percent of fatal crashes in the United States in 2005 were attributed to speeding. The economic impact due to speed-related crashes in 2005 was estimated to be about $40 billion *(3)*.

In 2005, in the state of Texas alone, 1426 of 3504 traffic fatalities were related to speeding. The 2005 statistics indicate that among the total fatal crashes, the percentage of speed-related crashes in Texas was 40 percent, which is 10 percent higher than the national average. In addition, the 2005 statistics reveal that 15 percent of the fatal crashes in Texas were reported to have involved large trucks. Total truck-related fatalities were estimated to be around 50 percent of the total fatalities in Texas during 2005 *(3)*.

The state of Texas serves as an economic gateway between the east and west coasts of the United States (U.S.), as well as to Canada, Mexico, and Central America. As a crossroads of sorts, the state has experienced an increasing amount of truck traffic on its highways. The Texas Department of Public Safety states on its website that as a result of the North American Free Trade Agreement (NAFTA), commercial truck traffic has increased dramatically in Texas. Commercial motor vehicle miles traveled in Texas increased 47 percent from 1993 to 1999, and according to TxDOT, approximately 16 percent of all trucks traveling in Texas are NAFTA-related. According to figures from the U.S. Customs Service, about 69 percent of the commercial truck traffic from Mexico comes through Texas. This ever-increasing number of trucks on our state highways affects traffic operations in various ways, including increasing the potential for truck conflicts and crashes.

As the trucking industry continues to grow and employs newer and less-experienced drivers, the number of truck drivers with limited knowledge of Texas freeway facilities will increase. These less-experienced truck drivers must rely on the signing and pavement marking techniques that the Texas Department of Transportation and other state Departments of Transportation (DOTs) use to convey the appropriate advisory speeds to motorists while negotiating freeway-to-freeway connector ramps.

Previous data collected in the Houston region on freeway-to-freeway connector ramps indicated that all types of vehicles are exceeding the posted limits by varying amounts ranging from 5 mph to more than 15 mph *(1)*. Speeds in excess of the posted advisory speeds may be acceptable for driver comfort and vehicle physics during a majority of the time, but there are situations where inexperienced or inattentive drivers (especially drivers of large truck-trailer combinations with high centers of gravity) may exceed the posted advisory speed limit on some connectors, sometimes resulting in rollover crashes. Transporting high loads can be especially challenging for less experienced truck drivers, who may not fully understand the physics of the trailer and its cargo. This lack of driver experience may be compounded during inclement weather when tire/pavement friction supply may be reduced or during periods of high-volume traffic when vehicle headways may be less than desirable.

The national authoritative reference for the geometric design of horizontal curves is the American Association of State Highway and Transportation Officials' (AASHTO) Green Book *A Policy on the Geometric Design of Highways and Streets* *(4)*. The objective of the AASHTO policy on horizontal curve design is to select a curve radius and superelevation rate such that the unbalanced lateral acceleration remains within "comfortable" limits. While skidding is of concern for passenger vehicles on freeway-to-freeway connector ramps with lower radii, rollover is a major concern for trucks and other heavy vehicles. In the critical review on design criteria adopted by AASHTO, questions were raised on the criteria adopted for design of curves with respect to rollover thresholds. It also suggests that curves may not be safe for high speeds when both skidding as well as rollovers are to be avoided *(2)*. Hence, it is deemed essential to have some kind of traffic control measure that advises drivers of the necessary speed reduction required to safely traverse many freeway-to-freeway connector ramps.

In response to the above factors and findings from previous research, there was an identified need to determine if there were any passive traffic control devices that could contribute to a noted speed reduction on freeway-to-freeway connectors. As a part of this project, the researchers conducted a survey to understand the use of pavement markings and signing practices adopted by TxDOT and other State Department of Transportation agencies throughout the United States. Detailed information from the survey analysis was reported in the first report of this project *(2)*.

According to the previous survey results of TxDOT District Transportation Operations Engineers, 4 out of 24 responses indicated the existing use of non-standard pavement markings to warn drivers of speeding on freeway-to-freeway connector curves *(2)*. When responses from other state DOTs were analyzed, three out of 22 state traffic engineers indicated that their agencies used some kind of pavement markings to warn truck drivers or other heavy vehicles about speeds on freeway-to-freeway connector curves *(2)*.

Although there have been many lower-cost pavement marking treatments (such as transverse bars or chevron markings) conceived of that could potentially reduce speeds on roadways, there have been very few evaluations to justify the effectiveness of these treatments. Effectiveness of pavement markings, especially converging chevron markings, has been evaluated in the United States at only one known location. A study by researchers at Marquette University showed that there was a significant reduction in speed and crashes due to the installation of converging chevron markings on one freeway-to-freeway connector ramp in Milwaukee, Wisconsin. While the results of the Milwaukee study were promising, there was a perceived need to evaluate the converging chevron pavement marking treatments at other locations, with detailed evaluation, before any definitive conclusions could be drawn on the potential use of converging chevron pavement marking treatments for speed reduction on freeway-to-freeway connectors.

The researchers attempted to determine whether chevron pavement markings would be a viable treatment to influence a reduction in speeds on freeway-to-freeway connector curves for all vehicle classes. Specifically, this project evaluated a converging chevron pavement marking implementation on the US 54 westbound to IH 10 westbound freeway-to-freeway connector ramp in El Paso, Texas. In completing the before-after analysis of the converging chevron pavement marking concept, the following hypotheses were evaluated:

- Is there a significant reduction in overall mean speed on the approach to the freeway-to-freeway curve as well as at different points on the curve?
- Is there a significant reduction in mean speeds for different vehicle classes?
- Do daylight and non-daylight conditions have an impact on effectiveness the of chevron pavement markings in reducing speeds?

CHAPTER 2. METHODOLOGY

This chapter briefly summarizes each of the work tasks of this research project. The project sought to provide a literature review of previous studies of pavement marking treatments aimed at speed reduction, particularly those studies that focused more on converging chevron pavement markings as an effective tool for speed control. The core of this project involved developing and implementing a converging chevron pavement marking treatment on a freeway-to-freeway connector ramp and analyzing the effectiveness of chevron pavement markings as a tool for reducing speeds at desired locations. The general methodologies used in the evaluation of converging chevron pavement markings are highlighted in this chapter.

TASK 1. LITERATURE REVIEW

The researchers conducted a comprehensive literature search to identify publications on previous studies and existing practices on the use of pavement marking-treatments intended to reduce speeds, particularly on freeway-to-freeway connector ramps. This search used all available bibliographic resources including the internet and various catalogs and databases such as Texas A&M University's Sterling C. Evans Library local library database, Online Computer Library Center (OCLC) database, National Technical Information System (NTIS), and Transportation Research Information Service (TRIS).

The researchers selected key words and word combinations to conduct a systematic search of these databases. Key words and key word combinations used in the search included: freeway ramp, trucks, speed reduction, ball-bank indicator, chevron markings, transverse markings, and pavement markings, among others. After identifying potential literature sources, researchers acquired abstracts and reviewed those abstracts for applicability to the project. Those documents identified as being of interest were obtained for incorporation into the literature review. Chapter 3 of this report summarizes this effort.

TASK 2. PROJECT SITE SELECTION

The intent of this research was to identify an appropriate test site for deployment of the converging chevron pavement marking treatment. Earlier in this research project, transportation operations engineers for each of the 25 TxDOT districts were contacted to gather information on their experiences with problematic freeway-to-freeway connector curves. These problematic sites were considered for the converging chevron pavement marking treatment installation, with the selection criteria based on the experience of the local engineers. After a review of several potential study sites, one freeway-to-freeway connector in El Paso, Texas, was selected for a prototype converging chevron pavement marking treatment installation and testing.

Chapter 4 of this report presents the details of the converging chevron pavement marking treatment design and the characteristics of the study site. Detailed information on survey results was presented in the previous report of this project *(2)*, and therefore, detailed discussion related to the surveys is omitted from this report. However, Appendix A and Appendix B replicate the survey results pertaining to pavement markings.

TASK 3. DEVELOP DATA COLLECTION PLAN

Researchers sought to obtain volume, speed, and vehicle classification measurements at multiple locations on the freeway-to-freeway connector ramp selected for this project. These data were collected in order to measure any operational impacts of the chevron pavement marking treatment. Automatic data collection devices were deployed to measure vehicle speeds and classifications at four (three in the case of data collected before the installation of chevrons) locations on the study curve:

- *far upstream* of the connector ramp (not considered in before period),
- *upstream* of connector ramp,
- at the point of curvature of the freeway-to-freeway connector ramp (denoted as "s*tart of curve*"), and
- at the midpoint of the freeway-to-freeway connector ramp curve (denoted as "*middle of curve*").

Data were collected in three discrete periods, termed here as "before," "early-after," and "late-after." Before refers to the data collection period before the installation of chevrons, early-after refers to data collected soon after the installation of chevrons, and late-after refers to the period in which data was collected a few months after the chevron installation (about three months after). Data collection periods were designed to be able to compare the effectiveness of the chevron pavement marking treatment on motorist speed, early-after their installation and within several months of their installation, to determine if any impacts seen early-after the installation changed. Chapter 5 of this report provides more details on the data collection efforts involved in this project.

TASK 4. ANALYSIS OF CHEVRON PAVEMENT MARKING EFFECTIVENESS

Once speed data were collected, the researchers completed a thorough quality control and analysis of the data for each site. The analysis steps included the following:

- segmentation of free flowing vehicles;
- categorization and aggregation of vehicles by classification using the Federal Highway Administration's (FHWA's) 13-vehicle classification method;
- volume, speed, and classification dataset processing and quality control;
- speed data analysis:
  - calculation and analysis of the general statistics for the study curve:
    - by curve location (far upstream, upstream, start of curve, and middle of curve),
    - vehicle class (passenger vehicles, rigid trucks, and heavy trucks), and
    - study period (before, early-after, and late-after);
  - before-after comparison of the mean speed for the study curve by:
    - curve location,
    - vehicle class, and
    - study period;
  - Analysis of Variance (ANOVA) for mean speeds at each curve and vehicle type;

- in-curve speed reductions by vehicle type and study period (before, early-after, and late-after); and
- driver compliance of the advisory speeds at different locations on the curve (start of curve or middle of curve) by vehicle type and study period.

Chapter 6 of this report provides more detail on each of these steps and presents the results of each analysis conducted in this project.

TASK 5. RECOMMENDATIONS FOR IMPLEMENTATION OR FURTHER STUDY

Researchers evaluated the converging chevron pavement marking treatment as outlined in Task 4. The results of the analysis were used to formulate conclusions about the hypotheses that were initially proposed:

- Is there a significant reduction in overall mean speed at different points of the curve?
- Is there a significant reduction in mean speeds for different vehicle classes at different points of the curve?
- Do daylight and non-daylight conditions have an impact on effectiveness of chevrons?
- Can converging chevron pavement markings be an effective tool for long-term benefit in speed reduction?

The primary goal of this project was to evaluate the effectiveness of the converging chevron pavement marking treatment in reducing speeds on freeway-to-freeway connector ramps. Given the results of the project, researchers were then able to make some recommendations on the potential for the converging chevron pavement marking concept as a tool for reducing speeds on freeway-to-freeway connector curves. Chapter 7 presents a discussion summarizing the findings of the project and documents suggestions for further implementation of the converging chevron pavement marking treatment.

CHAPTER 3. LITERATURE REVIEW

Of the geometric elements that characterize our freeway systems, freeway-to-freeway direct connector ramps may be considered some of the more complicated features that drivers must negotiate. Speeds required to safely traverse freeway connector ramps are often much lower (by 15+ miles per hour) than the upstream free flow speed. This makes deceleration and selection of an appropriate speed problematic for some drivers. The combination of horizontal and vertical curvature at freeway-to-freeway connectors often limits sight distance, which complicates speed selection with drivers not being able to view the curve in its entirety.

Several studies have focused on freeway-to-freeway connector ramps over the past few decades. Two particular aspects or operations on freeway-to-freeway curves mentioned most often are: 1) the connector's impact on truck operations; and 2) recognition that the current advisory speed setting guidelines may result in the posting of unrealistic advisory speeds as compared to field observed operating speeds. This section of the report reviews the available literature found that addresses these concepts.

This project focused on the operations of trucks at freeway-to-freeway connectors. However, the converging chevron pavement marking treatment is non-vehicle classification specific, and the prevention of all crashes on freeway-to-freeway connector ramps is the goal. Heavy trucks and truck-trailer combinations, as compared to lighter, more maneuverable passenger cars, light trucks, and sport-utility vehicles, have limitations on the vehicles' ability to traverse horizontal curves on freeway-to-freeway connectors. These limitations include size and weight characteristics, mechanical performance parameters, and dynamics of the cargo loading to name a few. Excessive speed when entering or traversing a horizontal curve causes many truck rollover incidents.

There are likely several reasons why drivers exceed the posted advisory speed on a freeway-to-freeway connector. The most prominent reasons for drivers, especially truck drivers, to exceed a posted advisory speed include the desire of the driver to hold speed for merging into freeway mainlanes and inadequate deceleration distance entering the connector. In addition, as discussed earlier, drivers may also lack understanding of the many geometric limitations of freeway connectors.

Drivers of passenger vehicles typically want to exceed the posted advisory speeds on freeway-to-freeway connector curves for some of the same reasons as truck drivers. Although this is true, the consequences of a passenger vehicle crash on a freeway connector may have less of an impact on the freeway system than crashes involving larger and heavier vehicles. Truck crashes on freeway-to-freeway connectors can significantly affect the capacity and mobility of freeway facilities, especially during peak traffic periods.

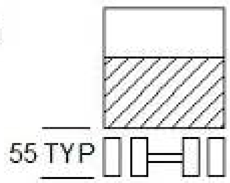
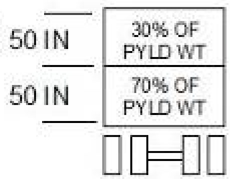
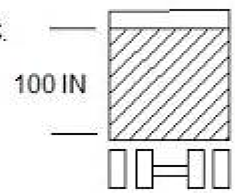
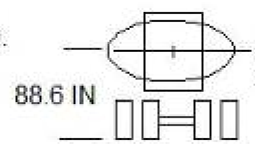
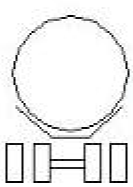
The basis for geometric design of freeways and freeway-to-freeway connectors in the United States is AASHTO's Green Book *A Policy on the Geometric Design of Highways and Streets* *(4)*. Over its history, the AASHTO Green Book has provided design guidance based primarily on passenger vehicle representations and not necessarily from a perspective of heavy trucks. As a result, many freeway-to-freeway ramps may not adequately accommodate the different operational parameters of trucks, and the AASHTO guidelines may be out of date in terms of setting the advisory speeds for modern passenger vehicles. Highway alignments depend on developing a preferred design based on trade-offs between several mitigating factors. The trade-off often involves the cost for right-of-way plus the cost of construction, against vehicle operating costs and safety. The horizontal alignment features that govern a given vehicle's performance on a curve include radius (or degree of curvature) and pavement width. Other factors necessary to define the design include design speed, superelevation rate, and side friction factor. All of these factors work together during the design process to determine a safe and efficient freeway-to-freeway connector ramp curve alignment.

As a truck travels through a curve, speed, combined with ramp curvature and superelevation, creates lateral acceleration *(5)*. For every truck and cargo loading circumstance, there is a maximum lateral acceleration threshold that, if exceeded, will cause the truck to roll over *(6)*. The University of Michigan Transportation Research Institute (UMTRI) developed typical rollover threshold values for various trucks and loading conditions using various static and dynamic testing *(7, 8)*. Figure 1 presents these thresholds values.

Side friction factors recommended for design are based on driver comfort levels and not necessarily on the physics of passenger cars or trailers pulled by trucks. For example, with a semi-trailer combination with a rollover threshold of 0.35, the margin of safety for skidding or rolling increases as the design speed increases *(9)*. Additional examples of this can be illustrated using four curves with the same superelevation of 0.04.

- Truck rollover speeds would be:
    - 27 mph for a 20 mph curve (margin of safety: 7 mph), and
    - 40 mph for a 30 mph curve (margin of safety: 10 mph).
- But the truck would skid at:
    - 54 mph on a 40 mph curve (margin of safety: 14 mph), and
    - 67 mph on a 50 mph curve (margin of safety: 17 mph).

While the mode of failure changes from rollover to skid as speeds increase, there appears to be much less margin for error on lower speed ramps when designed to current design criteria. These factors indicate that the most dangerous situations for trucks, given current design criteria, are on the lower-design speed curves, typical of many freeway connector ramps. This condition may also indicate the need for additional truck-specific warning devices for these types of curves.

CASE	CONFIGURATION	WEIGHT (LB) GVW	PAYLOAD CG HEIGHT (IN.)	ROLLOVER THRESHOLD (G's)
A.	 FULL GROSS, MEDIUM-DENSITY FREIGHT (34.0 LB/CF)	80,000	83.5	0.34
B.	 TYPICAL LTL FREIGHT LOAD	73,000	95.0	0.28
C.	 FULL GROSS, FULL CUBE, HOMOGENEOUS FREIGHT (18.7 LB/CF)	80,000	105.0	0.24
D.	 FULL GROSS, GASOLINE TANKER	80,000	88.5	0.32
E.	 CRYOGENIC TANKER (He2 and H2)	80,000	100.0	0.26

Source: *(5)*

Figure 1. Rollover Thresholds for Various Heavy Vehicles.

HORIZONTAL CURVES AND THE BALL-BANK INDICATOR

The most commonly used tool for selecting a posted advisory speed on horizontal curves is the ball-bank indicator. A study by Fitzpatrick et al. presented a survey indicating that 88 percent of states, cities, or counties that responded use the ball-bank indicator to set advisory speeds on curves *(10)*. The ball-bank indicator measures relative lateral acceleration that drivers and passengers sense while traversing a curve.

Merritt, in his *Safe Speeds on Curves: A Historical Perspective of the Ball-bank Indicator*, gave a general history of the use of the ball-bank indicator *(11)*. In 1935, the need for a consensus method to determine safe speeds on curves lead the Bureau of Public Roads to issue instructions for measuring superelevation and curvature and defined the maximum safe speed under normal driving conditions. The maximum safe speed was set at the minimum speed where the centrifugal force caused a driver or passenger to feel a "side pitch outward." The thought was that there would be a significant factor of safety between the higher speed at which an out-of-control skid would take place and the lower comfort threshold. This comfort feeling was curiously termed the "driver's judgment of incipient instability." After many driving experiments with test vehicles during the 1930s, researchers found that a 10-degree ball-bank reading was about equal to a side friction factor or 0.14 or 0.15, depending on the body roll of the vehicle *(12)*.

Testing in the mid-1930s indicated that the maximum side friction that a driver would accept before discomfort was about 0.14 or 0.15; therefore, the 10-degree ball-bank limit was deemed a close fit to the side friction at discomfort for higher speeds *(11)*. For lower speeds, it was found that drivers would accept higher levels of side pitch due to the perceived lessened consequences of a mistake, thus using the 12-degree reading for curves of 30 mph and 14 degrees for curves of 20 mph or less became more accepted. These recommendations were promulgated throughout many publications over the next several decades and were included in AASHTO policies in the late 1930s and early 1940s. These recommendations are also stated in various handbooks and guidelines, including the Institute of Transportation Engineer's (ITE) *Transportation and Traffic Engineering Handbook*, *Traffic Control Devices Handbook*, and some older federal and state versions of the *Manual on Uniform Traffic Control Devices*. Merritt notes that since these guidelines were produced, there have been significant improvements in roadway and vehicle characteristics. However, he states that the criteria based on 1930s' technology remains an accepted method to determine maximum safe speed on curves. He later explains that attitudes about these guidelines have changed recently to use higher ball-bank readings to set advisory speeds.

The ball-bank test studies are typically made with a driver and an observer. After checks of calibration to ensure that the ball-bank indicator is calibrated to "zero" when the vehicle is in a horizontal position and at rest, the vehicle is driven on the subject curve at a constant speed parallel to the center of the curve *(11)*. The criterion for setting the advisory speed on the curve is the speed associated with a ball-bank reading of 14 degrees for speeds below 20 mph, 12 degrees for speeds between 20 and 35 mph, and 10 degrees for speeds of 35 mph or greater. The decision to provide an advisory speed sign is made when the safe operating speed as determined by the ball-bank indicator study is less than the prevailing speed on the roadway. The value posted on the sign usually corresponds to the lowest speed (to the nearest 5 mph) obtained during trial runs that created a target ball-bank reading within the suggested speed ranges *(12, 13)*.

The physics that explain the mathematical relationships involved in depicting motion around a horizontal curve can be described using several equations *(4, 14)*. Given that a vehicle is moving at a constant speed *v* on a curve or constant radius *R*, the acceleration is directed toward the center of the circle, perpendicular to the velocity at any instant. This phenomenon is termed *centripetal acceleration* (or *lateral acceleration* in highway engineering) and is represented by Equation 1:

$$a_{per} = \frac{v^2}{R} \tag{Eq. 1}$$

where:

a_{per}	= centripetal acceleration (ft/s^2),
v	= velocity of vehicle (ft/s), and
R	= radius of curve (ft).

As a vehicle generates this measure of lateral acceleration as it traverses a curve of constant radius, each is counterbalanced by the vehicle weight, roadway superelevation, and side friction development between the tires and pavement surface.

The AASHTO Green Book uses a point mass model to determine the minimum radius of curvature for a superelevation rate and design speed such that the lateral acceleration may be kept at a desirable maximum level based on driver and passenger comfort. When combined with the second law of physics, the point mass model equation used to represent vehicle motion on a horizontal curve is:

$$e + f = \frac{v^2}{15R} \tag{Eq. 2}$$

where:

e	= superelevation rate (decimal),
f	= side friction factor,
v	= speed (mph), and
R	= radius of curve (ft).

Equation 2 should be thought of as a supply-demand equation. The left side of the equation represents the amount of lateral acceleration supplied, while the right side represents the lateral acceleration that is demanded for the vehicle to safely travel around the curve.

Traffic engineers have historically used the ball-bank indicator to determine a threshold operating speed that causes discomfort for drivers and passengers on curves. The ball-bank unit consists of a steel ball enclosed in a glass tube. The ball moves freely, with the exception that the movement is dampened by the liquid that fills the tube *(4)*. The ball-bank reading can be expressed by the following equation:

$$\alpha = \theta - \varphi + \rho \tag{Eq. 3}$$

where:

α = ball-bank reading,
θ = body roll angle,
φ = centrifugal force angle, and
ρ = superelevation angle.

Moyer and Berry recommended overlooking the body roll term of Equation 3 as long as the observers understood its impact *(12)*. Carlson and Mason examined this assumption further and confirmed that the knowledge of the body roll of the passenger car vehicle (using a Ford Taurus) was unnecessary to calculate safe speeds on curves, as it was found to be statistically insignificant *(14)*. Carlson and Mason concluded that ball-bank indicators could be correlated directly with driver comfort and lateral acceleration values used in curve design; however, no further studies to examine the validity of the AASHTO values of lateral acceleration were recommended.

At the time of the Carlson and Mason study (1999), the following AASHTO guidelines for setting advisory speeds on curves were in effect:

- maximum 14 degrees for speeds 20 mph or less,
- maximum 12 degrees for speeds 25 to 30 mph, and
- maximum 10 degrees for speeds 35 to 50 mph.

Again, these criteria were based on tests conducted in the 1930s and were intended to represent the 85- to 90-percentile curve speed. These limits correspond to side friction values of 0.21, 0.18, and 0.15, respectively. Chowdhury et al. argue that these side friction values reflect an average comfortable speed and that modern cars on dry pavement are capable of reaching side friction coefficients of 0.65 and higher before skidding *(15)*. These guidelines resulted from the Moyer and Berry study of vehicles in the 1940s *(12)*.

Over the past few decades, various research efforts have presented arguments that these criteria may no longer be appropriate given the changes in vehicle stability and driver comfort levels. A Transportation Research Board paper by Chowdhury, Warren, Bissell, and Taori suggested that the existing criteria be changed to the following:

- maximum 20 degrees for speeds 30 mph or less,
- maximum 16 degrees for speeds 30 to 40 mph, and
- maximum 12 degrees for speeds 40 mph or higher *(15)*.

The Chowdhury et al. study further concluded that at most curves the posted advisory speeds were not only well below the prevailing traffic speed but also below the posted advisory speed that would be recommended by the existing ball-bank criteria *(15)*. The study further argued that the ball-bank criterion suggests driver discomfort thresholds at very low and unrealistic associated operating speeds and concluded that this is why the profession should not expect compliance from drivers. Note that this study did not appear to distinguish trucks from passenger vehicles.

The disparity between the AASHTO advisory speed-setting criteria and operating speeds on curves has recently been recognized and codified into the 2003 federal version of the Manual on Uniform Traffic Control Devices. The Manual on Uniform Traffic Control Devices (2003 Edition) indicates in Section 2C.36 that:

> *"A Curve Speed sign may be used at and beyond the beginning of a curve following a Horizontal Alignment and Advisory Speed sign combination, or when there is a need to remind road users of the recommended speed, or where the recommended speed changes because of a change in curvature (see Section 2C.06). Based on engineering judgment, the Curve Speed sign may be installed on the inside or outside of the curve to enhance its visibility.*
>
> *The advisory speed may be the 85th percentile speed of free-flowing traffic, the speed corresponding to a 16-degree ball-bank indicator reading, or the speed otherwise determined by an engineering study because of unusual circumstances.*
>
> *Support: A 10-degree ball-bank indicator reading, formerly used in determining advisory speeds, is based on research from the 1930s. In modern vehicles, the 85th percentile speed on curves approximates a 16-degree reading. This is the speed at which most drivers' judgment recognizes incipient instability along a ramp or curve (16)."*

The 2006 Texas Manual on Uniform Traffic Control Devices (TMUTCD) omits the federal language quoted above and does not reference recommended ball-bank readings to be used for advisory speeds on curves *(17)*. The 2003 TMUTCD also does not contain language or guidance on the recommended ball-bank readings to be used to determine advisory speeds for curve warning signs *(17)*.

One study of curve operations in New Zealand also found results similar to recent studies in the United States *(18)*. The study suggested changing New Zealand's advisory speed system to more accurately reflect the actual operating speed. This study also compared the methods of determining lateral accelerations. Researchers studied readings from both the ball-bank indicator and accelerometer and concluded that both devices may be used to set advisory speeds. Researchers also concluded that any data collected by an accelerometer should be smoothed to reduce lateral acceleration peaks, avoiding potential errors in suggesting appropriate advisory speeds.

A study by Voigt et al. examined the speeds of various types of vehicles on freeway-to-freeway connector ramps in Houston, Texas *(1)*. Researchers collected speed data at chosen locations to determine compliance with posted advisory speed limits and average speeds at points along connector ramp curvature. From these measurements, researchers determined that the driving public often exceeds the posted advisory speed limit, sometimes by more than 10 mph. In addition to examining speed characteristics, the researchers conducted lateral acceleration studies on four different vehicles: passenger car, sport-utility vehicle, dump truck, and 18-wheeler tractor-trailer combination with a loaded trailer. The vehicles were driven through the curves at varying speeds, while researchers monitored a manual ball-bank indicator in addition to collecting lateral acceleration data electronically. While there were no seemingly discernable differences in lateral accelerations by type of vehicle for a given speed along a curve, there appeared to be a 5 to 10 mph difference in the operating speed that caused driver discomfort between passenger cars/sport-utility vehicles and larger vehicles.

The findings of the Voigt et al. study indicated that there may be differences between the maximum comfortable speeds that drivers of heavy vehicles and passenger car-type vehicles will accept while traversing a freeway-to-freeway curve. The study concluded that designers should take care to provide adequate deceleration and acceleration distances for tractor-trailers and other heavy vehicles; reduce, where possible, the side friction demand on trucks in the curve by developing superelevation more on the tangent; and place curve advisory speed signing with more regard to the deceleration needs of trucks. The authors also recommended modifying the current advisory speed setting criteria to use a 10-degree ball-bank indication level to set a truck advisory speed and a 13-degree ball-bank level for setting passenger car advisory speeds. These lateral acceleration levels are thought to better represent the 85th percentile speed of the two vehicle types during curve traversal.

TRUCK OPERATIONS

Ervin et al. recognized several cases where roadway geometrics or driver misjudgment may increase the potential for freeway connector crashes *(19)*. The following three cases are most important to this project:

1. Side friction factor is excessive given the roll stability limits of many trucks.
2. Truck drivers assume that the ramp advisory speed does not apply to all curves on the ramp (if there is more than one curve).
3. Deceleration lane lengths are deficient for trucks, resulting in excessive speeds at the entrance of sharply curved ramps.

For the first case, the authors assert that the margin of safety for trucks on horizontal curves designed to AASHTO guidelines is much less than the margin of safety for passenger cars. Considering that for many horizontal curves (and as specified in AASHTO guidelines) the superelevation is not fully developed until well into the curve, this means that the side friction factors in some parts of the curve, especially the beginning, are typically higher than the side friction factors used in determining the design superelevation. These side friction factors, in some cases, may exceed the static rollover thresholds that exist for many fully loaded, high center-of-gravity tractor-trailers.

The lower stability threshold of a truck-trailer combination results from the height of the center of gravity of the truck's payload relative to the tractor-trailer's track width, along with many other parameters such as suspension, tires, etc. The general relationship, assumed to be valid for curve design, is that the roll stability limit is:

$$g = t_w / (2*h_{c.g.}) \qquad \text{(Eq. 4)}$$

where:

g = roll stability limit;
t_w = track width, or distance between tires on opposite ends of the axle; and
$h_{c.g.}$ = height of the center of gravity.

Equation 4 is only valid when the trailer is considered rigid. However, trailers tend not to be rigid and may flex under stressed conditions. Ervin et al. state that the roll stability limit may be reduced by nearly 40 percent when considering actual truck-trailer flexibilities. This reduction becomes critical when considering that a non-rigid trailer produces enough g to quickly approach the rollover threshold at side friction factors very near design limits. Consider a truck, with a very high center-of-gravity trailer, which is exceeding an advisory speed that was selected according to existing guidelines. In this situation, a good possibility exists that a rollover incident will occur simply because of the physics involved with a flexing trailer. In addition, the incident could take place without ever exceeding the comfort level of the truck driver. Some truck drivers realize this phenomenon; some inexperienced truck drivers may not.

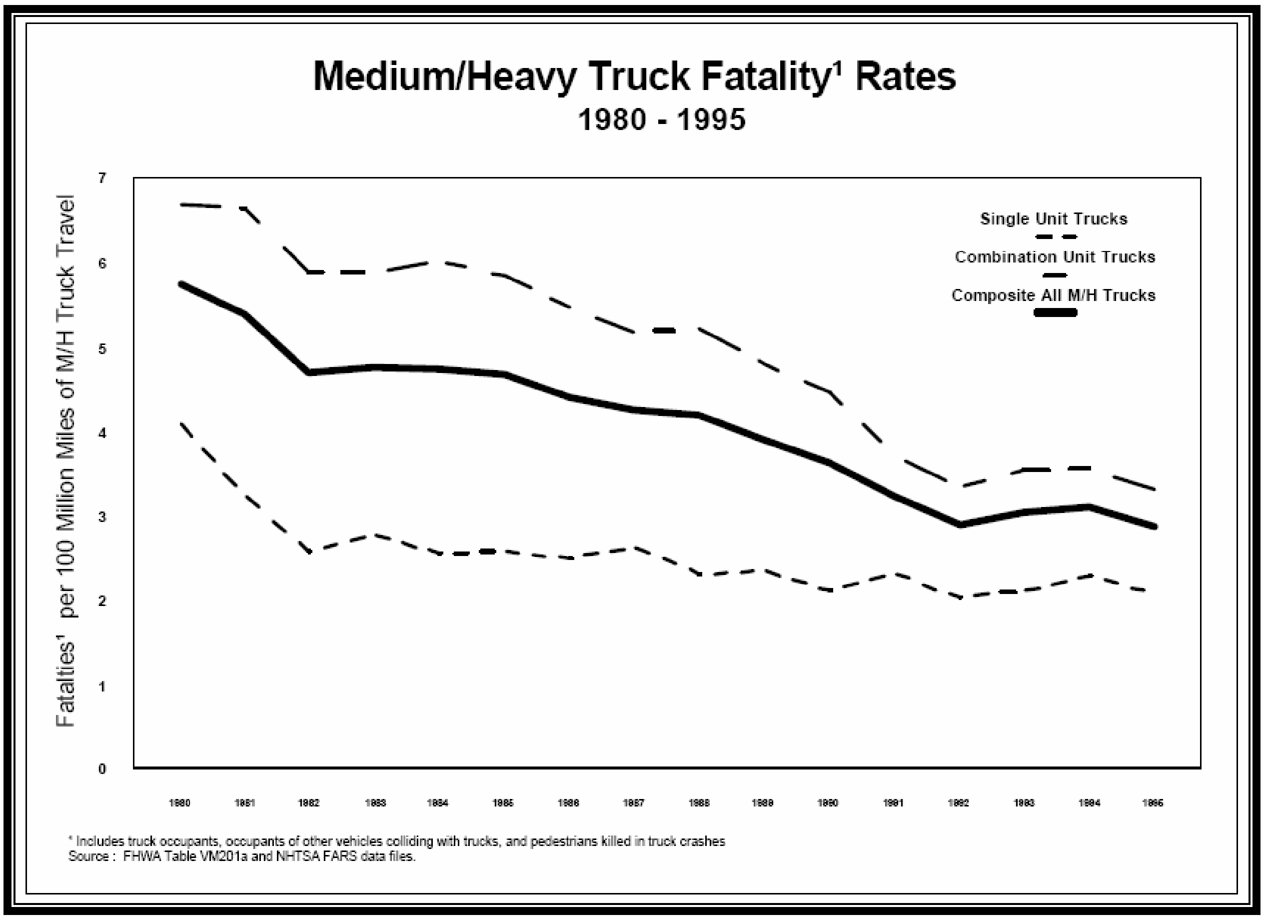
Ervin et al. argue that many truck drivers assume that the first advisory speed for a multiple curve ramp is for the first curve, when the limiting curve may be further downstream. It may be observed that truck drivers tend to accelerate after leaving the first curve in order to reach the speed needed to merge with high-speed freeway mainlane traffic, only to then encounter a second curve requiring a slower speed. This is of particular concern for connector ramps on a downgrade. This geometry along a connector ramp may cause not only rollover crashes but jack-knife crashes as well. If a truck driver recognizes the upcoming curve and identifies a need to slow down, the onset of heavy braking to reduce speed may cause the cargo to shift and increase the risk of a jack-knife event.

Ervin et al. also concluded that deceleration lanes are not long enough for trucks to reduce speeds and safely negotiate a curve *(19)*. This rationale was because the previous design guidelines assumed that average speeds for trucks are generally lower than those of passenger cars. Although the latest AASHTO Green Book did not repeat this assumption, the publication also did not significantly change its recommendations for deceleration lengths. Recent observations could also dispute this assumption as truck speeds appear to be equal to passenger car speeds in most cases.

The *Comprehensive Truck Size and Weight (TS&W) Study* also cited several previous studies that identify concerns with instances of excessive side friction factor demand and limited deceleration lengths *(20)*. The study indicated that trucks with rollover thresholds of 0.30 g can roll over on freeway ramps when traveling as little as 5 mph over the design speed. In similar fashion to studies performed by Ervin et al., this study also recognized that, in many cases, the length of deceleration lanes is not adequate to accommodate the characteristics of safe truck deceleration *(19)*. Vehicles failing to correctly transition from freeway mainlanes that have higher design speeds than the connector ramp curvature may enter with excessive speed. Excessive speed combined with a lack of adequate deceleration length may lead to rollover crashes. The TS&W study also referenced an ITE publication that compared deceleration lane requirements as stated in the AASHTO Green Book (for passenger vehicles) and those requirements that would be required by trucks and found that deceleration lengths would have to increase by more than 50 percent to adequately accommodate the operational characteristics of trucks *(21)*.

SAFETY

The TS&W study indicated that medium to heavy trucks account for 3 percent of vehicles in use on United States roadways and that trucks account for 7 percent of vehicle miles of travel *(20)*. Trucks are involved in only 3 percent of all crashes but account for 8 percent of involvement in fatal crashes. Figure 2 shows that the relative involvement of trucks in fatal crashes has decreased in the last decade *(20)*.



Source: *(20)*.
Note: NHTSA FARS is the *National Highway Traffic Safety Administration Fatality Analysis Reporting System*

Figure 2. Medium/Heavy Truck Fatality Rates, 1980-1995.

The following factors contributed to this decline:

- the use of uniform truck driver licensing and tracking of violations under the federal/state Commercial Driver's License Program,
- increased federal and state inspections and audits completed under the Motor Carrier Safety Assistance Program,

- upgrades in training and safety awareness at institutions abiding by guidelines published by the Professional Truck Driver Training Institute,
- awareness of safety management, and
- advances in safety technology in truck designs (seat belts, anti-lock braking systems, under ride guards, etc.) *(20)*.

Although each of these factors is important, the most critical component in the safe operation of a heavy truck is driver performance. Factors that affect overall driver performance include skill level, experience, awareness, and fatigue. While experienced drivers may have developed the skills necessary to overcome difficult driving conditions or vehicles with less than desirable stability characteristics, inexperienced drivers are more prone to crashes because of these characteristics. One of the most common crash causative factors attributed to the judgment of the driver is traveling at excessive speed *(20)*. Professional truck drivers are typically male and older than the general driving population. However, studies have indicated that younger truck drivers are involved in more crashes than older truck drivers are, a statistic that parallels that of the general driving population *(21)*. Other studies have noted that truck drivers have negative opinions of other drivers, but they do not demonstrate "self-enhancement" that indicates overconfidence *(22)*. As a group, truck drivers do not believe that just because they drive more miles or because they drive a truck, they should become (or feel) overconfident about their abilities. Because they view themselves as driving professionals, more experienced truck drivers use their experience to try to avoid negative driving situations. More recently, the Transportation Research Board's Commercial Truck and Bus Safety Program produced *Synthesis 4 – Individual Differences and the "High-Risk" Commercial Driver*, which provides a very good overview of the factors related to driver risk and confirms earlier works completed on the subject *(23)*.

While the driver is the most critical factor in the safe operation of a truck, the driving environment may also have significant effects on truck operations. Roadway features, traffic congestion, and weather all contribute to the overall operational capabilities of both the driver and vehicle. Roadway features that may affect truck operations include roadway surface type and grade, interchange and intersection geometry, entry and exit ramps, and acceleration and deceleration lanes. Visibility also has a significant impact on truck operation safety. The TS&W study determined that about 35 percent of fatal crashes and 26 percent of nonfatal crashes occur in conditions other than normal daylight. Inclement weather conditions (rain, sleet, snow, ice, fog, and standing water) always present a challenge to the truck driver and may influence the operating characteristics of the truck. Weather and poor visibility both may combine to reduce the available factor of safety for sight distance, decision distance, and time available for evasive maneuvers *(20)*.

Several studies have quoted crash rates for trucks. Janson et al. estimated that 20 to 30 percent of freeway truck crashes occur at or near ramps, despite the fact that interchanges account for less than 5 percent of freeway miles *(24)*. Rollover crashes account for 8 to 12 percent of all truck crashes but account for 60 percent of all truck driver/occupant fatalities *(20)*. These types of crashes are extremely disruptive to the freeway network in the urban environment, especially when hazardous materials are involved. The trucking industry could reduce rollovers by making trailers more roll-stable by using lower deck heights, more axles, and/or stiffer suspensions. However, an immediate help in reducing rollover crashes is for truck drivers to adhere to the posted (or reasonable) advisory speeds through the entire length of a freeway ramp or curve *(20)*. Other studies found that a disproportionate amount of truck rollover crashes occur on freeway ramps (17 percent) *(25)*. A study by Garber et al. found that truck crashes increase on freeway ramps with an increase in ramp curvature and with the differential between the truck speed on the curve approach and the posted advisory speed on the ramp *(26)*.

The study by Janson et al. concluded that no statistical relationship could be found between crashes and roadway geometry (grade, curvature, or curve length) *(24)*. This study concluded that traffic crashes are random events with many causative factors, including driver factors that complicate the identification of specific causes for crashes. This study presents a method to "flag" crash-prone ramps for further investigation and potential improvements and summarizes the process in three steps of statistical analysis. However, these procedures are highly dependent on crash reporting measures that may not be available in typical crash reporting procedures.

The American Automobile Association (AAA) Foundation for Traffic Safety recently completed a study based upon the Fatality Analysis Reporting System (FARS) data for 35,244 fatal car crashes and 10,732 fatal car-truck crashes for 1995–1998 *(27)*. This analysis supports previous studies *(20, 24, 25, 26)* of fatal car-truck crashes but also shows that unsafe actions by car drivers are more likely to be recorded than unsafe actions by truck drivers—80 percent for passenger cars compared to 27 percent of heavy vehicles (with at least one unsafe driving act recorded in FARS). Of unsafe actions examined in the AAA study of fatal crashes, 75 percent were linked to car drivers and 25 percent were linked to truck drivers. The majority of the crashes were attributed to just a few unsafe driving actions (independent of whether car- or truck-driver was involved). Five of the 94 listed potential attributing crash factors accounted for about 65 percent of the unsafe driving actions by drivers. The top five factors were:

- failure to stay in the lane or running off of the road (21 percent),
- failure to yield the right-of-way (16 percent),
- driving too fast for conditions or above the speed limit (12 percent),
- failure to obey signs and signals (9 percent), and
- driver inattention (9 percent).

PAVEMENT MARKINGS FOR SPEED REDUCTION

This section reviews some of the pavement marking patterns that have been used as a traffic engineering tool for speed control. The use of converging chevron pavement markings and transverse bar markings for speed reduction is reviewed in detail.

Chevron Pavement Markings

Chevron pavement markings are a type of pavement marking pattern intended to influence driver behavior resulting in a reduction in speed. Chevron pavement markings consist of a set of inverted 'V' shape or arrow shaped markings with some spacing between each individual chevron marking. A report suggests that the shape of chevron marking is derived from a French word "Chevron," which means rafter *(28)*. There are several ways chevrons are designed to achieve required speed reduction or maintain safe distance to avoid collision. Some of the studies involving chevron markings will be further discussed in this section.

Chevron Pavement Marking Studies in Japan

One study reported on the use of chevron pavement markings in Japan *(29)*. A converging pavement marking pattern was applied on the Yodogawa Bridge in Osaka, Japan. Converging chevron markings with broken lines on either edges of the lane were used in this study (as shown in Figure 3) to reduce speed by creating an illusion to the driver that the vehicle is over speed. Chevron pavement marking sets were placed increasingly closer as a driver moved into the pattern. When the driver moves along the pattern, more sets of chevrons are crossed in a given unit of time, giving an illusionary effect of speeding, potentially leading to a reduction in vehicle speed. The broken, transverse edge lines provide an illusion of the lane narrowing and hence requiring more driver attention. Specifications of converging chevron placement used on the Yodogawa Bridge and the quantitative benefits that resulted from the use of this pavement marking are not available in any of the reports reviewed in this study. However, the study does claim that no accidents occurred in the six-month period after chevron markings were deployed, whereas 10 accidents had been reported in the one-year period prior to installation of the chevron pavement markings on the Yodogawa Bridge *(29)*.

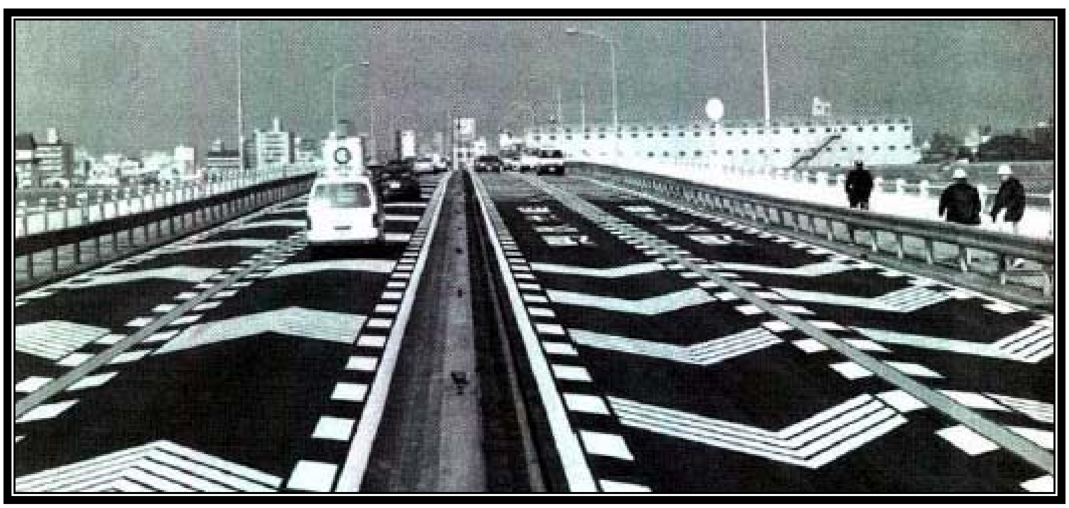


Figure 3. Chevron Markings on Yodogawa Bridge, Japan *(28)*.

Another study from Japan, reported by Mr. Kazuyuki Terada in 1997, examined the accident-reduction effectiveness of speed reduction markings. This study was reviewed by researchers at Marquette University. The review stated that some of the objectives of this study were to observe crash experience, lane changing behavior, and vehicle positioning characteristics within the lane due to installation of chevron pavement markings on six different sections of roadway *(30)*. The sections consisted of left-curving, right-curving, and tangent sections. The study used data from two years before installation of the chevrons and two years after. Results of the study as summarized by Marquette University indicated that the crash frequency was reduced overall; however, the reduction in crash numbers was very small in four out of six study sites. Though there was a consistent reduction in accidents at the other two study sites, the overall reduction was found to be statistically insignificant. Speed observations on one of the non-curved segments, where the average speeds ranged between 64 mph and 86 mph, showed that in general, overall average speeds were reduced due to the chevron pavement markings. However, a per-lane analysis indicated that left and middle lanes had an increase in average speed after chevron pavement markings were installed, especially with the speed data collected during the morning peak hours. There was a 7.5 mph average increase in speed for the middle lane during the morning peak hours. The speed in the right lane dropped by 8.75 mph for automobiles during morning peak hours and by 6.88 mph for trucks in the evening peak hours. However, the effectiveness of the chevron pavement markings in causing speed reduction was inconclusive due to the mixed results observed. Observation of lane changing behavior on six segments, three with chevron markings and three without chevron markings, indicated that passenger cars change lanes fewer times when chevron markings were used compared to the unmarked segments, but trucks had mixed results in different segments. Researchers at Marquette University suggested that a study period of two years was insufficient to get any conclusive results on the safety or speed-reduction effectiveness of chevron pavement markings.

Researchers at Marquette University also provided discussion on a summary of three other Japanese papers dealing with effectiveness of chevron and comb markings in speed reduction *(29)*. Figure 4 shows the chevron and comb patterns reported in these studies.

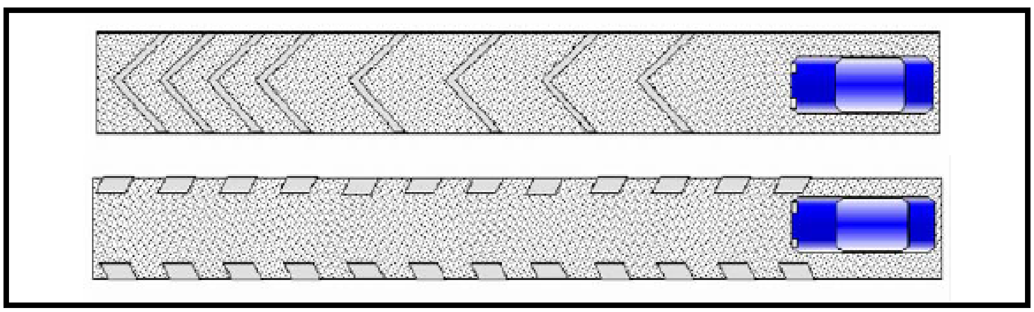


Figure 4. Chevron Marking Pattern with Comb Markings *(28)*.

The Marquette summary of the three papers concluded that chevron pavement markings used with roadside delineators and chevron signs were found to be effective in reducing speed just before entering a curve. Comb pavement markings reduced speed from 1 to 3.6 mph and also contributed to a reduction in the number of lane changes. The Marquette review stated that in the summary they obtained, quantified results were not available, so the operational conclusions are uncertain. Figures 5 and 6 present the two selected study site photographs of the Japanese studies, as obtained and published by researchers at Marquette University. The figures show the use of anti-skid markings that were deployed in an original study in Japan *(30)*. The anti-skid chevron pavement marking installation as illustrated in Figures 5 and 6 enhances road surface friction. This anti-skid setup could be especially useful when chevrons are used on high speed curved roadways or in locations where wet weather is frequent.

Figure 5. Anti-skid Chevron Markings *(30)*.

Figure 6. Closer View of Anti-skid Chevron Markings *(30)*.

The M1 Chevron Marking Study in the United Kingdom

A 1995 study in the United Kingdom documented the use of chevron pavement markings on the M1 *(30)*. Figures 7 and 8 show the chevron pavement marking scheme used in this study. The primary intention of this study was not to induce speed reduction but to encourage drivers to maintain a sufficient gap in order to reduce the potential for accidents at the study site.

As seen in Figure 8, the installation of chevron pavement markings consisted of chevrons spaced approximately 122 feet apart and supplemented with a roadside sign instructing the drivers to maintain at least two chevrons headway distance. Figure 7 shows the pattern of chevron applications, and Figure 8 shows the associated sign indicating "Keep Apart 2 Chevrons." Chevron pavement markings were installed on the slow (left lane) and middle lanes of the three-lane rural highway. The purpose of this deployment was to encourage drivers to keep sufficient headway to avoid rear-end collisions. At a driving speed of 70 mph, this configuration would result in 2.4 seconds of headway.

Before and after data were compared to quantify the effectiveness of the chevron setup. Three years of pre-chevron pavement marking installation and two years of post-chevron pavement marking installation were analyzed. Study results showed a 56 percent reduction in total crashes and 40 percent reduction in multi-vehicle collisions with the chevron pavement marking installation as compared to the control sites. Also, single vehicle collisions were reduced from eight per year to a total of just two crashes in two years of post-chevron pavement marking installation. Researchers also investigated the possibility that crashes migrated further downstream of the test section but did not find any evidence of such a migration *(30)*.

The United Kingdom researchers also conducted a public attitude survey along with the experiments and received positive feedback from those questioned *(28)*. The overall results of this study showed that the chevron pavement marking deployment was effective in achieving the desired objectives of reducing crashes and improving safety.

Figure 7. M1 Chevron Marking in United Kingdom *(28)*.

Figure 8. Sign Associated with M1 Chevron Markings in UK *(30)*.

The Milwaukee, Wisconsin Study

A 1999 deployment of a converging chevron pavement marking pattern was evaluated in Milwaukee, Wisconsin *(30)*. The experimental converging chevrons were installed on an exit ramp on IH 94 (as shown in Figure 9) with an installation cost of $40,000. The off-ramp consists of two lanes with both lanes marked with a chevron pattern. The AAA Foundation for Traffic Safety (AAAFTS) sponsored the evaluation project, and researchers at Marquette University completed the evaluation.

Figure 9. Converging Chevron Markings at Milwaukee, Wisconsin *(30)*.

The chevron pavement markings used in this study were very similar to that of the pattern used on Yodogawa Bridge in Japan. The study pattern consisted of a set of converging chevron markings, with longitudinal broken lines on either edge of the lanes. The intent was to encourage speed reduction by creating an illusion to the driver that the vehicle was over speed and the road width narrowing using the pavement markings. The sets of chevron pavement markings were placed increasingly closer together as the driver moved towards the curve to provide the illusionary effect of speeding.

Two ramps were used for comparison in this study: one test ramp and one control and comparison ramp. Figure 10 shows the aerial view of the test and the control sites, along with the detector location. Both ramps had similar average daily traffic (ADT) and geometry. However, the historical crash rates differed between the two study sites. Before the installation of the chevron pavement markings, the comparison site had a crash rate more than five times that of the test site.

Figure 10. Converging Chevron Markings at Milwaukee, Wisconsin.

The length of each individual set of chevron pavement marking patterns was based on the initial speed of vehicles entering the pattern, the desired speed when exiting, brake reaction time, and deceleration rate. Based on those variables, the converging chevron marking pattern resulted in a 610-foot long pattern consisting of 16 sets of chevrons, each set consisting of 10 individual chevrons. In comparison, the Japanese chevron pavement marking deployments varied from four to eight individual chevrons per set, with the number of chevrons per set gradually decreasing in the direction of travel.

The chevron pavement marking pattern design for the Milwaukee site was adopted from the Japanese study as shown in Figure 11. From the figure, it can be seen that the width of individual chevron was 6 inches and the width of the spacing between chevrons in a given set was 2 inches. Broken edge lines were 3.3 feet long segments with 3.35 feet spacing between the segments. However, unlike shown in Figure 11, each set had 10 chevrons as implemented in Milwaukee.

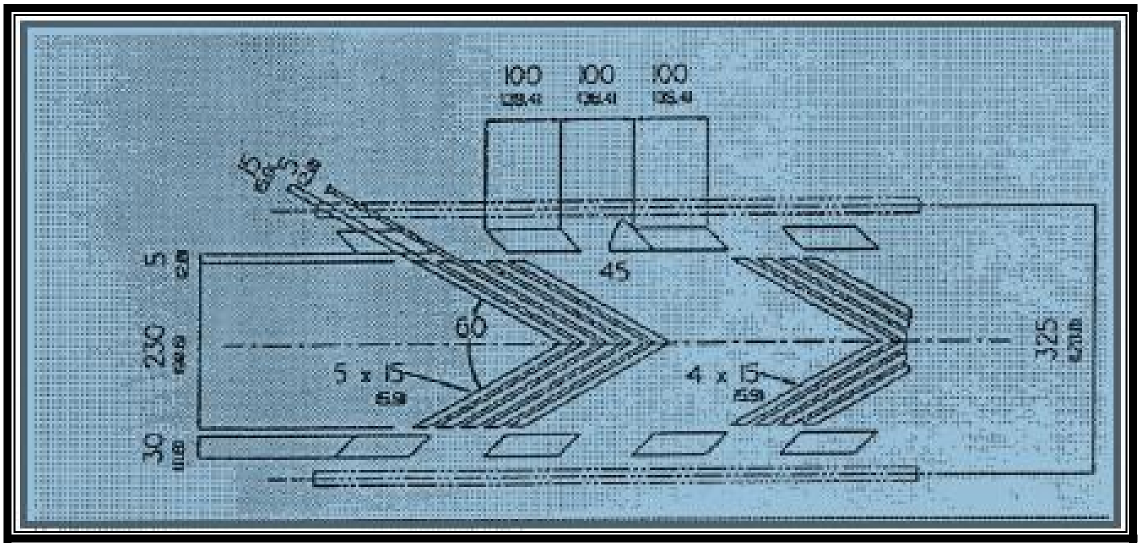


Figure 11. Chevron Pattern Design as Specified at Yodogawa Bridge, Japan *(30)*.

The upstream approach speed at the beginning of the chevron pavement marking pattern in Milwaukee was 65 mph, and the posted advisory speed at the start of the freeway-to-freeway connector curve at the end of the pattern was 50 mph. These operational characteristics were used to calculate the length of the pattern and spacing of the sets of chevron pavement markings. The calculations used by researchers to determine the length of the treatment is shown in Equation 5. The total length of the pattern, L, was calculated to be 610 feet.

$$L = v_1 t_b + \frac{\left(v_1^2 - v_2^2\right)}{2a} \qquad \text{(Eq. 5)}$$

where: v_1 = speed entering pattern
v_2 = speed exiting pattern
t_b = reaction time
a = deceleration braking

With an average speed of 57.5 mph across the chevron pavement markings, the estimated time to traverse the total pattern of 610 feet was 7.2 seconds. Assuming a time of 0.45 seconds to traverse a set, the number of sets to include in the pattern was determined to be 15.8 sets, rounded up to 16 sets. Since the chevron arrow was designed with a 60 degree internal angle and given the widths of the chevron pavement marking and the length of the spacing between markings, the pattern size along the direction of travel was determined by geometry. In the Milwaukee case, each of the chevrons in each set was 12 inches in width with spacing of four inches between each chevron pavement marking. Table 1 presents the spacing between chevron pavement marking sets that were used in the Milwaukee deployment. The distance values provided in the Table 1 are with respect to near the start of the freeway-to-freeway connector curve.

Table 1. Spacing Between Chevron Sets Used at Milwaukee (30).

Set	Distance
1	-618
2	-576
3	-534
4	-492
5	-450
6	-410
7	-370
8	-330
9	-292
10	-254
11	-216
12	-180
13	-144
14	-108
15	-74
16	-40

The Milwaukee study was initially intended to evaluate two years of speed and crash data pertaining to pre-chevron installation and two years of post-chevron installation data. Due to failure of hardware in the loop detectors after installation of the chevrons, only one year of post-chevron installation data was available for analysis. Table 2 shows the results of the before and after speed comparisons and shows a 14 mph reduction in the 85th percentile speed at detector B, near the beginning of the freeway-to-freeway connector curve.

Table 2. Speed Measurements as Reported from Milwaukee Study.

Detector	Before		After		After - Before	After - Before
	Mean	85th %tile	Mean	85th %tile	Mean	85th %tile
A	60	63	57	60	-3	-3
B	64	70	49	53	-15	-17
C	50	53	49	51	-1	-2
D	46	48	48	51	2	3

Researchers also reported a reduction in crashes after installation of the chevron pavement markings, but the reduction in overall crashes was not found to be statistically significant. Researchers did note, however, that when crashes due to snow and other factors (like collisions with animals) were removed from the analysis, the reduction in crashes was significant at a 90 percent confidence level. Researchers concluded that the converging chevron pavement marking treatment did have the potential to influence significant speed reduction, but they also recommended a more detailed study over a longer period of time. Previously there have been questions raised about the longevity of chevrons functional goals. There have been doubts that drivers could gradually get accustomed to the illusionary effects of chevrons, and the chevrons may not be as effective as they tend to be initially. However, the Milwaukee study, conducted 24 months after the installation of the chevrons, provides some insight into the longevity concerns of the chevrons' sustained ability to provide illusionary effect, and resulting speed reduction.

Other Chevron Pavement Marking Implementations in United States

Some of the other chevron pavement marking implementation studies were undertaken in the cities of Eagan, Minnesota, and Columbus, Ohio. A converging chevron pavement marking treatment was installed in 1997 in the City of Eagan on a residential street approaching an intersection with a posted speed limit of 30 mph. The before and after observation of the 85th percentile speed showed that a week after implementation there was a 15 percent reduction in speed, from 41 mph to 35 mph. Two years after the installation, the 85th percentile speed had increased back to 39 mph but was still lower than the 85th percentile speed before installation of chevron pavement markers. However, when the chevron pavement markings were repainted four years after the initial implementation, the measured 85th percentile speed dropped back down to 35 mph *(31)*.

A converging chevron pavement marking treatment was also deployed in Columbus, Ohio, in 1997. In this application the chevron pavement marking was implemented on a two-way, two-lane road approaching a double 'S' curve with a posted speed limits of 35 mph and 15 mph. Prior to the installation of chevron pavement markings, the 85th percentile speed on the facility was reported as 37 mph, but after about 15 months of installation, the speed was found to have reduced to 33 mph, an 11 percent decrease *(31)*.

Remarks on Chevron Pavement Marking Studies

Only a handful of studies were found in the literature that address the effectiveness of chevron pavement markings to reduce speed. However, while limited in the number of studies, the information available indicates that there is good potential for the use of chevron pavement markings as a traffic engineering tool to influence a driver reduction in speed. Most of these studies, however, also suggest testing various chevron pavement marking treatments, as well as suggesting more evaluation in order to better quantify the benefits of the treatment with respect to speed reduction and safety.

Transverse Bar Pavement Markings

The transverse bar pavement marking treatment is another marking scheme used for passive speed control. In terms of functionality, transverse bars are similar to chevron markings in that they create to the driver an illusion of over speeding and narrow lane width, thereby potentially influencing drivers to reduce their speed. Transverse bar pavement markings are more widely researched than chevron pavement markings. A brief review of some of the research efforts found in the literature involving transverse bar pavement markings follows.

Review of Research Studies Involving Transverse Bars

In 1971, Denton reported one of the earlier experiments on the effectiveness of transverse bar pavement markings to influence speed. In this study, Denton used driving simulators to investigate the effectiveness of transverse bar pavement markings by showing that gradually decreasing intervals between the bars resulted in significantly reducing the driver speeds *(28)*. Denton used a set of 90 yellow transverse bars with spacing between them exponentially reduced from an initial 20 feet to a minimum of 10 feet toward the end of the pattern. The results of this study, as shown in Table 3, indicated a consistent reduction in the 85th percentile speed, ranging from a 25 percent to 35 percent speed reduction during various periods throughout the day.

Table 3. Speed Reduction at Peak Periods as Reported from Milwaukee Study.

	Average Speed (in mph)			
	7–9 a.m.	**2–4 p.m.**	**6–8 p.m.**	**Mean**
Before Installation of Markings	48.1	47.2	45	46.7
After Installation of Markings	31.6	33.6	33.4	32.8
Percent Reduction in Speed	34.30%	28.80%	25.80%	29.80%

In 1975, Agent reported another evaluation of transverse bar pavement markings in the United States. Transverse bar pavement markings were installed just before a sharp horizontal curve with high crash rates on US 60 in Meade County, Kentucky. Prior to the installation of transverse bar pavement markings, 48 crashes were reported, out of which 36 were determined to be speed-related crashes. After the installation of the transverse bar pavement markings, a reduction in speed and crashes was observed at the study site. Speeds observed over the six-month period after the installation of transverse bars indicated a 12 mph drop in average daytime speed when compared to average speed before installation of the transverse markings *(32)*.

Researchers in Australia reported a literature review of perceptual counter measures for speed reduction. Several studies involving transverse markings were reviewed in this effort. After summarizing many significant experimental and simulation studies, the report concluded that transverse pavement markings had the potential to be a low-cost traffic engineering tool to reduce speeds. However, the review raised doubts on the mechanism of speed reduction as reported by many of the studies reviewed. The researchers noted that the observed speed reduction due to transverse pavement markings was not necessarily due to the perceptual illusion caused by the transverse pavement markings but that the effect could also have been due to transverse pavement markings acting as a warning device for drivers to reduce speed *(33)*.

A detailed study on perceptual counter measures to speeding was reported by Goldey of Monash University's Accident Research Center. The study, employing a driving simulator methodology, experimented with two sets of transverse bar marking patterns (Figure 12). The study concluded that both treatments were found to be effective in reducing travel speeds. Full-width transverse bars were reported to have resulted in up to a 6.8 mph speed reduction, and peripheral bars showed a 3.75 mph speed reduction. The results of this study also indicated that transverse bars with diminishing spacing resulted in greater speed reduction than transverse bars with constant spacing *(34)*.

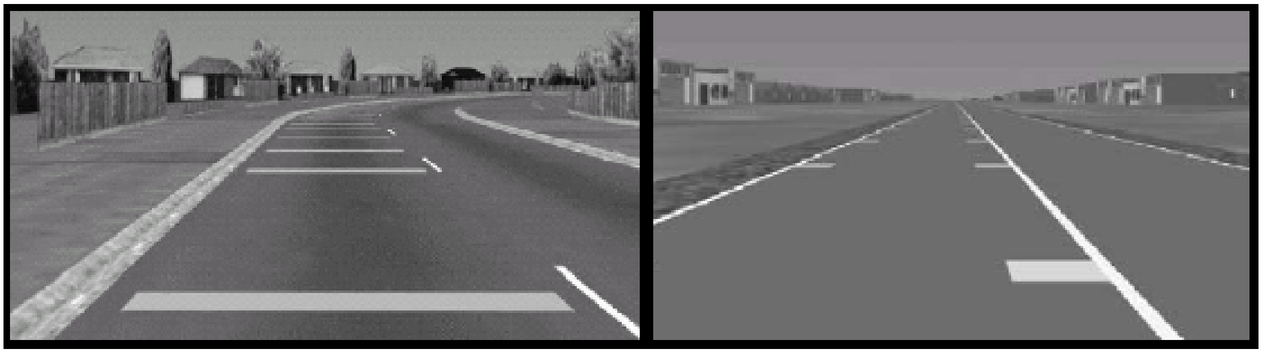


Figure 12. Transverse Bar Patterns as Simulated by Monash University Researchers *(28)*.

A study by Meyer, involving transverse bar pavement marking deployment at work zones in Kansas, was also reviewed. This study evaluated several transverse bar pavement marking pattern designs for work zone application. The study began with a driver simulation study to develop the pavement marking patterns deployed in the field. Based on the driver simulation results, transverse pavement markings were implemented at a test site on IH 70 west of Topeka, Kansas *(35)*. In the field implementation, three combinations of transverse bar pavement markings (as shown in Figure 13) were evaluated for their effectiveness in reducing speed. The first treatment was called a "leading" pattern, which consisted of 20 transverse bars of constant width and constant spacing intended to warn the drivers. The second pattern tested was called the "primary" pattern, where the transverse bars varied in width and spacing, giving an illusory perception of speeding to drivers. The third pattern was called the "work zone" pattern, where four sets of six bars were implemented for motorists with reduced speed to continue without accelerating in the work zone.

The study results showed that the transverse pattern implemented in this study had the potential to reduce speed. The results of this study indicated that both the warning and perceptual effects of transverse pavement markings influenced drivers to reduce their speeds.

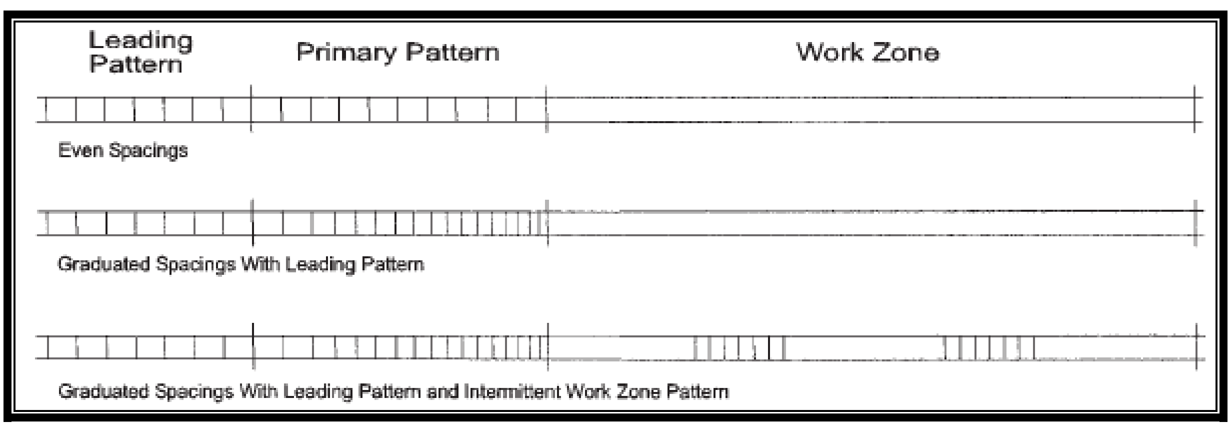


Figure 13. Transverse Bar Pattern Experimented in Kansas *(35)*.

Katz conducted detailed experiments with peripheral transverse pavement markings at three locations. Peripheral transverse bars were implemented at curves on IH 690 in Syracuse, New York; on Highway 468 in Flowood, Mississippi; and on Farm-to-Market (FM) Road 362 in Waller, Texas. At the first location on IH 690 in Syracuse, the treatment was applied on an exiting lane leading to a freeway-to-freeway connector ramp with a posted advisory speed of 30 mph on the ramp and a speed limit of 65 mph on the freeway. The second location on Highway 468 in Flowood consisted of a two-lane, bi-directional rural highway with a posted advisory speed of 40 mph at the treatment location and a speed limit of 45 mph at the tangent section. The third installation also was on a two-lane rural highway in Waller, Texas, with an advisory speed of 40 mph and a speed limit of 65 mph on the tangent section. This study evaluated the impact of the experimental markings on speed by vehicle classification and by vehicle headways. The results of the study concluded that the experimental peripheral transverse bars resulted in an overall reduction in speed at all three test sites. However, the magnitude of speed reduction varied among all three test sites with the least reduction in speed observed at Waller, Texas. The New York and Mississippi test sites showed that mean speeds decreased by 1.8 mph and 4 mph, respectively. Researchers noted that these results may indicate that these treatments, if implemented on interstates and rural roads with unfamiliar drivers, may have a more significant impact in reducing speed as compared to local roads. The author states that various factors resulted in the variability of speed reduction at different test sites, including familiarity of the drivers on the test sections, horizontal curvature, and contrast between experimental markings and pavement color *(36)*.

The Katz study also found that significant speed reduction was observed when data were analyzed by vehicle class (two axles or more than two axles). Researchers in this study also found that vehicles with headways four seconds or greater showed particularly significant reduction in speeds. A before-after speed analysis indicated that the Waller site was a bit different than the other two locations. Although at Waller no significant reduction in speed was observed where the treatment was applied, a significant reduction was observed upstream of the test section. This observation was justified by the researchers by stating that the presence of higher populations of local drivers using the test facility at Waller had influenced the speed reduction characteristics. Researchers stated that local drivers, who in the long run were aware of oncoming transverse markings, gradually started reducing their speed just ahead of the markings. As such, the transverse bars did act as a warning device for familiar drivers to reduce their speed rather than creating perceptual effect on motorists in reducing speed at the Waller site *(36)*.

Remarks on Transverse Bar Pavement Marking Studies

There have been several studies published on the use of transverse bar pavement markings to influence drivers to reduce speed. The evaluations found in the literature are representative of both United States and international experience. Most of these studies indicated a general effectiveness of transverse bar pavement markings in reducing speed on roadway segments, even though there has been debate on the mechanism of speed reduction with transverse bars (either through perceptive illusion or by providing a warning mechanism). The amount of speed reduction was noted to vary on different facilities under different conditions, but there was not a sufficient amount of research to define the exact influence of those stated conditions with respect to the treatment influence on speed reduction. The previous research generally concluded that, especially considering the low cost associated with transverse bar pavement marking implementations and minimal cost of maintenance, the transverse bar pavement marking treatment was a potentially useful tool for speed management.

Other Pavement Marking Treatments Worth Mentioning

Although transverse bar pavement marking and chevron pavement marking treatments were the predominant low-cost pavement marking tools researched for speed reduction, a few other pavement markings have been also studied. Very brief information on a few of those other studies is provided here for completeness.

Retting experimented with pavement marking treatments to determine if they had any potential influence on speed reduction. The first treatment involved installing the word "SLOW" with a left arrow mark 220 feet upstream of a sharp left-hand curve. Researchers found that this treatment resulted in a 3 percent reduction of speed from before to after at the site *(37)*. In a second treatment, researchers widened the gore area and moved the edge lines further into the lane to artificially reduce lane width at four freeway off-ramp locations in Virginia and New York. Though there was no significant difference in the overall mean speeds after the pavement marking treatment was deployed, it was found that there was a significant reduction in the percentage of vehicles exceeding the advisory speed limit *(38)*.

Another study reported on the test of an advanced curve warning pavement marking treatment installed by the Pennsylvania Department of Transportation. The treatment consisted of two transverse bars with the word "SLOW" and a curved arrow placed between the transverse bars. The report stated that the cost for implementation for this treatment was $1350, and an evaluation of the original pilot program of this treatment was expected to show a reduction of the 90th percentile speed, which was surmised to translate to 25 percent reduction in crashes *(39)*.

CHAPTER 4. PROJECT SITE CHARACTERISTICS

The results of the TxDOT transportation operations engineers survey, particularly the listing of problematic freeway connector curves (Appendix A, Question 2), was used to screen candidate sites for the converging chevron pavement marking installation.

PROJECT SITE SELECTION

Of those 57 sites, the researchers and the project monitoring committee concluded that one of the sites, a freeway-to-freeway connector ramp joining US 54 westbound to IH 10 westbound in El Paso, Texas, was an appropriate candidate for deployment of the converging chevron pavement marking treatment for evaluation. Figure 14 shows an aerial view of the connector curve location.

Texas Transportation Institute (TTI) staff visited the El Paso project site to assess curve conditions and applicability to the project. During the site visit, conditions at the freeway-to-freeway connector ramp were noted, including presence of skid marks or barrier hits (and location), signing and pavement markings present (and condition), and operational conditions (ball-bank readings and travel speed observations from several runs of each curve). Data collection was also completed at the potential test site consisting of classification, speed, and volume counts using automated tube counters for a period of three to five days.

The interchange of US 54 at IH 10 is located about three miles east of downtown El Paso. The freeway-to-freeway connector curve under study was "Ramp K," on the northwest corner of the interchange. Ramp K has a restricted horizontal geometry caused by right-of-way limited by a cemetery located directly northwest of the interchange. The freeway-to-freeway connector is also characterized by a vertical geometry featuring an approach on an upgrade, which provides limited sight distance to the most geometrically-limiting portion of the connector. It is the combination of tight horizontal curvature, limited sight distance due to vertical curvature, and long, straight upstream approach conditions that can be problematic for some drivers. The connector consists of two 11-foot wide lanes with 9- to 10-foot shoulders on the right and about 4-foot shoulders on the left of the connector curve.

The average weekday daily traffic volume on the freeway-to-freeway connector curve was about 18,000 vehicles per day during the data collection periods, with about 2 percent trucks. The posted curve advisory speed was 30 miles per hour, and advisory speed signs with flashing beacons were present both before and after the test markings were installed. The connector curve radius was approximately 500 feet, and the curve was about 600 feet in length.

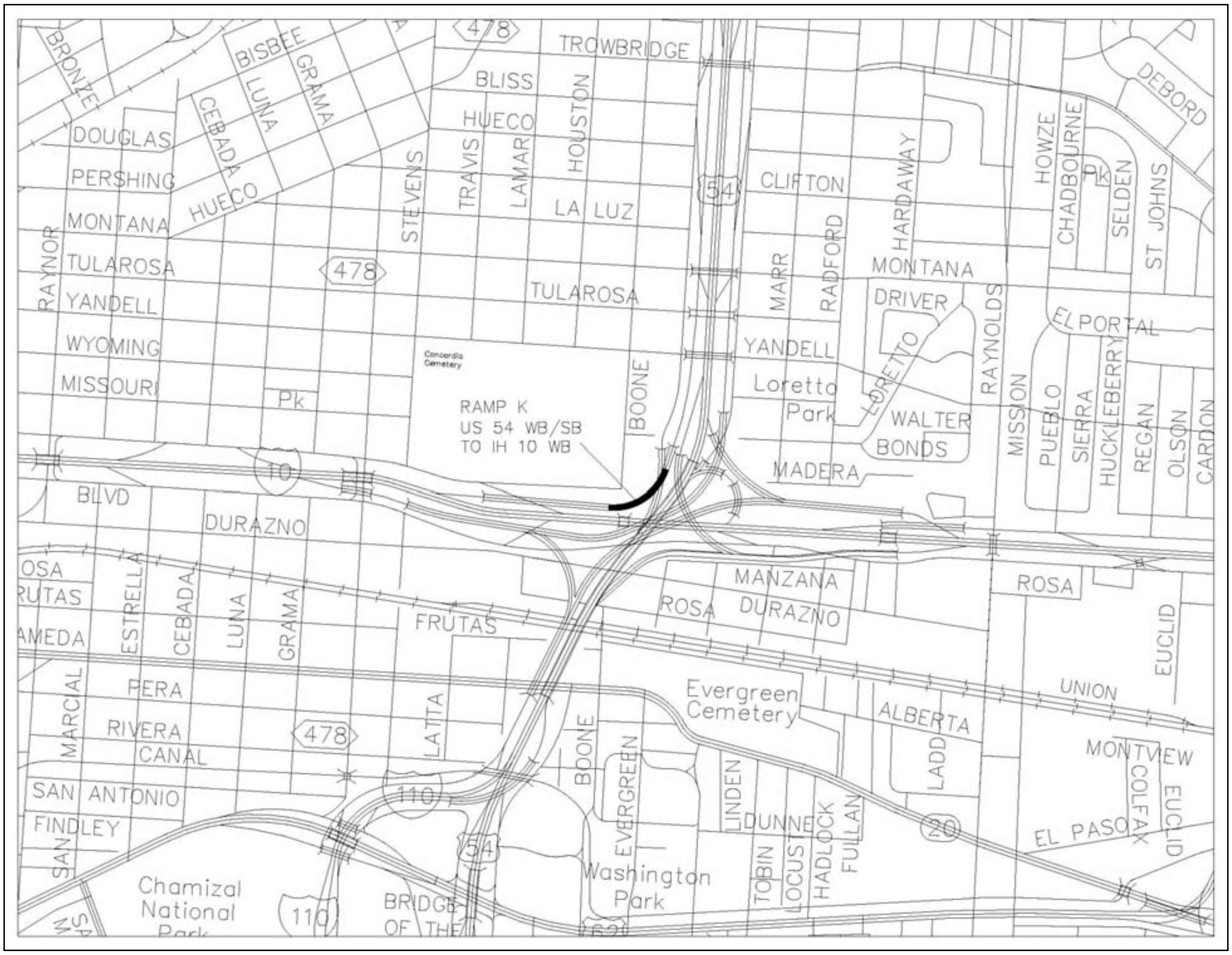


Figure 14. US 54 at IH 10, El Paso, Texas - Project Location.

CONVERGING CHEVRON PAVEMENT MARKING PATTERN DETAILS

The converging chevron pavement marking pattern used for this project was similar to the pattern used in the Milwaukee study, with some modification in the design pertaining to the chevron width and number of markings per set of chevrons. The chevron marking scheme evaluated for this project was a combination of chevrons and longitudinal skip (or comb) markings.

The initial spacing between sets of the converging chevron pavement marking patterns was determined using essentially the same methodology as the Milwaukee trial. This methodology uses the upstream, free-flow speed and the posted advisory speed for the connector curve. However, the pavement marking layout was modified from the Milwaukee methodology in order to simplify some of the chevron spacing and reduce the number of sets of chevrons. This was done primarily to reduce the installation cost, which was a deployment constraint. The spacing between sets was modified per Table 4. Field changes during pavement marking installation required further changes in the marking layout, as compared to the initial design shown in Figure 15.

Figure 16 shows the specific design of each set of chevron patterns. Figures 17 to 20 show some of the pictures captured at the project site after the installation of chevrons. As compared to the Milwaukee markings, the El Paso markings consisted of five individual chevrons per set instead of eight. There was a set of horizontal signing pavement markings placed at about 900 feet in advance of the start of the freeway connector curve stating "CURVE AHEAD 30 MPH." These advisory markings were eliminated as part of the deployment of the converging chevron pavement marking pattern.

Table 4. Spacing Between Chevron Sets.

Set Number	Spacing (ft)	Set Number	Spacing (ft)
1		19	40
2	25	20	40
3	25	21	45
4	25	22	45
5	25	23	45
6	30	24	50
7	30	25	50
8	30	26	50
9	30	27	55
10	30	28	55
11	30	29	55
12	35	30	60
13	35	31	60
14	35	32	60
15	35	33	60
16	35	34	65
17	35	35	65
18	40	36	65

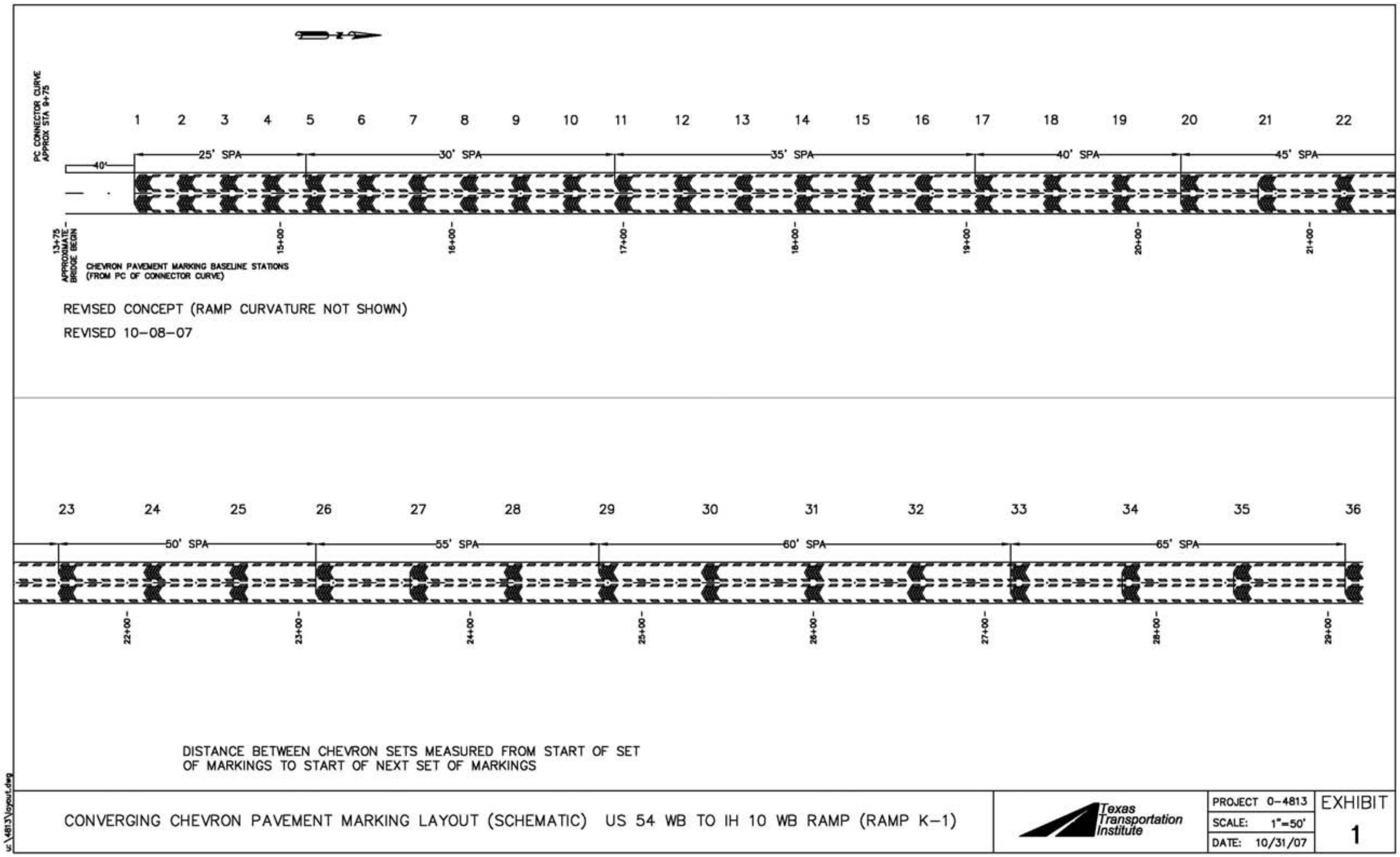


Figure 15. Overview of Chevron Marking Pattern Designed for the Project.

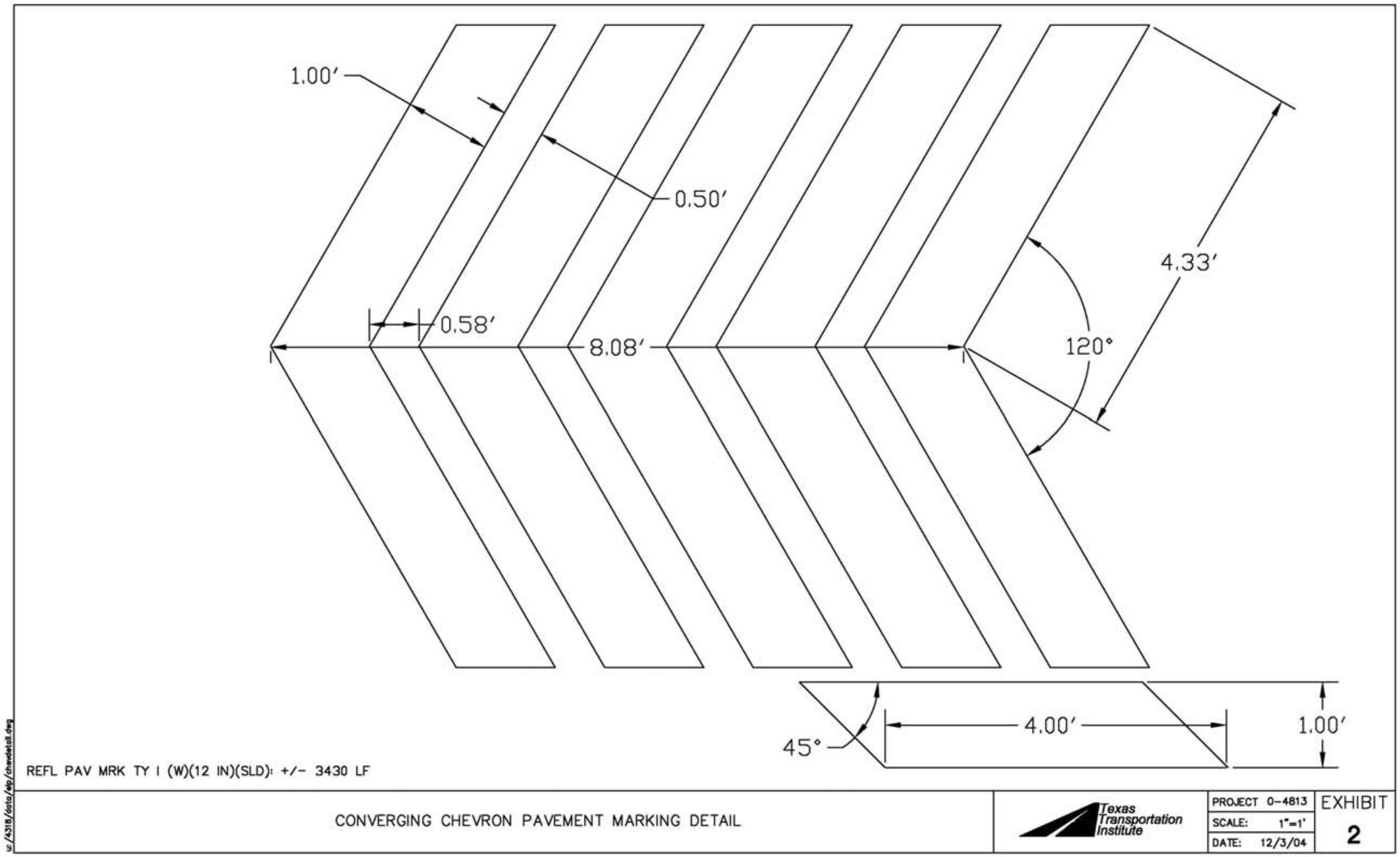


Figure 16. Design Specifications of Chevron Markings.

Figure 17. View of Chevron Pavement Markings Implemented on the Approach to the US 54 Westbound to IH 10 Westbound Freeway-to-Freeway Connector Curve.

Figure 18. Closer View of Chevron Markings Implemented at Site.

Figure 19. Chevron Markings Ending Near the Beginning of the Connector Curve.

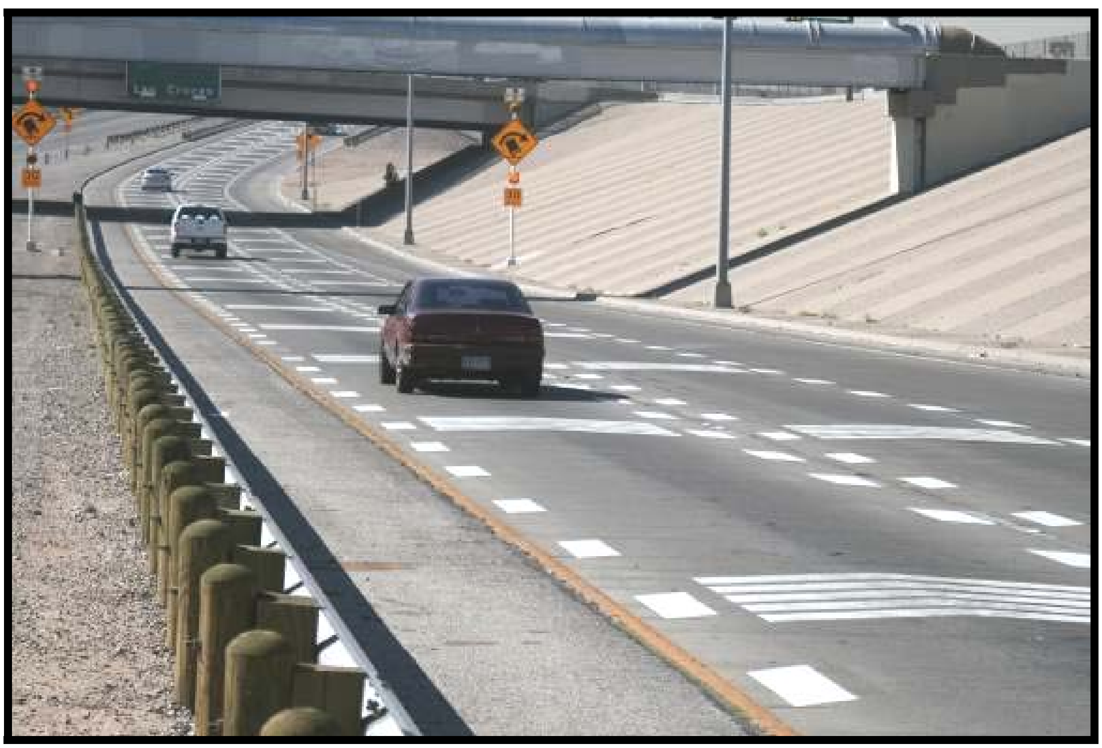


Figure 20. View of Chevron Markings with Existing Flashing Advisory Speed Warning Sign (30 mph).

CHAPTER 5. DATA COLLECTION

Automatic data collection devices were deployed at the project location to measure traffic volume, individual vehicle speeds, and vehicle classification at four locations on the study curve:

- far upstream of the connector ramp,
- upstream of the connector ramp,
- at the beginning of the connector curve, and
- at the midpoint of the connector curve.

The duration of each data collection period was typically three to five days.

Figure 21 shows the locations of the data collection points on the curve. In the early-after and late-after data collection periods, data was collected at an additional location on the approach farther upstream from the before data upstream location to adequately gather free flow speeds in advance of the pavement marking treatment.

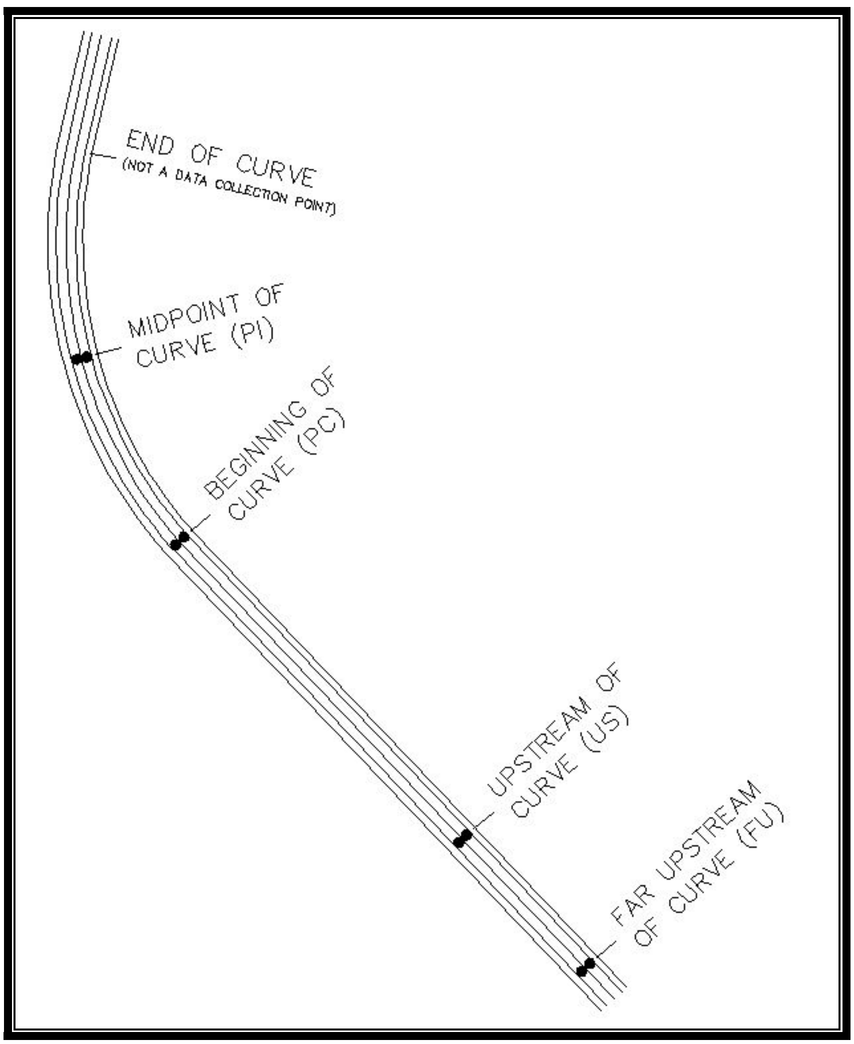


Figure 21. Schematic of Data Collection Location on the Curve.

Researchers sought to obtain volume, speed, and vehicle classification measurements at multiple locations on each connector ramp to measure any operational impacts of the converging chevron pavement markings. Researchers collected the volume, speed, and classification data using TimeMark Delta IIIB portable road tube classifiers. These automatic pneumatic tube classifiers are designed for use on multiple-lane high-volume roadways. For a two-lane connector ramp, the equipment deployment process required installing one set of road tubes across both lanes of the connector ramp spaced exactly 16 feet apart and a second set of road tubes across a single lane of traffic (Figure 22). For the purpose of consistency, researchers separated each of the two sets by 18 inches for each data collection setup.

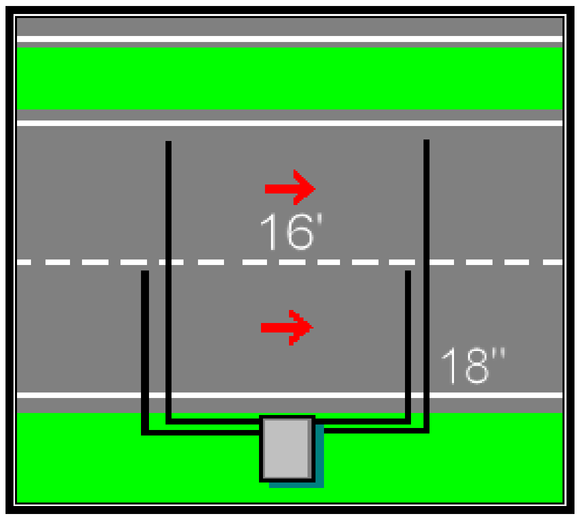


Figure 22. Automatic Traffic Counting Equipment Deployment Layout.

Researchers programmed the classifiers/counters to provide volume, classification, speed, and gap data for each lane of the connector ramps. While most portable traffic data collection equipment is only capable of providing speed and classification data in summarized totals, the TimeMark Delta IIIB units provide a time-stamped "per vehicle" output. This output includes:

- date/time of day,
- total number of axles,
- spacing between each of the axles,
- estimated spot speed of the vehicle, and
- gap between vehicles.

The TimeMark Delta IIIB also assigns a vehicle classification based upon the number of vehicle axles and axle spacing. Table 5 identifies these FHWA classifications used by the TimeMark software.

Table 5. Delta IIIB Counters: Vehicle Classification Table.

Classification Number	Vehicle Description
1	Motorcycle
2	Car (also with 1- or 2-axle trailer)
3	Light roads vehicle (also with 1-, 2-, or 3-axle trailer)
4	2- or 3-axle bus
5	2-axle rigid (heavy goods vehicle) truck
6	3-axle rigid (heavy goods vehicle) truck
7	4- or more axle rigid (heavy goods vehicle) truck
8	Tractor trailer, 3 or 4 axles
9	Tractor trailer, 5 axles
10	Tractor trailer, 6 axles
11	Multi-trailer truck, 5 axles or less
12	Multi-trailer truck, 6 axles
13	Multi-trailer truck, 7 or more axles

TTI collected data for about four days at each of the data collection points on the connector ramp during each of the data collection periods. The converging chevron pavement markings were installed February 15-17, 2007. Before data was collected in October of 2006, early-after data was collected in April/May of 2007, and late-after data was collected in August of 2007.

CHAPTER 6. EFFECTIVENESS OF CONVERGING CHEVRON PAVEMENT MARKINGS

In order to determine the operational impacts of the converging chevron pavement marking installation, researchers formulated several data analysis procedures. This chapter presents the analysis efforts conducted for this project. The objectives of the analyses conducted in this project were to address the following hypotheses:

- Is there a significant reduction in overall mean speed on the approach to the freeway-to-freeway curve as well as on different points on the curve?
- Is there a significant reduction in mean speeds for different vehicle classes?
- Do daylight and non-daylight conditions have an impact on the effectiveness of chevron pavement markings in reducing speeds?

To quantitatively assess the installation of the converging chevron pavement markings with respect to speed reduction on the freeway-to-freeway connector curve, the following analyses were completed:

- calculation and analysis of the general speed statistics for curve by curve location, vehicle class, and study period;
- ANOVA for mean speeds with study period, vehicle type, curve location, peak or non-peak as factors; and
- before-after comparison of the mean speed on the project curve by study period, curve location, and vehicle class.

To facilitate the analysis in determining the impact of the converging chevron pavement markings, researchers collected speeds in three study periods: "before" the installation of the chevron pavement markings, "early-after" (about one to three months after chevron installation), and "late-after" (about four to six months after chevron installation.) Researchers collected automatic tube data at several locations on the project curve during all periods of data collection: upstream of curve, beginning of curve, and at the middle of curve. Chapter 5 described the details of the data collection.

VOLUME/SPEED/CLASS DATASET PROCESSING AND QUALITY CONTROL

Prior to the use of data in any analysis, researchers reviewed the speed data collected via the automatic tubes for accuracy and quality control. Quality control consisted of a review of hourly count data and checks for consistency during the AM and PM peak periods to identify any irregularities potentially due to incidents or congestion. Since the study freeway-to-freeway connector is a two-lane facility and data were collected on both lanes individually, quality checks were also performed to ensure consistency between the two lanes of traffic.

Data were also checked for outliers, which were free-flowing vehicles traveling at either excessive speeds or very low speeds, even though headways were greater than five seconds. Data deemed unreliable or potentially caused by measurement error were trimmed from the dataset.

TTI personnel also reviewed the amount of data collected, setting a requirement of at least 125 heavy vehicle measurements to be included in the project site and curve location for meaningful analysis.

Segmentation of Free Flowing Vehicles

It was necessary to isolate those vehicles that were assumed to be under no outside influence in their choice of speed as they approached and traversed the project curve. It was then necessary to define a free-flow condition for vehicles approaching and traversing the connector ramp. The Highway Capacity Manual (HCM) defines free flow speeds as "traffic unaffected by upstream or downstream conditions" *(40)*. Under the assumption that the freeway-to-freeway connector ramp geometry influences the free-flow speed, researchers filtered data so that individual speed measurements reflect only those assumed affected by ramp geometry and not influenced by other vehicles, incidents, or congestion. To accomplish this, all datasets were filtered by the headway (in seconds) between vehicles. Only vehicles with headway of more than five seconds were considered for the analysis.

Vehicle Classification

Using Statistical Analysis Software (SAS), researchers combined vehicles that had been automatically categorized by the TimeMark counter software into the 13 FHWA vehicle classification groups. Table 6 summarizes how three categories of vehicles were grouped for analysis. Researchers grouped several FWHA vehicle classifications into three distinct groups in order to facilitate a comparison among vehicle types with similar operating characteristics (passenger vehicles, rigid vehicles, and heavy trucks).

Table 6. Vehicle Groups for Comparison among Types.

Group	FHWA Classification	Description
Passenger Vehicles	1, 2, and 3	Passenger Cars, Light Trucks, Motorcycles
Rigid Vehicles	4, 5, 6, and 7	Larger vehicles between 2 to 4 axles that did not have a detachable trailer for transporting goods
Heavy Vehicles	8, 9, 10, 11, 12, and 13	Various configurations of tractor-trailer combinations

DESCRIPTIVE STATISTICAL ANALYSIS

A descriptive statistical analysis was performed on the datasets, by time-period of data collection, by location on the curve, by vehicle class, and by daylight conditions. The purpose of the descriptive analysis was to develop an indication about the trend of speeds and the quality of data. The initial step of this analysis was to generate frequency plots of the speed data for various groups, such as curve location, vehicle class, time-period, and daylight conditions. These frequency plots were examined for normality of data, and cross-verified with kurtosis value for the speed distribution. In cases where speed distributions were not normal, the tail values of the distribution were examined for their reasonableness. Speeds that appeared to be outliers were deleted from the distribution, and normality assumptions were verified. An objective measure to determine normality was to ensure that the kurtosis score for the distributions lie within +2 and -2. In some cases, a range of kurtosis score of +3 to -3 was accepted when all the speeds in the distribution were reasonable and if no further trimming of tails of distribution was justifiable based on engineering judgment. The next step in the analysis involved calculation of mean speeds and 85th percentile speeds for the datasets under all the subgroups for analysis. Table 7 summarizes the number of vehicles sampled, calculated mean speed and 85th percentile speed for several possible combinations of categories. The header row indicates three time periods of data collection (before, early-after, and late-after). The columns contain various categories by vehicle class for measurements taken at four different locations of the curve (far upstream, upstream, start, and middle of curve). Table 8 summarizes the mean and 85th percentile speeds by lighting condition.

Speed differentials between different points of the curve were calculated from the mean speed and 85th percentile speeds derived for descriptive statistics for a given study period by vehicle class. Speed differentials computed were the following:

- Speed differential between upstream and start of curve,
- Speed differential between upstream and middle of curve, and
- Speed differential between start of curve and middle of curve.

Tables 9 and 10, respectively, present the results of the speed differential by vehicle type and by daylight conditions.

From Table 7, in all time periods except the late-after period, mean speeds and 85th percentile speeds decreased for the 'All Vehicle' category as the vehicles traversed from upstream of the curve to start of the curve, and from start of the curve to the mid of the curve. In the late-after period, though, there was a decrease in mean speed as vehicles traversed from upstream to start of curve and a slight increase in mean speed as vehicles traversed from start of curve to middle of curve. When segregated by vehicle type, in all periods, rigid vehicles showed a trend of increasing speeds as they traversed from start of the curve to middle of the curve; however, the increase in mean speed was higher for the late-after period. It should also be noted that at the upstream location, when mean speeds among each vehicle type are compared at different time periods, late-after period has the lowest mean speeds. This indicates that motorists do reduce their speeds well before the curve and significantly on the curve approach, as soon they see the chevron pavement markings.

Comparison of the mean speed differential from Table 9 indicates that maximum speed differential occurs between upstream and start of the curve, about 13 to 16 mph, while only a slight reduction of less than 1 mph is observed between start to mid of the curve for all vehicle classes. However, by segregating by vehicle class, reduction in mean speed from upstream to start of curve is higher for heavy vehicles compared to passenger vehicles. This indicates that heavy vehicles experience higher deceleration than passenger cars before entering the curved section. However, the mean speed differential between start and mid of the curve indicates that heavy and rigid vehicles increase their speeds in the early-after periods, and increase in speed among all vehicle categories in the late-after period. This increase in speed from start of the curve to mid of the curve is notably higher for heavy vehicles (about 3.5 mph), whereas passenger vehicles increase their mean speed by about 0.85 mph in the late-after period.

Table 7. Speed Summary by Vehicle Type Classification.

Measured at	Vehicle Category	Before (10/16/2006 – 10/20/2006)			Early-After (4/30/2007 – 5/1/2007)			Late-After (8/3/2007 – 8/9/2007)		
		No. of Speed Observations	Mean Speed	85th %tile Speed	No. of Speed Observations	Mean Speed	85th %tile Speed	No. of Speed Observations	Mean Speed	85th %tile Speed
Far Upstream of Curve	Heavy Vehicles				99	60.73	66.50	470	59.61	65.60
	Passenger Vehicles				4443	66.55	72.30	25,302	63.97	69.90
	Rigid Vehicles				534	65.65	71.90	1989	64.63	71.20
	All Vehicles				5076	66.34	72.30	27761	63.94	69.90
Upstream of Curve	Heavy Vehicles	703	60.19	64.80	224	61.00	67.10	667	55.92	61.50
	Passenger Vehicles	28,283	63.35	68.60	6842	63.31	68.30	43,078	60.06	66.80
	Rigid Vehicles	1220	60.83	66.20	289	61.48	66.80	1610	59.63	68.60
	All Vehicles	30,206	63.17	68.30	7355	63.17	68.30	45,355	59.98	66.80
Start of Curve	Heavy Vehicles	507	40.45	46.00	148	39.38	44.70	834	39.26	44.50
	Passenger Vehicles	28,512	48.31	53.60	7026	47.89	53.00	44,698	47.56	52.60
	Rigid Vehicles	1195	44.64	50.80	297	44.36	50.20	1815	43.75	50.20
	All Vehicles	30,214	48.03	53.40	7471	47.58	52.80	47,347	47.26	52.60
Middle of Curve	Heavy Vehicles	537	39.55	43.90	142	39.90	43.70	941	42.91	48.80
	Passenger Vehicles	25,756	47.46	52.30	6457	47.35	51.90	36,777	48.39	53.40
	Rigid Vehicles	2459	46.39	52.30	460	45.57	50.80	4010	47.71	53.20
	All Vehicles	28,752	47.22	52.10	7059	47.08	51.70	41,728	48.20	53.20

Table 8. Speed Summary by Daylight Condition.

Measured at	Daylight Category	Before (10/16/2006 – 10/20/2006)			Early-After (4/30/2007 – 5/1/2007)			Late-After (8/3/2007 – 8/9/2007)		
		No. of Speed Observations	**Mean Speed**	**85th %tile Speed**	**No. of Speed Observations**	**Mean Speed**	**85th %tile Speed**	**No. of Speed Observations**	**Mean Speed**	**85th %tile Speed**
Far Upstream of Curve	Daytime				2544	66.07	71.60	13,801	63.99	69.60
	Nighttime				976	65.62	72.30	6066	62.55	69.00
Upstream of Curve	Daytime	13,465	63.70	68.60	4072	63.49	68.30	26,283	60.50	67.10
	Nighttime	5621	62.65	68.30	1286	62.03	67.70	8252	57.96	64.80
Start of Curve	Daytime	14,115	48.50	53.70	4304	47.85	53.00	27,248	47.72	52.80
	Nighttime	4803	47.46	53.20	1147	46.46	52.10	8771	45.74	51.50
Middle of Curve	Daytime	13,327	47.54	52.30	4094	47.09	51.70	22,826	48.46	53.40
	Nighttime	4556	46.96	52.30	1086	46.54	51.50	8327	47.21	52.80

Table 9. Speed Differentials Between Different Points of the Curve by Vehicle Type.

Speed Difference at	Vehicle Type Category	Before (10/16/2006 – 10/20/2006)		Early-After (4/30/2007 – 5/1/2007)		Late-After (8/3/2007 – 8/9/2007)	
		Difference in Mean Speed	Difference in 85th %tile Speed	Difference in Mean Speed	Difference in 85th %tile Speed	Difference in Mean Speed	Difference in 85th %tile Speed
Upstream of Curve to Middle of Curve	Heavy Vehicles	-20.64	-20.90	-21.10	-23.40	-13.01	-12.70
	Passenger Vehicles	-15.89	-16.30	-15.96	-16.40	-11.67	-13.40
	Rigid Vehicles	-14.44	-13.90	-15.91	-16.00	-11.92	-15.40
	All Vehicles	-15.95	-16.20	-16.09	-16.60	-11.78	-13.60
Start of Curve to Middle of Curve	Heavy Vehicles	-0.90	-2.10	0.52	-1.00	3.65	4.30
	Passenger Vehicles	-0.85	-1.30	-0.54	-1.10	0.83	0.80
	Rigid Vehicles	1.75	1.50	1.21	0.60	3.96	3.00
	All Vehicles	-0.81	-1.30	-0.50	-1.10	0.94	0.60
Upstream of Curve to Start of Curve	Heavy Vehicles	-19.74	-18.80	-21.62	-22.40	-16.66	-17.00
	Passenger Vehicles	-15.04	-15.00	-15.42	-15.30	-12.50	-14.20
	Rigid Vehicles	-16.19	-15.40	-17.12	-16.60	-15.88	-18.40
	All Vehicles	-15.14	-14.90	-15.59	-15.50	-12.72	-14.20

Table 10. Speed Differentials Between Different Points of the Curve by Daylight Condition.

Speed Difference at	Daylight Category	Before (10/16/2006 – 10/20/2006)		Early-After (4/30/2007 – 5/1/2007)		Late-After (8/3/2007 – 8/9/2007)	
		Difference in Mean Speed	Difference in 85th %tile Speed	Difference in Mean Speed	Difference in 85th %tile Speed	Difference in Mean Speed	Difference in 85th %tile Speed
Upstream of Curve to Middle of Curve	Daytime	-16.16	-16.30	-16.40	-16.60	-12.04	-13.70
	Nighttime	-15.69	-16.00	-15.49	-16.20	-10.75	-12.00
Start of Curve to Middle of Curve	Daytime	-0.96	-1.40	-0.76	-1.30	0.74	0.60
	Nighttime	-0.50	-0.90	0.08	-0.60	1.47	1.30
Upstream of Curve to Start of Curve	Daytime	-15.20	-14.90	-15.64	-15.30	-12.78	-14.30
	Nighttime	-15.19	-15.10	-15.57	-15.60	-12.22	-13.30

ANALYSIS OF VARIANCE

A standard ANOVA test was completed using the SAS PROC GLM (generalized linear model) procedure. ANOVA analysis was completed on the mean speed. The ANOVA procedure requires that data be random, normally distributed, have a common variance, and have equal population means.

For the purposes of this report, mean speeds fit the underlying data requirement by the definition of the central limit theorem (CLT). The CLT essentially states that with a significant amount of data observation the data naturally conform to the shape of a normal distribution curve. In addition, researchers verified several sets of data graphically in order to ensure that the data used for this project follow an approximate normal distribution. Researchers compared relationships between study period, location on curve, and vehicle class. Researchers also compared a combination of factors such as study period, curve location, vehicle class, and traffic condition (peak or non-peak).

The ANOVA procedure is used to statistically determine if the sample means for varying subsets of data differ from each other. For the purposes of this report, researchers sought to determine whether or not the sample mean speed measurements differ between curve locations sampled, between study periods, and between vehicle class groups. Using the ANOVA, it is also possible to combine these parameters and determine if the combinations are different. For instance, the question could be asked: *Are the sample means for heavy vehicles from the before period at the middle of the curve different from the sample mean for heavy vehicles from the after period at the start of curve?* If the sample means are identified as statistically different, inferences about the sample population can be drawn *(41)*.

The null hypothesis for this project states that all population mean speeds are equal for all conditions. Equation 6 is an example of the null hypothesis. The hypothesis would vary based on the conditions being measured. The example below is for vehicle class. Similar hypotheses could be stated for other factors like study period, curve location, etc.

$$H_o = \mu_{Heavy\ Vehicles} = \mu_{Passenger\ Vehicles} = \mu_{Rigid\ Vehicles} \quad \text{(Eq. 6)}$$

where:

μ_n is the population mean speed for condition "n."

The alternate hypothesis (H_A) is that at least one of the population means differs from the others. The probability of a Type III error (or α) is equal to 0.05 for all tests of significance. Type III sum of squares values were considered for this analysis. Type III values were used instead of Type I because Type I considers variables in sequence. Type III considers each variable in the presence of other independent variables in the model *(42)*.

The test statistic used to determine the equality of means is the F statistic. The F test calculates the ratio between sample variance and within sample variance. Sample variance has also been described as the sum of squares value divided by its degrees of freedom. This value has also been called a mean square.

Whether or not the null hypothesis is rejected is based on the calculated F statistic. Significance is based on whether the calculated F statistic is above or below a tabular value based on the degrees of freedom between samples, the degrees of freedom within samples, and the probability, $\alpha = 0.05$. Conveniently, SAS generates a probability value, Pr > F. This value directly corresponds to the probability, $\alpha = 0.05$. If the Pr > F value is less than $\alpha = 0.05$, then the null hypothesis can be rejected, and the mean speeds are significantly different for a given category.

Table 11 presents the results of the ANOVA conducted for this project. The first column of the table provides some of the important factors that are of interest for this project. The following columns in the table provide the statistics for the analysis. Since all the source factors have "Pr > F" less than α (0.05), it can be inferred that mean speed for categories in each factor are statistically significantly different. For example, referring to the source "Period," it can be inferred that mean speeds between before, early-after, and late-after are significantly different. Hence, it can also be inferred that the treatment (chevron installation) has made some impact on the mean speeds. The F value corresponding to the source factors indicates the degree of influence of each of those factors on mean speeds. That is, location on curve (upstream, or start of curve or middle of curve) would be the most significant indicator of a difference in speed. Vehicle class will be the next biggest influence on speed, followed by time period (before, early-after, and late-after) and peak period (peak or non-peak). Appendix C presents the detailed ANOVA statistics.

Table 11. ANOVA Results for El Paso Speed Data.

Source	Degrees of Freedom	Type III Sum of Square	Mean Square	F Value	Pr > F
PEAK	1	1145.33	1145.33	37.39	<.0001
PERIOD	2	2521.37	1260.68	41.15	<.0001
CURVE	3	460,915.4	153,638.47	5015.13	<.0001
VEH_CLASS	2	42,071.53	21,035.77	686.66	<.0001

BEFORE-AFTER COMPARISON OF MEAN SPEEDS

In the previous section, ANOVA was conducted to determine the factors that influence the mean speeds at the project site. In this analysis, the study period was indicated to have an influence on the mean speeds according to the results from ANOVA. In this section, detailed before-after analyses were conducted to quantify the amount of increase or decrease in mean speeds between study periods. Study periods that were compared are before and early-after, before and late-after, early-after, and late-after. Before-after analysis was conducted between study periods by curve location and vehicle class. A statistical procedure called large-sample Z test was adopted in the before-after analyses.

The null hypothesis for this test states that there is no difference in the mean speeds of the two populations being compared. Or:

$$H_0: \mu_{before} - \mu_{after} = 0 \qquad \text{(Eq. 7)}$$

A 'Z' statistic is computed given the sample means and sample variances as shown in Equation 8 *(43)*.

$$Z_0 = \frac{X_{before} - X_{after}}{\sqrt{\frac{S_{before}}{N_{before}} + \frac{S_{after}}{N_{after}}}} \qquad \text{(Eq. 8)}$$

where:

X_{before}	: Mean speed of before sample
X_{after}	: Mean speed of after sample
S_{before}	: Sample variance for data collected in before period
S_{after}	: Sample variance for data collected in after period
N_{before}	: Sample size for data collected in before period
N_{after}	: Sample size for data collected in after period

The rejection criteria for the null hypothesis is $Z_0 > Z_{\alpha/2}$ or $Z_0 < - Z_{\alpha/2}$. For this project, type I error was taken as 0.05. This means that decision on the null hypothesis will be made with 95 percent confidence. Comparisons in mean speed were carried out at different locations on the curve and by vehicle class. Tables 12 and 13, respectively, summarize the results of this analysis by vehicle type and daylight conditions.

Differences in the mean speeds are presented in three columns: the first column presents the difference in mean speeds between the before period and early-after period, the second column presents the difference in speeds between the before period and late-after period, and the third column presents the difference in mean speeds between early-after and late-after periods.

As seen in Table 12, the speed comparisons between time-periods have been computed at different points on the curve by vehicle category. Table 13 provides speed comparisons between time periods at different points on the curve by daylight conditions. A negative value for the difference in speeds indicate that decrease in speeds from later time period to earlier time period, and a positive value indicated increase in speeds. The symbols "▼and ▲" adjacent to the difference values indicates that the difference in speed was statistically significant as per the "large-sample" Z test at a 95 percent confidence level.

The before and early-after comparison for all vehicles indicates that chevron markings do have influence in reducing speeds significantly at the start and middle of the curve. Among individual vehicle categories, mean speeds from the before to early-after period decreased for heavy vehicles, passenger vehicles, and rigid vehicles at the start of the curve. However, heavy vehicle speeds increased slightly at the middle of the curve from the before to early-after period. The rest of the vehicle categories had a decrease in mean speeds from the before to early-after period at the middle of the curve.

A comparison of the before and late-after periods shows that there is a significant reduction in speeds at the upstream location and at the start of the curve for all vehicle categories, but a significant increase in speeds was observed at the middle of the curve for all vehicle categories. Comparing the relative decrease (at upstream and start location) or increase (at middle of curve) among different vehicle categories, it can be seen that heavy vehicles are more influenced by installation of chevron markings than other vehicle categories. In the before and late-after comparison it should also be noted that the upstream location had a greater reduction in mean speeds than at the start of the curve. This could be an indication that motorists consider the presence of chevron markings as a warning to reduce their speeds, and hence greater speed reductions were observed at upstream location of the curve where markings began.

From Table 13, it can be seen that daytime and nighttime conditions have similar trends when comparing before and early-after or before and late-after at different locations of the curve. However, the reductions in speed are slightly more during nighttime periods in all cases except for the before and early-after comparison at the middle of curve.

Table 12. Speed Difference in Before and After Chevron Installation by Vehicle Type.

Measured at	Vehicle Type Category	Difference in Mean Speeds Between Study Periods: Early-After and Before	Late-After and Before	Late-After and Early-After
Far Upstream of Curve	Heavy Vehicles	*	-0.58	-1.12
	Passenger Vehicles	*	▲ **0.62**	▼ **-2.58**
	Rigid Vehicles	*	▲ **3.80**	▼ **-1.02**
	All Vehicles	*	▲ **0.77**	▼ **-2.40**
Upstream of Curve	Heavy Vehicles	0.81	▼ **-4.27**	▼ **-5.08**
	Passenger Vehicles	-0.04	▼ **-3.29**	▼ **-3.25**
	Rigid Vehicles	0.65	▼ **-1.20**	▼ **-1.85**
	All Vehicles	0.00	▼ **-3.19**	▼ **-3.19**
Start of Curve	Heavy Vehicles	▼ **-1.07**	▼ **-1.19**	-0.12
	Passenger Vehicles	▼ **-0.42**	▼ **-0.75**	▼ **-0.33**
	Rigid Vehicles	-0.28	▼ **-0.89**	-0.61
	All Vehicles	▼ **-0.45**	▼ **-0.77**	▼ **-0.32**
Middle of Curve	Heavy Vehicles	0.35	▲ **3.36**	▲ **3.01**
	Passenger Vehicles	-0.11	▲ **0.93**	▲ **1.04**
	Rigid Vehicles	▼ **-0.82**	▲ **1.32**	▲ **2.14**
	All Vehicles	▼ **-0.14**	▲ **0.98**	▲ **1.12**

* Speed data not collected at far upstream of curve during before period

Table 13. Speed Difference in Before and After Chevron Installation by Daylight Condition.

Measured at	Daylight Category	Difference in Mean Speeds Between Study Periods: Early-After and Before	Late-After and Before	Late-After and Early-After
Far Upstream of Curve	Daytime	*	▲ **0.29**	▼ **-2.08**
	Nighttime	*	-0.10	▼ **-3.07**
Upstream of Curve	Daytime	▼ **-0.21**	▼ **-3.20**	▼ **-2.99**
	Nighttime	▼ **-0.62**	▼ **-4.69**	▼ **-4.07**
Start of Curve	Daytime	▼ **-0.65**	▼ **-0.78**	-0.13
	Nighttime	▼ **-1.00**	▼ **-1.72**	▼ **-0.72**
Middle of Curve	Daytime	▼ **-0.45**	▲ **0.92**	▲ **1.37**
	Nighttime	▼ **-0.42**	▲ **0.25**	▲ **0.67**

* Speed data not collected at far upstream of curve during before period

DRIVER COMPLIANCE MONITORING

In this project, researchers also performed a comparison of motorist noncompliance to the posted advisory speed before and after the chevron markings were installed. Researchers generated noncompliance percentages by study period and vehicle type.

Another measure examined in the research project to evaluate the effectiveness of chevron pavement marking was to see what percent of motorists traveling on the freeway-to-freeway connector ramp were not complying with the posted advisory speed limits. The researchers also examined whether the noncompliance percentage reduced with the installation of chevrons.

The posted advisory speed on the project ramp was 30 mph. The evaluation of advisory speed noncompliance was conducted on speed data collected at start of the curve and middle of the curve. Figure 24 shows the noncompliance percentage for all vehicles at the start of the curve and middle of the curve for all study periods (before, early-after and late-after periods). Noncompliance percentage was found to be very high, indicating that very few motorists are traveling at or below the posted advisory speed. However, in the late-after period, noncompliance with the posted advisory speed decreased by about 2 percent when compared to the before periods at the same location on the curve.

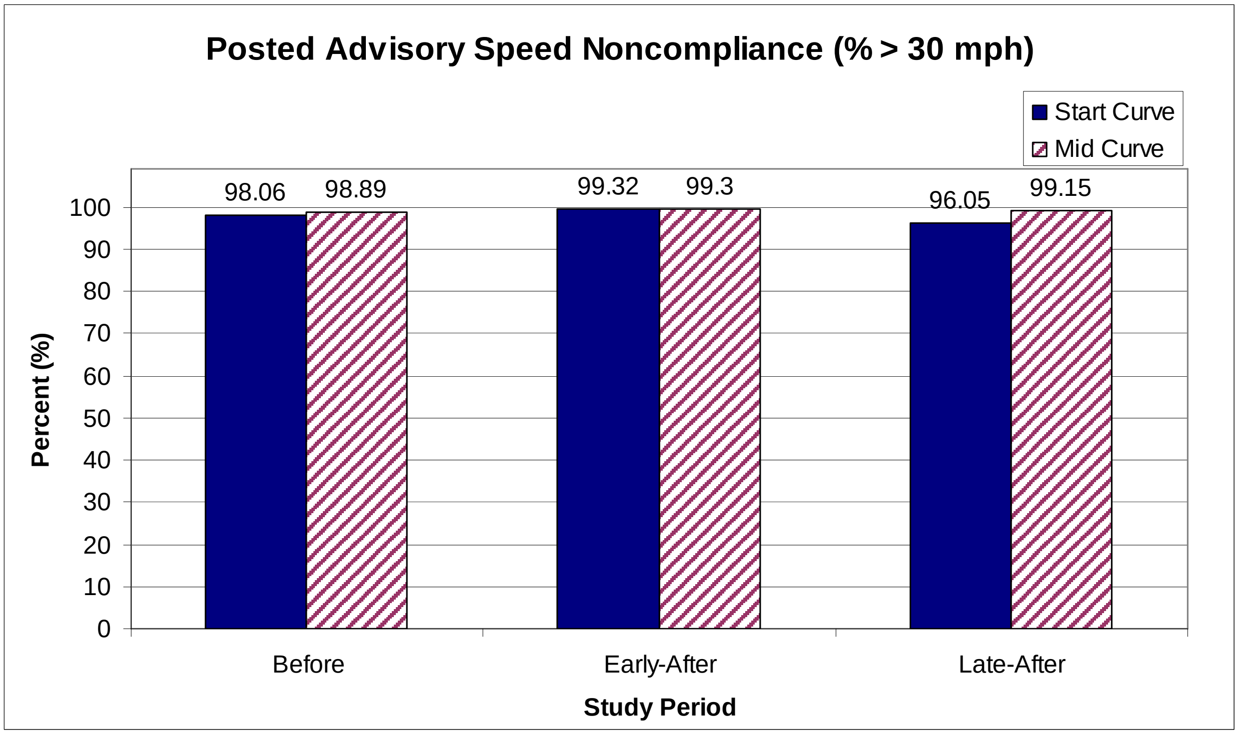


Figure 24. Posted Advisory Speed (30 mph) Noncompliance.

In order to get a better picture of what might be considered "critical" noncompliance, researchers examined the percentage of motorists traveling more than 15 mph above the posted advisory speed of 30 mph. Figure 25 shows the noncompliance percentage of motorists traveling above 45 mph at the start and middle of the curve. The trend as for noncompliance shows that the percentage of motorists not complying decreases with the installation of chevrons at the start of the curve. However, at the mid of the curve the noncompliance has an increasing trend. This is an important observation, as it appears that the percentage of noncompliance is decreasing at the start of the curve after the chevron pavement markings were installed.

The percentage of noncompliance at the middle of the curve has increased after the chevron pavement markings were installed. This could be due to the fact that once the motorist reaches the middle of the curve there is better visibility of the curve ahead encouraging motorists to speed up to make for the time lost at the start of the curve. However, results of noncompliance are encouraging in support of chevron markings at the start of the curve which is preferable, since sight distance at the start of the curve is critically limited, requiring motorists to slow down to be able to traverse the curve ahead.

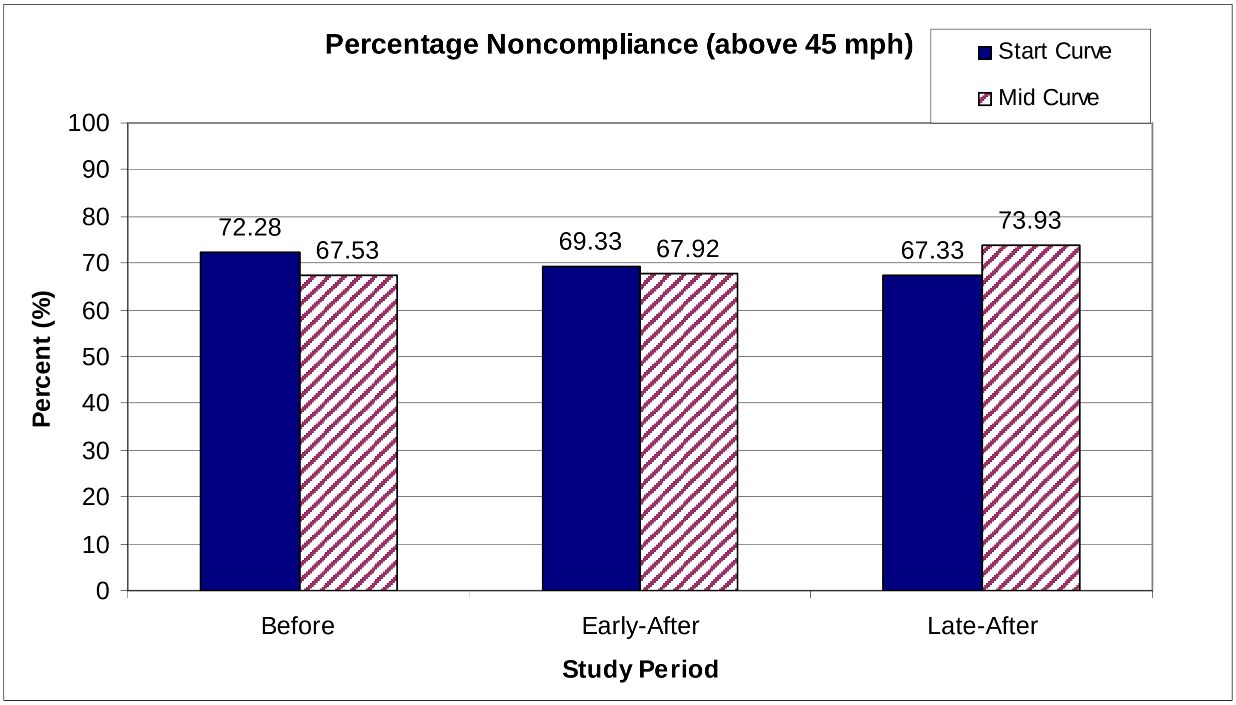


Figure 25. Noncompliance with 45 mph (Motorists Traveling 15 mph above Advisory Speed).

A detailed noncompliance study for motorists traveling 15 mph above the posted advisory speed was also done for different categories like vehicle class, and daylight conditions. Tables 14 and 15 present a matrix summarizing the results of noncompliance.

Table 14. Noncompliance Results for Motorists Traveling Above 45 mph by Vehicle Type.

Measured at	Category	Vehicles in Before Period			Vehicles in Early-After Period			Vehicles in Late-After Period		
		Total Count	45+ mph	% Non-Compliant	Total Count	45+ mph	% Non-Compliant	Total Count	45+ mph	% Non-Compliant
Mid Curve	Heavy Vehicles	540	64	**11.85**	142	15	**10.56**	944	315	**33.37**
	Passenger Vehicles	25,787	17,904	**69.43**	6458	4526	**70.08**	36,802	27,784	**75.50**
	Rigid Vehicles	2471	1481	**59.94**	460	254	**55.22**	4035	2789	**69.12**
	All Vehicles	28,798	19,447	**67.53**	7060	4795	**67.92**	41,781	30,888	**73.93**
Start Curve	Heavy Vehicles	515	105	**20.39**	148	18	**12.16**	836	116	**13.88**
	Passenger Vehicles	28,524	21,190	**74.29**	7026	5034	**71.65**	44,699	31,019	**69.40**
	Rigid Vehicles	1207	566	**46.89**	298	128	**42.95**	1817	749	**41.22**
	All Vehicles	30,246	21,861	**72.28**	7472	5180	**69.33**	47,352	31,884	**67.33**

Table 15. Noncompliance Results for Motorists Traveling Above 45 mph by Daylight Condition.

Measured at	Category	Vehicles in Before Period			Vehicle in Early-After Period			Vehicles in Late-After Period		
		Total Count	45+ mph	% Non-Compliant	Total Count	45+ mph	% Non-Compliant	Total Count	45+ mph	% Non-Compliant
Mid Curve	All Daytime	13,346	9420	**70.58**	4094	2806	**68.54**	22,856	17,511	**76.61**
	All Nighttime	4559	2931	**64.29**	1086	668	**61.51**	8331	5416	**65.01**
Start Curve	All Daytime	14,122	10,735	**76.02**	4304	3065	**71.21**	27,252	19,361	**71.04**
	All Nighttime	4809	3193	**66.40**	1147	703	**61.29**	8771	4753	**54.19**

CHAPTER 7. FINDINGS AND RECOMMENDATIONS

This project looked at the effectiveness of the converging chevron pavement marking in reducing speeds on a freeway-to-freeway connector ramp. A converging chevron pavement marking pattern was installed on Ramp K at the interchange of US 54 and IH 10 in El Paso, Texas. This ramp connects US 54 westbound to IH 10 westbound. To evaluate the impact of the pavement marking pattern, per-vehicle speed and classification data were collected during three discrete time periods (before, early-after, and late-after deployment of the markings) at four locations on the project ramp (two locations upstream of the curve, at the start of the freeway-to-freeway connector curve, and at the middle of the connector curve).

This chapter summarizes the characteristics of vehicular speeds at the project site for all time periods and presents the findings of the before-after effects of chevron pavement markings installation on vehicular speeds at the test facility. The final section of this chapter provides the recommendation for chevron marking installation as a tool for speed reduction on freeway-to-freeway connector ramps.

FINDINGS

The project facility has a posted advisory speed of 30 mph on the curve, which requires motorists to reduce from 60 mph to 30 mph as they enter the curve. General statistics computed on all three datasets (before, early-after, and late-after) indicate that mean speed of the motorists at upstream location was about 60-63 mph, mean speeds at the start of the curve and middle of the curve ranged between 47 to 48 mph. This indicates that on average motorists reduce their speed by about 12 to 16 mph as they traverse from the upstream location to the start or mid of the curve. This trend is observed in all study periods—before, early-after and late-after, and is in contrast to the 30 mph reduction recommended by the curve advisory speed of 30 mph (considering the upstream speed limit signing of 60 mph).

A general observation of deceleration characteristics by vehicle class indicates that heavy trucks decelerate more than passenger vehicles as motorists travel from upstream, through the chevron marking treatment, to the start of the curve in all time-periods (before, early-after and late-after periods). However, between the start of the curve and the middle of the curve there were varied observations, most notably in the late-after period when all vehicles seemed to accelerate between start of the curve and middle of the curve.

Although motorists decreased their speeds on an average by 12 to 16 mph in all time-periods on approach to the connector curve, the speeds of motorists driving 15 mph above the posted advisory speed did decrease after the installation of chevron pavement markings at the start of the connector curve.

The effectiveness of chevron pavement markings on speed reduction was obtained from the before-after analysis. In this analysis, mean speeds were compared between the following study periods at each location on the freeway-to-freeway connector curve (refer to Tables 12 and 13 of Chapter 7):

- before and early-after,
- before and late-after, and
- early-after and late-after.

Comparison of mean speeds for all vehicle classes from the before to early-after periods indicates a slight decrease in speeds after the installation of chevrons at the start and mid of the curve. However, the magnitude of the decrease in overall mean speed is about 0.14 mph to 0.45 mph. Although the magnitude is small, the effect of chevrons in decreasing the speeds was found to be statistically significant at a 95 percent confidence level. This indicates that chevron markings were effective in reducing overall mean speed on freeway-to-freeway connectors.

In the before and early-after comparison by vehicle classification, heavy vehicles had a higher reduction in mean speeds when compared to passenger vehicles at the start of the curve. This result is promising as trucks are more prone to speed-related incidents on freeway-to-freeway connector ramps. However, there was no significant difference in speeds from before to early-after period upstream of the curve. Hence, the converging chevron pattern that was installed between the upstream section and the start of the curve seems to have influenced motorists' perception to reduce speeds.

The before and early-after comparison segregated by daytime and nighttime conditions indicated that the reduction in mean speed after installation of chevron was slightly greater during nighttime conditions than daylight conditions. This could have resulted from the visual effect of pavement marking being more prominent with high retroreflectivity.

A comparison of mean speeds between the before and late-after periods indicates that there was a reduction in speed at upstream of the curve and at the start of the curve. However, at the middle of the curve, a significant increase in speeds was observed. A noticeable difference in the late-after period was that reduction in speed due to chevron marking installation was observed at the upstream section itself, whereas in the early-after period the speed reduction was observed only from the start of the curve. A possible reason for this could be that motorists become cognizant of the chevron markings over time and reduce their speeds even before they drive through the converging chevron markings.

Also, in the late-after period, all vehicle categories showed significant reduction in speed at the upstream and start of the curve, with heavy trucks being the most affected by the chevron markings to reduce speeds. Moreover, the magnitude of reduction in mean speeds from before to late-after was much greater than the reduction in speeds from before to early-after. This indicates that the effectiveness of chevron markings did not degrade over time.

However, the before to late-after comparison of mean speeds at the middle of the curve showed a significant increase for all vehicle classes. The observed increase in speeds at the middle of the curve could be due to motorists slowing more before the curve, but then judging the upcoming curve and accelerating through. For this particular freeway-to-freeway connector curve, motorists have much better sight distance by the time they reach the start of the connector curve, which could encourage some acceleration.

RECOMMENDATIONS

Based on the above findings, chevron pavement markings could be a promising traffic engineering tool for reducing speeds on the approach to freeway-to-freeway connector ramps. Researchers of this project recommend that TxDOT use converging chevron pavement markings on freeway-to-freeway connector ramps in the state of Texas where a speed reduction may need to be strongly encouraged to drivers. Researchers also recommend that converging chevron markings be used on curves with the following characteristics:

- high-speed approaches followed by lower-speed curves, typically with sight distance restrictions,
- curves with higher numbers and percentages of truck traffic,
- curves on vertical grade (either up- or downgrade),; and
- curves with a demonstrated crash experience.

However, TxDOT should be cognizant of the fact that wide use of chevron pavement markings on all curves could result in motorists losing the perceptual benefit of converging chevrons, and the intended purpose of chevron markings, to reduce speeds, may become less effective. Hence, chevron markings should be selectively used at minimal locations as recommended above and preferably, after other low-cost traffic engineering solutions like advisory signs and rumble strips have been tried.

However, considering that there have been few long-term detailed evaluations of converging chevron effectiveness, it is recommended that additional implementation studies should be conducted on connector ramps with varied geometries, speed characteristics, and vehicle characteristics. These further implementation studies should incorporate crash analysis to ascertain the benefits of converging chevron pavement markings.

The following recommendations would be of interest for future research evaluating chevron pavement markings:

- Different chevron pattern designs (with varied spacing, widths, etc.) could be evaluated for the ability to provide enhanced perceptual effect in reducing speeds.

- Evaluate the effectiveness of a shorter extent of chevron marking placement, with or without the converging design. There were indications in this project that chevron markings could have been more of a warning tool for motorists than a perceptual tool. Hence, shorter markings, if found as effective as the current chevron design, can be more cost effective. Traffic speed measurements could be made at more frequent locations along the approach to (and within) a curve to more precisely evaluate the chevron markings (such as exact deceleration patterns, exact locations where speed reduction occurs, etc.)

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APPENDIX A. TEXAS DEPARTMENT OF TRANSPORTATION SURVEY RESULTS

SURVEY RESULTS FOR THE STATE OF THE PRACTICE FOR ADVISORY SPEED SETTING ON FREEWAY-TO-FREEWAY CONNECTOR CURVES

TEXAS DEPARTMENT OF TRANSPORTATION TRANSPORTATION OPERATIONS ENGINEERS

SURVEY RESULTS

1. Has your agency encountered safety problems related to trucks (and/or other vehicles with high centers-of-gravity) on freeway-to-freeway interchange ramps?

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	10	42%	59%
No	7	29%	41%
No Response	7	29%	
Total Responses	24		

If YES, what types of countermeasures have you used for such problems? (Check all that apply):

1A. Advisory speed limits for all vehicles on particular ramps.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	10	42%	59%
No	7	29%	41%
No Response	7	29%	
Total Responses	24		

1B. Advisory speed limits for trucks on particular ramps.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	4	17%	24%
No	13	54%	76%
No Response	7	29%	
Total Responses	24		

1C. Differential advisory speed limits for cars and trucks on particular ramps.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	0	0%	0%
No	17	71%	100%
No Response	7	29%	
Total Responses	24		

1D. Regulatory speed limits for all vehicles on particular ramps.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	0	0%	0%
No	17	71%	100%
No Response	7	29%	
Total Responses	24		

1E. Regulatory speed limits for trucks on particular ramps.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	0	0%	0%
No	17	71%	100%
No Response	7	29%	
Total Responses	24		

1F. Special warning signs for trucks (truck rollover/tipping signs).

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	5	21%	29%
No	12	50%	71%
No Response	7	29%	
Total Responses	24		

1G. Special warning signs for trucks with permanent flashers.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	3	13%	18%
No	14	58%	82%
No Response	7	29%	
Total Responses	24		

1H. Special warning signs for trucks with flashers activated when a high-speed truck is detected.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	1	4%	6%
No	16	67%	94%
No Response	7	29%	
Total Responses	24		

1I. Special pavement marking warnings for all vehicles.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	4	17%	24%
No	13	54%	76%
No Response	7	29%	
Total Responses	24		

1J. Special pavement marking warnings for trucks.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	1	4%	6%
No	16	67%	94%
No Response	7	29%	
Total Responses	24		

1K. Reconstruction of ramp to change horizontal curve radius or superelevation.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	1	4%	6%
No	16	67%	94%
No Response	7	29%	
Total Responses	24		

1L. Others (please specify)

Beaumont District. Our location involves the mainlanes of IH 10 westbound at SH 12 in Vidor (Orange Co) between mile marker 861 and 862. Westbound has a 3-degree right curve at the SH 12 overpass. The location had various truck accidents in the past. This location has had numerous warning features installed for trucks. The other locations listed in Question 2 have not had the problems or the public concern that our IH 10 mainlane site has had. We have not had freeway-to-freeway connector problem with trucks as much as the mainlane curve problem mentioned.

Fort Worth District. We removed the southbound IH 35 to southbound SH 287 ramp. It was known for truck rollovers. Traffic is now routed to the IH 30 exit. We had signed for "No Trucks" on this ramp prior to its removal.

San Antonio District. Used linear delineators (6 inch), Type E Yellow along the IH 35 southbound to IH 10 westbound connector ramp.

2. With respect to truck crash history, please list any existing problematic freeway-to-freeway connector locations below (use the back of this page to list more than 10 locations).

Curve #	District	City	Freeway From	Freeway To	Advisory Speed	Ramp Type
1	AMA	Amarillo	IH 40 WB	IH 27 SB	35	Directional
2	ATL	Texarkana	IH 30 WB	US 59 SB	30	Cloverleaf
3	BMT	Port Arthur	SH 73 WB	US 69 NB	35	Directional
4	BMT	Port Arthur	SH 73 EB	US 69 NB	20	Cloverleaf
5	BMT	Orange	IH 10 WB	SH 87	15	Buttonhook Exit Ramp
6	BMT	Vidor	IH 10 WB	Mainlanes	45	Mainlanes
7	CRP	Corpus Christi	US 77 SB	IH 37 SB	45	Directional
8	CRP	Corpus Christi	US 77 SB	IH 37 NB	45	Directional
9	CRP	Corpus Christi	IH 37 SB	SH 358 EB	45	Directional
10	CRP	Corpus Christi	IH 37 NB	SH 358 EB	45	Directional
11	CRP	Corpus Christi	SH 358 WB	IH 37 NB	45	Directional
12	CRP	Corpus Christi	IH 37 SB	SH 286 SB	45	Directional
13	CRP	Corpus Christi	IH 37 NB	SH 286 SB	45	Directional
14	CRP	Corpus Christi	SH 386 NB	IH 37 SB	40	Directional
15	CRP	Corpus Christi	SH 386 SB	IH 37 NB	40	Directional
16	CRP	Corpus Christi	IH 37 SB	US 181 NB	35	Directional
17	DAL	Dallas	IH 45 NB	IH 20 EB	45	Directional
18	DAL	Dallas	US 80 WB	IH 635 NB	*	Directional
19	DAL	Dallas	US 80 WB	IH 635 SB	30	Directional
20	DAL	Dallas	IH 30 EB	Loop 12	20	Cloverleaf
21	DAL	Dallas	Loop 12	IH 30 WB	20	Cloverleaf
22	DAL	Dallas	IH 635 NB	IH 635 WB	*	Interchange
23	DAL	Dallas	Spur 366 WB	IH 356 SB	*	Cloverleaf
24	ELP	El Paso	US 54 EB	IH 10 WB	35	Directional
25	ELP	El Paso	Loop 375 NB	IH 10 WB	*	Cloverleaf
26	PAR	Sherman	Spur 503 WB	US 75 SB	45	Directional
27	FTW	Fort Worth	IH 35W SB	IH 820 EB (North)	*	Directional

Table continued on next page.

2. With respect to truck crash history, please list any existing problematic freeway-to-freeway connector locations below (use the back of this page to list more than 10 locations) (continued).

Curve #	District	City	Freeway From	Freeway To	Advisory Speed	Ramp Type
28	FTW	Fort Worth	IH 35W NB	IH 820 WB (North)	*	Directional
29	FTW	Fort Worth	IH 35W SB	Spur 280	*	Exit Ramp
30	FTW	Fort Worth	US 287 NB	IH 35W NB	*	Entrance Ramp/ Connection
31	FTW	Fort Worth	IH 820 EB	US 287 NB	*	Entrance Ramp/ Connection
32	FTW	Arlington	IH 30 EB	SH 360 NB	*	Exit Ramp
33	FTW	Arlington	IH 30 WB	SH 360 SB	*	Exit Ramp
34	FTW	Fort Worth West	IH 820 SB	IH 30 WB	*	Exit Ramp
35	FTW	Fort Worth West	IH 30 EB	IH 820 SB	*	Exit Ramp
36	FTW	Fort Worth	SH 360 NB	SH 183 EB	*	Exit Ramp Directional
37	SAT	San Antonio	IH 35 SB	IH 10 WB	25	Directional
38	SAT	San Antonio	IH 35 NB	US 281 NB	24	Directional
39	SAT	San Antonio	IH 410 EB	IH 35 SB	25	Directional
40	SAT	San Antonio	IH 35 NB	IH 410 WB	25	Directional
41	HOU	Houston	IH 10 WB	IH 610 NB	40	Directional
42	HOU	Houston	IH 10 WB	IH 610 SB	40	Directional
43	HOU	Houston	IH 610 EB (North)	IH 45 NB	35	Directional
44	HOU	Houston	IH 610 WB (North)	IH 45 SB	40	Directional
45	HOU	Houston	IH 45 NB	IH 610 WB (North)	40	Directional
46	HOU	Houston	IH 45 NB	IH 610 EB (North)	40	Directional
47	HOU	Houston	IH 45 SB	IH 610 WB (North)	45	Directional
48	HOU	Houston	IH 45 SB	IH 610 EB (North)	40	Directional
49	HOU	Houston	IH 610 EB (North)	US 59 NB	25	Directional
50	HOU	Houston	US 59 NB	IH 610 WB (North)	30	Directional
51	HOU	Houston	US 59 SB	IH 610 EB (North)	35	Directional
52	HOU	Houston	SH 225 WB	IH 610 NB (East)	40	Directional
53	HOU	Houston	IH 610 EB (South)	SH 288 NB	40	Directional
54	HOU	Houston	SH 288 NB	IH 610 WB (South)	40	Directional
55	HOU	Houston	SH 288 NB	IH 610 EB (South)	40	Directional
56	HOU	Houston	US 290 WB	IH 610 NB (West)	40	Directional
57	HOU	Houston	IH 45 SB	IH 10 WB	40	Directional

3. Of the freeway-to-freeway connectors listed in question #2, have any traffic control treatments (signing, pavement markings, barriers, truck barriers, chevrons, delineators, etc.) been used to correct truck operational problems?

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	9	38%	75%
No	3	13%	25%
No Response	12	50%	
Total Responses	24		

3A. If YES, please list location (by #) and modifications made. Please also indicate if any "before-after" study had quantified the benefits of the modifications made (use back of sheet for more space).

Table 2 Curve Number	Modifications Noted
1	Installed chevrons and special signing
2	Installed oversize chevrons
4	Installed signing, pavement markings
5	Installed signing, chevrons, pavement markings
6	Installed large warning signs with tipping-truck graphic and flashing lights, pavement marking with "Trucks 45 mph," large chevrons and metal orange flags
7-16	Yes, modifications made (not specified), results not quantified
17-23	Yes, modifications made (not specified), results not quantified
26	Added tipping truck on curve sign; no studies but trucks stopped losing their loads on curve
27-36	Do not have specific information
37-40	Installed delineators, chevrons, and raised pavement markers on curves #37,#39, and #40; installed dynamic feedback sign for speed advisory on #37; installed chevrons on curve #38
41-57	All locations have flashing lights with tipping-truck signs (ground mounted) except for curve #49, which has overhead tipping-truck signs and roadside signs with flashing lights; no before/after studies

4. Have any of the ramps listed in question #2 been re-designed or geometrically modified to address a higher truck crash frequency (by increasing curve radius, superelevation, etc.)?

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	2	8%	18%
No	9	38%	82%
No Response	13	54%	
Total Responses	24		

4A. If YES to question #4, please list location (by #) and modifications made.

Table 2 Curve Number	Modifications Noted
2	Currently on schedule for directional interchange
7-16	We have modified some ramps but not specifically for trucks
25	New interchange is under construction

5. Has any signing been installed on any of these freeway-to-freeway connector ramps that specifically address truck warning speeds?

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	3	13%	25%
No	9	38%	75%
No Response	12	50%	
Total Responses	24		

5A. If YES, please list location (by #) and modifications made.

Table 2 Curve Number	Modifications Noted
18	Unspecified
20	Unspecified
21	Unspecified
47-57	All locations have flashers with tipping-truck signs (ground mounted) except curve #49, which has overhead tipping-truck signs and roadside signs with flashers

6. Has any signing been installed on any freeway-to-freeway connectors that has different advisory speeds for trucks as opposed to the posted advisory speed for other vehicles?

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	0	0%	0%
No	13	54%	100%
No Response	11	46%	
Total Responses	24		

7. At what staff level is advisory speed setting decisions made?

Response	Number of Responses	Percent of Total
Director of Trans. Operations	12	63%
Traffic Engineering Section	2	11%
Engr. Tech. Supervisor	1	5%
Maintenance Supervisor	3	16%
Engineering Tech.	1	5%
Total Responses*	19	

*Eight districts did not respond; some districts indicated more than one level of decision making

8. Does your agency have a training program that addresses setting advisory speeds on curves?

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	2	8%	12%
No	15	63%	88%
No Response	7	29%	
Total Responses	24		

9. Does your agency use the ball-bank indicator as the measuring device to set freeway-to-freeway curve advisory speeds?

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	15	63%	88%
No	2*	8%	12%
No Response	7	29%	
Total Responses	24		

*Note: The Beaumont District indicated use of ball-bank and electronic inclinometer; San Antonio responded that they use engineering judgment and/or speed observations.

10. Does your agency use the traditional ball-bank readings (14^{o} for speeds below 20 mph, 12^{o} for speeds 20 to 35 mph, and 10^{o} for speeds above 35 mph) to set the freeway-to-freeway connector curve advisory speed?

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	15	63%	88%
No	2*	8%	12%
No Response	7	29%	
Total Responses	24		

*Note: The Houston District uses 10 degrees on all curves; the San Antonio District uses engineering judgment and/or speed observations.

11. Are you aware that proposed revisions (part 2) to the 2001 MUTCD will allow engineering judgment to set advisory speeds on curves using up to a 16^{o} reading on the ball-bank indicator?

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	2	8%	12%
No	15	63%	88%
No Response	7	29%	
Total Responses	24		

12. If empirically justified, would you use a ball-bank indicator reading higher than 10^o to set advisory speeds on curves?

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	8	33%	57%
No	6	25%	43%
No Response	10	42%	
Total Responses	24		

13. What criteria are used to install signing for freeway-to-freeway connectors that addresses advisory speeds for trucks (safety record, speed studies, etc.)?

Response	Number of Responses*	Percent of Total	Percent of Responding
No Specific Criteria	1	5%	12%
Crash History	5	25%	63%
Speed Study	2	10%	25%
No Response	12	60%	
Total Responses	20		

*Fifteen districts did not respond; some districts indicated more than one level of criteria

14. If any signing has been installed on any freeway-to-freeway connector that has different advisory speeds for trucks as opposed to the posted advisory speed for other vehicles, what criteria were used to set the different advisory speeds?

No responses to this question – no districts have used differing speeds for trucks versus cars.

15. Has your agency developed any non-standard signs or sign panels for advisory speed limits on freeway-to-freeway connectors?

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	3	13%	19%
No	13	54%	81%
No Response	8	33%	
Total Responses	24		

APPENDIX B. UNITED STATES STATE DEPARTMENTS OF TRANSPORTATION SURVEY RESULTS

SURVEY RESULTS FOR THE STATE OF THE PRACTICE FOR ADVISORY SPEED SETTING ON FREEWAY-TO-FREEWAY CONNECTOR CURVES

UNITED STATES STATE DEPARTMENTS OF TRANSPORTATION

SURVEY RESULTS

1. At what staff level are freeway-to-freeway connector advisory speed-setting decisions made ?

Response	Number of Responses	Percent of Total	Percent of Responding
District/Region Level	16	73%	80%
State Level	4	18%	20%
No Response	2	9%	
Total Responses	22		

2. Has your agency developed any non-standard signs or sign panels for advisory speed limits on freeway-to-freeway connectors (Non-standard refers to signing not in the Standard Highway Signs Manual)?

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	8	36%	36%
No	14	64%	64%
No Response	0	0%	
Total Responses	22		

3. Does your agency use overhead signing for advisory speed limits on freeway-to-freeway connectors?

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	9	41%	43%
No	12	55%	57%
No Response	1	5%	
Total Responses	22		

4A. Has your agency encountered any safety problems related to the operation of heavy trucks on freeway connectors?

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	14	64%	64%
No	8	36%	36%
No Response	0	0%	
Total Responses	22		

4B. If YES to question 4A, what is the nature of these problems?

Response	Number of Responses	Percent of Total
Truck Overturns/Tipping/Excessive Speed	10	53%
Only on Low Speed Ramps (<20 mph)	2	11%
Cloverleaf Ramps	3	16%
Run-off-the-road Crashes	1	5%
Geometrics (reverse curve)	2	11%
Downgrade	1	5%
Total Responses	19	

(each state could have multiple responses)

5. If your agency has experienced safety problems related to trucks freeway-to-freeway interchange ramps, what types of countermeasures have you used for such problems?

5A. Advisory speed limits for all vehicles on particular ramps.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	16	73%	73%
No	6	27%	27%
No Response	0	0%	
Total Responses	22		

5B. Advisory speed limits for trucks on particular ramps.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	8	36%	36%
No	14	64%	64%
No Response	0	0%	
Total Responses	22		

5C. Differential advisory speed limits for cars and trucks on particular ramps.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	0	0%	0%
No	22	100%	100%
No Response	0	0%	
Total Responses	22		

5D. <u>Regulatory speed limits</u> for all vehicles on particular ramps.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	0	0%	0%
No	22	100%	100%
No Response	0	0%	
Total Responses	22		

5E. <u>Regulatory speed limits for trucks</u> on particular ramps.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	0	0%	0%
No	22	100%	100%
No Response	0	0%	
Total Responses	22		

5F. <u>Special warning signs for trucks</u> (truck rollover/tipping signs)

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	13	59%	59%
No	9	41%	41%
No Response	0	0%	
Total Responses	22		

5G. <u>Special warning signs for trucks with permanent flashers</u>.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	8	36%	36%
No	14	64%	64%
No Response	0	0%	
Total Responses	22		

5H. <u>Special warning signs for trucks with flashers activated when a high-speed truck is detected</u>.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	2	9%	9%
No	20	91%	91%
No Response	0	0%	
Total Responses	22		

5I. Special pavement marking warnings for all vehicles.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	1	5%	5%
No	21	95%	95%
No Response	0	0%	
Total Responses	22		

5J. Special pavement marking warnings for trucks.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	1	5%	5%
No	21	95%	95%
No Response	0	0%	
Total Responses	22		

5K. Reconstruction of ramp to change horizontal curve radius or superelevation.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	6	27%	27%
No	16	73%	73%
No Response	0	0%	
Total Responses	22		

5L. Others (please specify).

- Regulatory sign "TRUCKS USE RIGHT LANE" on a 2-lane ramp (Minnesota)
- Installation of larger (60 inch x 60 inch) graphic truck rollover/tipping signs
- Installation of larger (24 inch x 30 inch) advisory speed signs
- Installation of "Safe-T-Spins" on warning signs
- Use of W1-13 sign (truck rollover sign)
- Move advance warning signs back upstream before downgrade
- Add chevrons (ASTM TY IX) for curve delineation
- Installation of large diagrammatic signing
- Installation of warning signs that light up with fiber optic lights

6A. Has your state installed any warning signs on freeway-to-freeway connector ramps that specifically address truck warning speeds?

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	10	45%	45%
No	12	55%	55%
No Response	0	0%	
Total Responses	22		

6B. If answering YES to question 6A, please list some example locations.

IH 20 EB at SC RT 277; IH 20 WB at IH 26 EB; IH 126 at Greystone Blvd.; IH 95 NB at IH 26 WB (South Carolina)

IH 85 at IH 77 in Charlotte, North Carolina. (Truck rollover signs with permanent flashers); IH 26/IH 240 at US 19-23 (Truck rollover signs with speed advisories); Note: We are in the process of installing continuous flashers on these signs due to continued problems (North Carolina)

TRUCKS-CURVE TIGHTENS-MAX SPEED XX MPH (Iowa)

IH 64 Eastbound at IH 77 (Bigley Avenue Interchange) in Charleston, West Virginia; IH 64 at IH 77 split in Charleston; IH 64 at IH 77 split in Beckley (West Virginia)

San Antonio, Beaumont (Texas)

Use of W4-22 (CA Code) – Tipping-truck symbol with advisory speed limit, used on ramps or branch connectors (California).

We use truck rollover/tipping sings on some off ramps that have tight horizontal curves (Nevada).

Truck rollover sign W1-13 (Vermont)

7. If your answer to question 6A was YES, what criteria are used to install signing for freeway-to-freeway connectors that addresses advisory speeds for trucks (safety record, speed studies, etc.)?

Response	Number of Responses	Percent of Total
Crash History	8	80%
Ramp Geometrics	1	10%
Spot Speed Study	1	10%
Total Responses	10	

8A. Has your state installed any signing on any freeway-to-freeway connectors that has different advisory speeds for trucks as opposed to the posted advisory speed for other vehicles.

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	0	0%	0%
No	20	91%	100%
No Response	2	9%	
Total Responses	22		

8B. If YES, please list location and describe the signing scheme.

(Iowa) IH 380 SB ramp to IH 80 WB. The sign described in the answer to question 6B is installed at the beginning of the ramp, and a standard RAMP advisory speed is installed further down the ramp for other vehicles.

9. If your answer to question 8A was YES, please describe what criteria were used to set the different advisory speeds for trucks and cars?

(Iowa) Crash History

10. What method does your agency use to set freeway-to-freeway curve advisory speeds (ball-bank, operating speeds, etc.)?

Response	**Number of Responses**	**Percent of Total**
Ball-bank Indicator	17	60%
Electronic Ball-bank Indicator	1	4%
Based on Curve Geometric Features (e+f, D)	5	17%
Speed Study	1	4%
Sight Distance	1	4%
Design Speed	2	7%
No Response	1	4%
Total Responses	28	100%

(Each State could have multiple responses)

11. If your agency uses the ball-bank indicator, what readings are used and how do they vary with speed (for example: 14 for <= 20 mph)?

- 10 degrees is used for all speeds.
- 14 – Below 20 mph; 12 – 20 to 30 mph; 10 – 35 mph and above
- 10 degrees for above 30 mph (advisory at 35); 12 degrees for 30 and under
- 10 degrees (for speeds 35 mph and higher); 12.5 degrees (for speeds 25 mph and 30 mph); 15 degrees (for speeds 20 mph and below)
- We have a spread sheet program that the electronic ball-bank readings are downloaded into, and it calculates the safe speed for a particular curve.

- NY State MUTCD recommends a maximum deflection of 10 degrees. However, this may be somewhat conservative at lower speeds. The AASHTO manual, A Policy on Geometric Design of Highways and Streets, allows 14 degrees at speeds equal to or less than 20 mph and 12 degrees for speeds of 25 and 30 mph.

12. Are you aware that the 2003 MUTCD (Section 2C.36) allows engineering judgment to set advisory speeds on curves using up to a 16^{o} reading on the ball-bank indicator?

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	9	41%	41%
No	13	59%	59%
No Response	0	0%	
Total Responses	22		

13. Will your state consider using a ball-bank indicator reading higher than 10^{o} to set advisory speeds on higher-speed connector curves?

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	12	55%	55%
No	10	45%	45%
No Response	0	0%	
Total Responses	22		

14A. Has your agency used any pavement markings or marker treatments intended to warn trucks or other heavy vehicles about making appropriate speed decisions at freeway-to-freeway connectors?

Response	Number of Responses	Percent of Total	Percent of Responding
Yes	3	14%	14%
No	18	82%	86%
No Response	1	5%	
Total Responses	22		

14B. If YES, what is the nature of these problems and how were pavement markings or markers used?

- The Wisconsin DOT has used transverse rumble strips to call attention to advisory signs.
- We have used chevron warning signs in some of the ramp curves.
- Trucks going too fast on a reverse curve. Markers used on pavement “TRUCK SPEED 45 MPH.”

15. Do you have any additional thoughts or comments on signing and pavement markings for heavy trucks on freeway-to-freeway connectors?

Colorado, Pennsylvania, and a few other states have had success in using automated warning systems to reduce the number of truck rollovers. Their systems weigh each truck in motion, measure the height of a truck's load, calculates an appropriate advisory speed, and then displays the speed on a Dynamic Message Sign. The North Carolina DOT has considered installing similar type systems, but as of today, no systems like this have been installed.

There is a pooled fund study for traffic control devices that is looking at using markings for speed reduction. The results of this study should be out later this year.

New York is experimenting with speed reduction markings at a freeway ramp from NY 690 to IH 90 near Syracuse. This is being done as one of four test sites for a FHWA Pooled Fund Traffic Control Study. Other sites will be in Texas and Mississippi. Although the markings are meant for all vehicles, it is hoped that they will have a positive impact on trucks as well.

APPENDIX C. DETAILED STATISTICAL ANALYSIS OUTPUT

The GLM Procedure

Class Level Information

Class	Levels	Values
PEAK	2	OFF_PEAK PEAK
PERIOD	3	Before EARLY_AFTER LATE_AFTER
CURVE	4	End Mid Start Upstream
VEH_CLASS	3	Heavy Trk Pass Veh Rigid Veh

Number of Observations Read	278324
Number of Observations Used	278324

ANOVA - WITH GAPS >5, Before_EarlyAfter_LateAfter

The GLM Procedure

Dependent Variable: SPEED

Source	DF	Sum of Squares	Mean Square	F Value	Pr > F
Model	65	15160990.45	233246.01	7613.71	<.0001
Error	278258	8524428.59	30.63		
Corrected Total	278323	23685419.04			

R-Square	Coeff Var	Root MSE	SPEED Mean
0.640098	10.30349	5.534888	53.71855

Source	DF	Type I SS	Mean Square	F Value	Pr > F
PEAK	1	12523.58	12523.58	408.80	<.0001
PERIOD	2	131258.87	65629.44	2142.30	<.0001
CURVE	3	14564112.01	4854704.00	158469	<.0001
VEH_CLASS	2	195935.67	97967.84	3197.91	<.0001
PEAK*PERIOD	2	2119.46	1059.73	34.59	<.0001
PEAK*CURVE	3	3450.18	1150.06	37.54	<.0001
PERIOD*CURVE	5	197637.82	39527.56	1290.28	<.0001
PEAK*PERIOD*CURVE	5	19.15	3.83	0.13	0.9868
PEAK*VEH_CLASS	2	81.06	40.53	1.32	0.2664
PERIOD*VEH_CLASS	4	4088.97	1022.24	33.37	<.0001
CURVE*VEH_CLASS	6	42389.87	7064.98	230.62	<.0001
PEAK*PERIOD*VEH_CLAS	4	296.54	74.13	2.42	0.0462
PEAK*CURVE*VEH_CLASS	6	1630.72	271.79	8.87	<.0001
PERIOD*CURVE*VEH_CLA	10	5197.15	519.72	16.96	<.0001
PEAK*PERI*CURV*VEH_C	10	249.41	24.94	0.81	0.6150

Source	DF	Type III SS	Mean Square	F Value	Pr > F
PEAK	1	1145.3293	1145.3293	37.39	<.0001
PERIOD	2	2521.3698	1260.6849	41.15	<.0001
CURVE	3	460915.4005	153638.4668	5015.13	<.0001
VEH_CLASS	2	42071.5340	21035.7670	686.66	<.0001
PEAK*PERIOD	2	459.1891	229.5945	7.49	0.0006
PEAK*CURVE	3	679.2277	226.4092	7.39	<.0001
PERIOD*CURVE	5	12282.5236	2456.5047	80.19	<.0001
PEAK*PERIOD*CURVE	5	213.8495	42.7699	1.40	0.2221
PEAK*VEH_CLASS	2	485.5392	242.7696	7.92	0.0004
PERIOD*VEH_CLASS	4	862.6163	215.6541	7.04	<.0001
CURVE*VEH_CLASS	6	10992.0664	1832.0111	59.80	<.0001

PEAK*PERIOD*VEH_CLAS	4	320.5858	80.1465	2.62	0.0333

The GLM Procedure

Dependent Variable: SPEED

Source	DF	Type III SS	Mean Square	F Value	Pr > F
PEAK*CURVE*VEH_CLASS	6	898.4031	149.7339	4.89	<.0001
PERIOD*CURVE*VEH_CLA	10	2220.5329	222.0533	7.25	<.0001
PEAK*PERI*CURV*VEH_C	10	249.4059	24.9406	0.81	0.6150

The GLM Procedure

Level of PEAK	N	------------SPEED------------ Mean	Std Dev
OFF_PEAK	43014	53.2224090	9.31142244
PEAK	235310	53.8092419	9.20623467

Level of PERIOD	N	------------SPEED------------ Mean	Std Dev
Before	89172	52.9008388	9.0612425
EARLY_AFTER	26961	55.2347279	10.1007381
LATE_AFTER	162191	53.9160884	9.1184299

Level of CURVE	N	------------SPEED------------ Mean	Std Dev
End	77539	47.7376830	5.17822851
Mid	85032	47.5650473	5.39919713
Start	82916	61.4264895	6.32574142
Upstream	32837	64.3127965	6.05656455

Level of VEH_CLASS	N	------------SPEED------------ Mean	Std Dev
Heavy Trk	5272	48.1159522	10.8654768
Pass Veh	257174	53.9465280	9.0698270
Rigid Veh	15878	51.8862388	10.2726895

Level of PEAK	Level of PERIOD	N	------------SPEED------------ Mean	Std Dev
OFF_PEAK	Before	11419	52.4979245	9.1524081
OFF_PEAK	EARLY_AFTER	4348	55.6174103	10.3747348
OFF_PEAK	LATE_AFTER	27247	53.1438470	9.1355413
PEAK	Before	77753	52.9600118	9.0463245
PEAK	EARLY_AFTER	22613	55.1611462	10.0457615
PEAK	LATE_AFTER	134944	54.0720143	9.1070625

Level of PEAK	Level of CURVE	N	------------SPEED------------ Mean	Std Dev
OFF_PEAK	End	11818	47.3194618	5.38899916
OFF_PEAK	Mid	12660	46.7582543	5.57977915
OFF_PEAK	Start	12641	60.3493315	6.42832105
OFF_PEAK	Upstream	5895	63.6559627	6.44874716

The GLM Procedure

Level of PEAK	Level of CURVE	N	------------SPEED------------ Mean	Std Dev

PEAK	End	65721	47.8128878	5.13584168
PEAK	Mid	72372	47.7061792	5.35454521
PEAK	Start	70275	61.6202476	6.28760673
PEAK	Upstream	26942	64.4565140	5.95779563

Level of PERIOD	Level of CURVE	N	------------SPEED------------ Mean	Std Dev
Before	End	28752	47.2233166	5.11258189
Before	Mid	30214	48.0326604	5.41656364
Before	Start	30206	63.1745349	5.31260382
EARLY_AFTER	End	7059	47.0805922	4.73290486
EARLY_AFTER	Mid	7471	47.5825994	5.38591049
EARLY_AFTER	Start	7355	63.1658600	5.31671588
EARLY_AFTER	Upstream	5076	66.3449764	5.81967923
LATE_AFTER	End	41728	48.2032568	5.24931037
LATE_AFTER	Mid	47347	47.2638752	5.36889310
LATE_AFTER	Start	45355	59.9802425	6.71989384
LATE_AFTER	Upstream	27761	63.9412197	6.02531356

Level of PEAK	Level of PERIOD	Level of CURVE	N	------------SPEED------------ Mean	Std Dev
OFF_PEAK	Before	End	3555	46.7252883	5.37365540
OFF_PEAK	Before	Mid	3742	47.2952699	5.55468708
OFF_PEAK	Before	Start	4122	62.1995391	5.63110941
OFF_PEAK	EARLY_AFTER	End	1085	47.1082028	4.88533057
OFF_PEAK	EARLY_AFTER	Mid	1125	47.3494222	5.68787905
OFF_PEAK	EARLY_AFTER	Start	1222	62.7438625	5.99104088
OFF_PEAK	EARLY_AFTER	Upstream	916	66.3438865	6.52117826
OFF_PEAK	LATE_AFTER	End	7178	47.6456673	5.44286101
OFF_PEAK	LATE_AFTER	Mid	7793	46.4150520	5.54919876
OFF_PEAK	LATE_AFTER	Start	7297	58.9031657	6.53357889
OFF_PEAK	LATE_AFTER	Upstream	4979	63.1614581	6.31252547
PEAK	Before	End	25197	47.2935826	5.07084182
PEAK	Before	Mid	26472	48.1368956	5.38872697
PEAK	Before	Start	26084	63.3286114	5.24405915
PEAK	EARLY_AFTER	End	5974	47.0755775	4.70509388
PEAK	EARLY_AFTER	Mid	6346	47.6239363	5.32999857
PEAK	EARLY_AFTER	Start	6133	63.2499429	5.16828909
PEAK	EARLY_AFTER	Upstream	4160	66.3452163	5.65439705
PEAK	LATE_AFTER	End	34550	48.3190999	5.20078279
PEAK	LATE_AFTER	Mid	39554	47.4311119	5.31676586
PEAK	LATE_AFTER	Start	38058	60.1867544	6.73546404
PEAK	LATE_AFTER	Upstream	22782	64.1116364	5.94724627

The GLM Procedure

Level of PEAK	Level of VEH_CLASS	N	------------SPEED------------ Mean	Std Dev
OFF_PEAK	Heavy Trk	450	48.4824444	11.2418712
OFF_PEAK	Pass Veh	40678	53.3546389	9.2010957
OFF_PEAK	Rigid Veh	1886	51.5013786	10.6166582
PEAK	Heavy Trk	4822	48.0817503	10.8302535
PEAK	Pass Veh	216496	54.0577396	9.0406479
PEAK	Rigid Veh	13992	51.9381146	10.2247232

Level of PERIOD	Level of VEH_CLASS	N	------------SPEED------------ Mean	Std Dev
Before	Heavy Trk	1747	48.1175157	11.1356487
Before	Pass Veh	82551	53.1983174	8.9547195

Before	Rigid Veh	4874	49.5769389	8.8817901
EARLY_AFTER	Heavy Trk	613	50.8479608	11.8991699
EARLY_AFTER	Pass Veh	24768	55.3557413	9.9531178
EARLY_AFTER	Rigid Veh	1580	55.0396835	11.2033239
LATE_AFTER	Heavy Trk	2912	47.5399038	10.3789958
LATE_AFTER	Pass Veh	149855	54.1257823	8.9420334
LATE_AFTER	Rigid Veh	9424	52.5518888	10.5408789

Level of CURVE	Level of VEH_CLASS	N	------------SPEED------------ Mean	Std Dev
End	Heavy Trk	1620	41.5316667	5.91130477
End	Pass Veh	68990	47.9475982	4.97463671
End	Rigid Veh	6929	47.0985857	5.97095242
Mid	Heavy Trk	1489	39.6766958	5.57186199
Mid	Pass Veh	80236	47.8531806	5.19295725
Mid	Rigid Veh	3307	44.1260054	6.28187850
Start	Heavy Trk	1594	58.5181932	6.76958656
Start	Pass Veh	78203	61.5318888	6.23756064
Start	Rigid Veh	3119	60.2701186	7.67570443
Upstream	Heavy Trk	569	59.8056239	5.93397089
Upstream	Pass Veh	29745	64.3541267	5.96566205
Upstream	Rigid Veh	2523	64.8420135	6.71173610

Level of PEAK	Level of PERIOD	Level of VEH_CLASS	N	------------SPEED------------ Mean	Std Dev
OFF_PEAK	Before	Heavy Trk	106	48.7566038	11.8425465
OFF_PEAK	Before	Pass Veh	10901	52.6804055	9.1215164
OFF_PEAK	Before	Rigid Veh	412	48.6322816	8.0773512
OFF_PEAK	EARLY_AFTER	Heavy Trk	56	54.6571429	12.5405358
OFF_PEAK	EARLY_AFTER	Pass Veh	4087	55.6479080	10.2593238

The GLM Procedure

Level of PEAK	Level of PERIOD	Level of VEH_CLASS	N	------------SPEED------------ Mean	Std Dev
OFF_PEAK	EARLY_AFTER	Rigid Veh	205	55.2717073	11.9421883
OFF_PEAK	LATE_AFTER	Heavy Trk	288	47.1809028	10.3489845
OFF_PEAK	LATE_AFTER	Pass Veh	25690	53.2759011	8.9985608
OFF_PEAK	LATE_AFTER	Rigid Veh	1269	51.8237983	10.8807169
PEAK	Before	Heavy Trk	1641	48.0762340	11.0909970
PEAK	Before	Pass Veh	71650	53.2771137	8.9265009
PEAK	Before	Rigid Veh	4462	49.6641641	8.9482271
PEAK	EARLY_AFTER	Heavy Trk	557	50.4649910	11.7765014
PEAK	EARLY_AFTER	Pass Veh	20681	55.2980030	9.8907164
PEAK	EARLY_AFTER	Rigid Veh	1375	55.0050909	11.0931291
PEAK	LATE_AFTER	Heavy Trk	2624	47.5793064	10.3834957
PEAK	LATE_AFTER	Pass Veh	124165	54.3016245	8.9202252
PEAK	LATE_AFTER	Rigid Veh	8155	52.6651870	10.4831458

Level of PEAK	Level of CURVE	Level of VEH_CLASS	N	------------SPEED------------ Mean	Std Dev
OFF_PEAK	End	Heavy Trk	134	41.3694030	6.40890156
OFF_PEAK	End	Pass Veh	10840	47.4314483	5.28749069
OFF_PEAK	End	Rigid Veh	844	46.8258294	5.92567017
OFF_PEAK	Mid	Heavy Trk	105	38.5695238	5.39036041
OFF_PEAK	Mid	Pass Veh	12232	46.9570389	5.44564203
OFF_PEAK	Mid	Rigid Veh	323	41.8922601	6.40749528
OFF_PEAK	Start	Heavy Trk	142	57.0338028	7.84757167
OFF_PEAK	Start	Pass Veh	12183	60.4663465	6.33403986
OFF_PEAK	Start	Rigid Veh	316	57.3278481	8.02535959

OFF_PEAK	Upstream	Heavy Trk	69	59.7826087	6.94937485
OFF_PEAK	Upstream	Pass Veh	5423	63.6479993	6.33388066
OFF_PEAK	Upstream	Rigid Veh	403	64.4263027	7.56911458
PEAK	End	Heavy Trk	1486	41.5462988	5.86646434
PEAK	End	Pass Veh	58150	48.0438160	4.90816328
PEAK	End	Rigid Veh	6085	47.1364174	5.97670678
PEAK	Mid	Heavy Trk	1384	39.7606936	5.57831392
PEAK	Mid	Pass Veh	68004	48.0143712	5.12964579
PEAK	Mid	Rigid Veh	2984	44.3677949	6.22128631
PEAK	Start	Heavy Trk	1452	58.6633609	6.64010385
PEAK	Start	Pass Veh	66020	61.7285186	6.19965809
PEAK	Start	Rigid Veh	2803	60.6018195	7.56531023
PEAK	Upstream	Heavy Trk	500	59.8088000	5.78790136
PEAK	Upstream	Pass Veh	24322	64.5115698	5.86898320
PEAK	Upstream	Rigid Veh	2120	64.9210377	6.53503578

The GLM Procedure

Level of PERIOD	Level of CURVE	Level of VEH_CLASS	N	------------SPEED------------ Mean	Std Dev
Before	End	Heavy Trk	537	39.5521415	4.78172439
Before	End	Pass Veh	25756	47.4625874	4.89129191
Before	End	Rigid Veh	2459	46.3923953	5.99353964
Before	Mid	Heavy Trk	507	40.4457594	5.43026186
Before	Mid	Pass Veh	28512	48.3096626	5.21899960
Before	Mid	Rigid Veh	1195	44.6424268	6.45923689
Before	Start	Heavy Trk	703	60.1931721	5.00463136
Before	Start	Pass Veh	28283	63.3498144	5.26265766
Before	Start	Rigid Veh	1220	60.8290164	5.56863306
EARLY_AFTER	End	Heavy Trk	142	39.9021127	4.40355221
EARLY_AFTER	End	Pass Veh	6457	47.3458572	4.57848451
EARLY_AFTER	End	Rigid Veh	460	45.5730435	4.94338442
EARLY_AFTER	Mid	Heavy Trk	148	39.3817568	5.49818816
EARLY_AFTER	Mid	Pass Veh	7026	47.8915172	5.18445795
EARLY_AFTER	Mid	Rigid Veh	297	44.3612795	5.72983677
EARLY_AFTER	Start	Heavy Trk	224	60.9968750	5.94149647
EARLY_AFTER	Start	Pass Veh	6842	63.3081409	5.22702792
EARLY_AFTER	Start	Rigid Veh	289	61.4785467	6.20345641
EARLY_AFTER	Upstream	Heavy Trk	99	60.7262626	4.94773883
EARLY_AFTER	Upstream	Pass Veh	4443	66.5538375	5.75096848
EARLY_AFTER	Upstream	Rigid Veh	534	65.6488764	5.96948688
LATE_AFTER	End	Heavy Trk	941	42.9072264	6.30112881
LATE_AFTER	End	Pass Veh	36777	48.3929140	5.05588201
LATE_AFTER	End	Rigid Veh	4010	47.7066334	5.98712645
LATE_AFTER	Mid	Heavy Trk	834	39.2615108	5.62693685
LATE_AFTER	Mid	Pass Veh	44698	47.5559734	5.15629971
LATE_AFTER	Mid	Rigid Veh	1815	43.7474931	6.22671832
LATE_AFTER	Start	Heavy Trk	667	55.9203898	7.72056288
LATE_AFTER	Start	Pass Veh	43078	60.0562050	6.57731410
LATE_AFTER	Start	Rigid Veh	1610	59.6296894	9.10385288
LATE_AFTER	Upstream	Heavy Trk	470	59.6117021	6.10843481
LATE_AFTER	Upstream	Pass Veh	25302	63.9678602	5.91888914
LATE_AFTER	Upstream	Rigid Veh	1989	64.6253896	6.88272075

Level of PEAK	Level of PERIOD	Level of CURVE	Level of VEH_CLASS	N	------------SPEED------------ Mean	Std Dev
OFF_PEAK	Before	End	Heavy Trk	30	37.7133333	5.42743563
OFF_PEAK	Before	End	Pass Veh	3288	46.8461375	5.28671326
OFF_PEAK	Before	End	Rigid Veh	237	46.1894515	5.58055934
OFF_PEAK	Before	Mid	Heavy Trk	22	38.7363636	5.47283729
OFF_PEAK	Before	Mid	Pass Veh	3645	47.4286420	5.45946016
OFF_PEAK	Before	Mid	Rigid Veh	75	43.3240000	6.70479538

OFF_PEAK	Before	Start	Heavy Trk	54	58.9740741	5.72584954
OFF_PEAK	Before	Start	Pass Veh	3968	62.3391129	5.58617865
OFF_PEAK	Before	Start	Rigid Veh	100	58.4030000	5.56370734

The GLM Procedure

Level of PEAK	Level of PERIOD	Level of CURVE	Level of VEH_CLASS	N	------------SPEED------------ Mean	Std Dev
OFF_PEAK	EARLY_AFTER	End	Heavy Trk	9	40.9777778	6.51378879
OFF_PEAK	EARLY_AFTER	End	Pass Veh	1020	47.2763725	4.79432939
OFF_PEAK	EARLY_AFTER	End	Rigid Veh	56	45.0303571	5.22340605
OFF_PEAK	EARLY_AFTER	Mid	Heavy Trk	11	39.5454545	6.95907517
OFF_PEAK	EARLY_AFTER	Mid	Pass Veh	1082	47.5817006	5.52317945
OFF_PEAK	EARLY_AFTER	Mid	Rigid Veh	32	42.1781250	6.52245424
OFF_PEAK	EARLY_AFTER	Start	Heavy Trk	26	62.2884615	6.94252559
OFF_PEAK	EARLY_AFTER	Start	Pass Veh	1161	62.8661499	5.90784981
OFF_PEAK	EARLY_AFTER	Start	Rigid Veh	35	59.0257143	6.91572559
OFF_PEAK	EARLY_AFTER	Upstream	Heavy Trk	10	63.7500000	3.05150236
OFF_PEAK	EARLY_AFTER	Upstream	Pass Veh	824	66.4321602	6.51894685
OFF_PEAK	EARLY_AFTER	Upstream	Rigid Veh	82	65.7731707	6.80604395
OFF_PEAK	LATE_AFTER	End	Heavy Trk	95	42.5610526	6.30084923
OFF_PEAK	LATE_AFTER	End	Pass Veh	6532	47.7502909	5.33555070
OFF_PEAK	LATE_AFTER	End	Rigid Veh	551	47.2820327	6.08414812
OFF_PEAK	LATE_AFTER	Mid	Heavy Trk	72	38.3694444	5.16566778
OFF_PEAK	LATE_AFTER	Mid	Pass Veh	7505	46.6379347	5.40380317
OFF_PEAK	LATE_AFTER	Mid	Rigid Veh	216	41.3527778	6.23395740
OFF_PEAK	LATE_AFTER	Start	Heavy Trk	62	53.1403226	7.97511416
OFF_PEAK	LATE_AFTER	Start	Pass Veh	7054	59.0179047	6.40188908
OFF_PEAK	LATE_AFTER	Start	Rigid Veh	181	56.4055249	9.19749195
OFF_PEAK	LATE_AFTER	Upstream	Heavy Trk	59	59.1101695	7.21121480
OFF_PEAK	LATE_AFTER	Upstream	Pass Veh	4599	63.1491629	6.16952046
OFF_PEAK	LATE_AFTER	Upstream	Rigid Veh	321	64.0822430	7.72421695
PEAK	Before	End	Heavy Trk	507	39.6609467	4.72439777
PEAK	Before	End	Pass Veh	22468	47.5527995	4.82423424
PEAK	Before	End	Rigid Veh	2222	46.4140414	6.03669859
PEAK	Before	Mid	Heavy Trk	485	40.5232990	5.42122526
PEAK	Before	Mid	Pass Veh	24867	48.4388024	5.17032911
PEAK	Before	Mid	Rigid Veh	1120	44.7307143	6.43591100
PEAK	Before	Start	Heavy Trk	649	60.2946071	4.93131389
PEAK	Before	Start	Pass Veh	24315	63.5147522	5.18942545
PEAK	Before	Start	Rigid Veh	1120	61.0456250	5.51990132
PEAK	EARLY_AFTER	End	Heavy Trk	133	39.8293233	4.24943458
PEAK	EARLY_AFTER	End	Pass Veh	5437	47.3588928	4.53718717
PEAK	EARLY_AFTER	End	Rigid Veh	404	45.6482673	4.90535960
PEAK	EARLY_AFTER	Mid	Heavy Trk	137	39.3686131	5.39554217
PEAK	EARLY_AFTER	Mid	Pass Veh	5944	47.9479139	5.11886269
PEAK	EARLY_AFTER	Mid	Rigid Veh	265	44.6249057	5.58279134
PEAK	EARLY_AFTER	Start	Heavy Trk	198	60.8272727	5.79610088
PEAK	EARLY_AFTER	Start	Pass Veh	5681	63.3984686	5.07250766
PEAK	EARLY_AFTER	Start	Rigid Veh	254	61.8165354	6.03591050
PEAK	EARLY_AFTER	Upstream	Heavy Trk	89	60.3865169	5.01536709
PEAK	EARLY_AFTER	Upstream	Pass Veh	3619	66.5815419	5.56193747
PEAK	EARLY_AFTER	Upstream	Rigid Veh	452	65.6263274	5.81300185
PEAK	LATE_AFTER	End	Heavy Trk	846	42.9460993	6.30369875
PEAK	LATE_AFTER	End	Pass Veh	30245	48.5317011	4.98264673

The GLM Procedure

Level of PEAK	Level of PERIOD	Level of CURVE	Level of VEH_CLASS	N	------------SPEED------------ Mean	Std Dev
PEAK	LATE_AFTER	End	Rigid Veh	3459	47.7742700	5.96962889
PEAK	LATE_AFTER	Mid	Heavy Trk	762	39.3458005	5.66445815

PEAK	LATE_AFTER	Mid	Pass Veh	37193	47.7412201	5.08492022
PEAK	LATE_AFTER	Mid	Rigid Veh	1599	44.0709819	6.15664591
PEAK	LATE_AFTER	Start	Heavy Trk	605	56.2052893	7.64379624
PEAK	LATE_AFTER	Start	Pass Veh	36024	60.2595187	6.59208700
PEAK	LATE_AFTER	Start	Rigid Veh	1429	60.0380686	9.01319407
PEAK	LATE_AFTER	Upstream	Heavy Trk	411	59.6836983	5.94009131
PEAK	LATE_AFTER	Upstream	Pass Veh	20703	64.1497271	5.84637034
PEAK	LATE_AFTER	Upstream	Rigid Veh	1668	64.7299161	6.70619055

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residual                                                  Cum.              Cum.
Midpoint                                          Freq    Freq  Percent  Percent
         ,
   -42.0 ,                                           1       1     0.00     0.00
   -41.2 ,                                           0       1     0.00     0.00
   -40.4 ,                                           0       1     0.00     0.00
   -39.6 ,                                           0       1     0.00     0.00
   -38.8 ,                                           0       1     0.00     0.00
   -38.0 ,                                           0       1     0.00     0.00
   -37.2 ,                                           0       1     0.00     0.00
   -36.4 ,                                           2       3     0.00     0.00
   -35.6 ,                                           0       3     0.00     0.00
   -34.8 ,                                           1       4     0.00     0.00
   -34.0 ,                                           1       5     0.00     0.00
   -33.2 ,                                           2       7     0.00     0.00
   -32.4 ,                                           3      10     0.00     0.00
   -31.6 ,                                           3      13     0.00     0.00
   -30.8 ,                                           2      15     0.00     0.01
   -30.0 ,                                           5      20     0.00     0.01
   -29.2 ,                                           4      24     0.00     0.01
   -28.4 ,                                           3      27     0.00     0.01
   -27.6 ,                                           4      31     0.00     0.01
   -26.8 ,                                           8      39     0.00     0.01
   -26.0 ,                                           8      47     0.00     0.02
   -25.2 ,                                          11      58     0.00     0.02
   -24.4 ,                                          13      71     0.00     0.03
   -23.6 ,                                          24      95     0.01     0.03
   -22.8 ,                                          21     116     0.01     0.04
   -22.0 ,                                          30     146     0.01     0.05
   -21.2 ,                                          36     182     0.01     0.07
   -20.4 ,                                          52     234     0.02     0.08
   -19.6 ,                                          67     301     0.02     0.11
   -18.8 ,                                         102     403     0.04     0.14
   -18.0 ,                                         138     541     0.05     0.19
   -17.2 ,                                         193     734     0.07     0.26
   -16.4 ,*                                        321    1055     0.12     0.38
   -15.6 ,*                                        412    1467     0.15     0.53
   -14.8 ,*                                        550    2017     0.20     0.72
   -14.0 ,*                                        661    2678     0.24     0.96
   -13.2 ,**                                       922    3600     0.33     1.29
   -12.4 ,**                                       993    4593     0.36     1.65
   -11.6 ,***                                     1351    5944     0.49     2.14
   -10.8 ,****                                    1836    7780     0.66     2.80
   -10.0 ,*****                                   2275   10055     0.82     3.61
    -9.2 ,******                                  3071   13126     1.10     4.72
    -8.4 ,********                                4125   17251     1.48     6.20
    -7.6 ,***********                             5535   22786     1.99     8.19
    -6.8 ,**************                          6886   29672     2.47    10.66
    -6.0 ,******************                      8911   38583     3.20    13.86
    -5.2 ,*********************                  10407   48990     3.74    17.60
    -4.4 ,************************               12010   61000     4.32    21.92
    -3.6 ,*****************************          14465   75465     5.20    27.11
    -2.8 ,*******************************        15347   90812     5.51    32.63
    -2.0 ,*********************************      16683  107495     5.99    38.62
    -1.2 ,**********************************     16977  124472     6.10    44.72
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 -0.4  ,***********************************   17602  142074    6.32    51.05
  0.4  ,*************************************  18363  160437    6.60    57.64
  1.2  ,*********************************     16604  177041    5.97    63.61
  2.0  ,******************************        15033  192074    5.40    69.01
  2.8  ,****************************          13920  205994    5.00    74.01
  3.6  ,***************************           13430  219424    4.83    78.84
  4.4  ,*********************                 10549  229973    3.79    82.63
  5.2  ,*****************                      8714  238687    3.13    85.76
  6.0  ,*****************                      8446  247133    3.03    88.79
  6.8  ,*************                          6405  253538    2.30    91.09
  7.6  ,**********                             4789  258327    1.72    92.82
  8.4  ,*********                              4399  262726    1.58    94.40
  9.2  ,*******                                3478  266204    1.25    95.65
 10.0  ,*****                                  2728  268932    0.98    96.63
 10.8  ,****                                   1989  270921    0.71    97.34
 11.6  ,***                                    1551  272472    0.56    97.90
 12.4  ,***                                    1352  273824    0.49    98.38
 13.2  ,**                                      936  274760    0.34    98.72
 14.0  ,*                                       720  275480    0.26    98.98
 14.8  ,*                                       544  276024    0.20    99.17
 15.6  ,*                                       497  276521    0.18    99.35
 16.4  ,*                                       409  276930    0.15    99.50
 17.2  ,*                                       285  277215    0.10    99.60
 18.0  ,                                        209  277424    0.08    99.68
 18.8  ,                                        208  277632    0.07    99.75
 19.6  ,                                        159  277791    0.06    99.81
 20.4  ,                                        124  277915    0.04    99.85
 21.2  ,                                        108  278023    0.04    99.89
 22.0  ,                                         62  278085    0.02    99.91
 22.8  ,                                         53  278138    0.02    99.93
 23.6  ,                                         43  278181    0.02    99.95
 24.4  ,                                         32  278213    0.01    99.96
 25.2  ,                                         15  278228    0.01    99.97
 26.0  ,                                         19  278247    0.01    99.97
 26.8  ,                                         15  278262    0.01    99.98
 27.6  ,                                         16  278278    0.01    99.98
 28.4  ,                                         10  278288    0.00    99.99
 29.2  ,                                          8  278296    0.00    99.99
 30.0  ,                                         12  278308    0.00    99.99
 30.8  ,                                          7  278315    0.00   100.00
 31.6  ,                                          3  278318    0.00   100.00
 32.4  ,                                          0  278318    0.00   100.00
 33.2  ,                                          1  278319    0.00   100.00
 34.0  ,                                          3  278322    0.00   100.00
 34.8  ,                                          2  278324    0.00   100.00
       ,
       Šƒƒƒƒƒƒƒˆƒƒƒƒƒƒƒˆƒƒƒƒƒƒƒˆƒƒƒƒƒƒƒˆƒƒƒƒƒ
              4000    8000    12000   16000
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