Dynamic Variable Speed Limit Strategies for Real-Time Crash Risk Reduction on Freeways

Mohamed Abdel-Aty, Ryan J. Cunningham, Vikash V. Gayah, and Liang Hsia

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Recent research at the University of Central Florida has focused on increasing safety on the I-4 corridor through Orlando, Florida. This study investigated the benefits of variable speed limit (VSL) implementation for real-time crash risk reduction. VSL has been used in various nations around the world (including the United Kingdom, the Netherlands, Germany, Australia, New Zealand, and even the United States) to decrease the intensity of traffic congestion (*1*) as well as for weather advisory scenarios (*2, 3*). VSL has also been used to reduce the occurrence of crashes in some areas (*4*).

In the Netherlands, Van de Hoogen and Smulders used VSLs to homogenize the traffic flow on a 20-km section of the six-lane A2 Motorway connecting Amsterdam and Utrecht (*1*). The primary function of the control was to decrease not average speed but rather speed differences on the freeway. According to the authors, decreasing speed differences implies more regulated headways and a reduction in the number and severity of shockwaves. The authors found that the implementation of variable speed limits provided calmer traffic and improved the distribution of the traffic over the available road space.

In the United States, VSLs are used in Washington State, New Jersey, New Mexico, and Wisconsin to identify and implement a safe speed limit when inclement weather converges on vulnerable freeways (*3*). The systems in these states have been successful in detecting dangerous roadway conditions and suggesting a safe speed to the drivers. They have been effective in reducing average speed in dangerous conditions and decreasing the risk of crashes on the road.

In the United Kingdom, mandatory variable speed limits are used to improve traffic flow and safety on the M25 London Orbital Motorway. Their incident detection system, Motorway Incident Detection and Automatic Signaling (MIDAS), monitors traffic flow and speeds to adjust the speed limit to avoid or delay congestion when the system is operating near capacity. MIDAS is also used to detect queuing behavior and prevent secondary accidents at the back of queues as they form. Borrough reported that the enforcement of VSLs effectively decreased the number of crashes on the M25 (*4*).

In this study, VSL was investigated using a microsimulation program to analyze its ability to reduce crash risk in real time. Unlike previous studies, this research applied VSL in a uniquely dynamic approach through a networkwide strategy. The concept of homogeneous speed zones was introduced, and VSL was made available for application at every station over a dynamic distance defined by the traffic flow. The objectives of this study were to examine multiple strategies for the implementation of VSLs and to recommend the optimum treatments for use on urban freeways under various loading conditions.

PREVIOUS STUDIES ON REAL-TIME CRASH RISK REDUCTION

Several studies have been conducted to examine the effects of ITS strategies on real-time crash risk, including the implementation of variable speed limits. Park and Yadlapati found that implementing VSL in work zones reduced the speed variation throughout the work zone area (*5*). Lee et al. performed a study in which VSL was applied to a freeway network using the PARAMICS microsimulation program (*6*). This study showed the effectiveness of VSL at reducing the crash potential; however, this study was extremely limited in scope, did not use real traffic data, and focused on only raising or lowering the speed limit, but never both simultaneously.

Abdel-Aty et al. (*7*) expanded on Lee's study by simulating a 20-mi freeway in PARAMICS, which allowed for more flexibility in the implementation of VSL. This study showed that simultaneously raising the speed limit downstream while reducing the speed limit

M. Abdel-Aty, R. J. Cunningham, and V. V. Gayah, Department of Civil and Environmental Engineering, University of Central Florida, Orlando, FL 32816-2450. L. Hsia, Florida Department of Transportation, 605 Suwannee Street MS 90, Tallahassee, FL 32399-0450. Corresponding author: M. Abdel-Aty, mabdel@mail.ucf.edu.

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upstream had the greatest safety benefits. It was also found that VSL was applicable only when the freeway was operating at higher speed (uncongested) conditions. During congested conditions, VSL was found to have no effect on the crash risk because the freeway was already operating beneath the speed limit and, therefore, altering the speed limit had little to no affect on the traffic flow. Abdel-Aty et al. noted that when VSL was applied, there was potential for crash risk migration or the lowering of the crash risk at one location coupled with the increase in the crash risk at another location (*8*). Abdel-Aty and Dhindsa combined VSL with ramp metering and proved that this combination held great safety potential (*9*). These previous studies all used a measure of crash-risk developed by Abdel-Aty et al. (*10*).

STUDY AREA

For this work, the PARAMICS microsimulation software was used to model a 36.25-mi segment of I-4 in Orlando, Florida. Throughout the downtown area, the freeway varies between 6 and 8 lanes with 12 ft width and a speed limit of 50 mph. Outside the downtown region, the freeway is typically six-lanes with speed limits as high as 65 mph. The composite annual average daily traffic (AADT) of I-4 through this area as given by the Florida Department of Transportation is 183,000 veh/day (*11*). The segment of the freeway that is modeled is fitted with dual inductance loop detectors at 0.5-mi intervals throughout its length in the field. These detectors yield measures of the speed, lane occupancy, and volume on each of the three mainline lanes at 30-s intervals. Loop detector data has been archived at the University of Central Florida between 1998 and 2004. These data were extracted and compared with the simulation to both calibrate and validate the simulation with respect to the field conditions. These loop detector data were also used in statistical models to create the crash risk measures used in this study.

MICROSIMULATION

Traffic simulation software packages have been used with increasing frequency to examine the effects of various intelligent transportation systems (ITS) on traffic flow. Simulation software allows researchers to examine multiple strategies, in ways that cannot possibly be tested in the field. They also allow for optimization of ITS strategies because multiple implementation methods can be tested before the most beneficial strategy is determined. The PARAMICS microsimulation program was selected because of its proven reliability on urban freeways and its use in previous works examining variable speed limits and real-time crash risk (*6, 7, 9*).

Before the simulated network could be used to examine the safety potential of variable speed limits, the network was first calibrated and then validated. The calibration procedure involved optimizing built-in calibration parameters that define how the vehicles behave in the network to ensure that the simulated vehicles mimic vehicular behavior in the field. The four major calibration parameters in PARAMICS are the mean driver headway, mean driver reaction time, queue speed (maximum speed boundary defining queuing conditions), and queue distance (maximum distance between two vehicles that define queuing conditions). The values of the calibration parameters were obtained from a previous study performed by Dhindsa (*12*) in which an extensive calibration procedure was performed for a smaller (20-mi) section of I-4, which is contained within this study area. The calibration done by Dhindsa (*12*) considered 5-min flows and 5-min speeds along the freeway, which makes his calibration procedure more detailed than most; on the basis of a review of available literature, other networks usually consider only hourly flows for their calibration procedures, and many do not even consider speed at all. The values found in Dhindsa's calibration resulted in realistic vehicular behavior, and they were deemed acceptable for this study because the study area is approximately the same, and the same simulation program was used to create both networks. The values of the calibration parameters are a queue speed of 8 mph, queue distance of 9 ft, mean driver headway of 1.0 s, and mean driver reaction time of 0.45 s.

The validation of the network was performed by comparing the simulated hourly flows at specific loop detector stations with real data extracted from the field. The hourly volumes were compared by calculating the GEH statistic (named after Geoff E. Havers), which considers both the relative and absolute difference between the simulated and observed volumes at each location. This measure is widely used by researchers working with PARAMICS and provides an estimate of how well the simulated data matches the real field data (*13*). Approximately 80% of the locations examined had a GEH statistic less than 5, which indicates an excellent representation of the field data; the remaining stations all had a GEH statistic less than 10, which means that the flows at these locations were not unreasonable.

MEASURE OF SAFETY

Traffic crashes are very complex events that involve numerous human and environmental factors in addition to traffic and roadway conditions. For this reason, microsimulation software cannot be used to directly measure crashes or safety. Therefore, a surrogate measure needs to be used when a simulation program is used to assess traffic safety. A surrogate measure of safety is a directly measurable variable that has a known relationship with traffic crashes. Typical surrogate measures of safety include speed, speed variance, time to collision, or postencroachment time (*14*). Some researchers have also developed statistical models using directly measurable variables to assess the risk of a collision on a roadway. One such example is research that created a simple model that compares the safe following distance of a vehicle to the actual following distance (*5*). Because following a vehicle too closely has a known relationship with rear-end crashes, this measure can be used as a surrogate measure of safety for rear-end incidents.

Another, more complex, surrogate measure of safety comes from models created by Abdel-Aty et al. (*10*), which describe the risk of a crash occurring on an urban freeway using logistic regression and real-time loop detector data. These models were created using the loop data taken from the same study area (I-4) used in this research, which make them applicable to this work. However, the drawback to using these models is that the crash risk values that are outputted are spatially independent. The models control for geometry or spatial input variables, and, because of this, the crash risk values cannot be compared across different locations. This means that when these models are used, there is no way to determine which areas along the freeway have the highest crash risk values and, therefore, which areas need the application of crash prevention strategies the most. Newer models created by Pande and Abdel-Aty using neural networks include factors to account for the location and geometry along the freeway and yield separate values for rear-end and lane-change crash risks (*15, 16*). These models use the 30-s loop data that has been aggregated over 5-min intervals and across the three lanes of the freeway to reduce the natural noise and variation within the data. The real-time measures considered are average speed, coefficient of variation of speed (the standard deviation divided by the average), standard deviation of speed, average occupancy, standard deviation of occupancy, average volume, and standard deviation of volume all taken either at the station of interest or at locations up to 1 mi upstream or downstream of the station of interest. These values are all calculated for the period of time 5 to 10 min before the time of interest, which means that if the models are implemented in real time, they will become predictive and give the crash risk for a time period 5 min in the future. This will allow for the implementation of a crash prevention strategy in real time to help reduce the crash risk before a potential crash occurs.

In the research by Pande and Abdel-Aty (*15*), rear-end crashes were determined to occur within one of two distinct traffic regimes: Regime 1 (congestion conditions) or Regime 2 (uncongested conditions). Separate models were created to determine the crash risk for each of the traffic regimes. The Regime 1 model used traffic data located at the location (loop detector station) of interest only. This was done because in the congested situation the traffic conditions do not vary much up to 1 mi upstream or downstream of the location of interest. Therefore, using traffic data from other nearby locations does little to improve the accuracy of the model. In this traffic regime, the average occupancy is the most important variable affecting the rear-end crash risk, because higher occupancies increase the risk of such a crash occurring in congestion situations. The Regime 2 model uses traffic information at the station of interest as well as up to 1 mi upstream and downstream of that location. In this model, the speed differential is very important because the crash risk is increased when faster moving vehicles approach slower moving vehicles. Therefore, average speeds at the location of interest and both upstream and downstream of this location are important to determine whether there is a large speed differential that would increase the risk of a rear-end collision. The outputs of the two models were normalized (by subtracting the mean and dividing by the standard deviation forcing the normalized mean to be 0 and the normalized standard deviation to be 1) and then combined using the probability that the respective traffic conditions belonged to Regime 1 or Regime 2. This yielded a single metric for the real-time rear-end crash risk along the freeway that was taken from the output of the two different regime models.

For the lane-change crash risk (*16*), only one neural network model was developed; however, the output was normalized using the previously described method so that the rear-end and lane-change crash risk metrics were on comparable scales. The main factors affecting the lane-change crash risk are the average speeds upstream and downstream of the station of interest as well as the difference in the lane occupancies across each individual lane on the freeway. The higher the absolute difference in the lane occupancy across adjacent lanes on the freeway is, the higher the chance of having a lane-change related crash is because this means that more vehicles will be switching lanes.

MEASURES OF EFFECTIVENESS

Using the aforementioned crash risk models, a value of the rear-end and lane-change crash risk was determined for every 5-min period at every location along the freeway for each of the simulation runs. When multiple different scenarios are simulated, plots of the crash risk versus time and crash risk versus location can be created to assess the scenario that has the lowest real-time crash risk value. However, graphical comparisons were not sufficient at determining the best strategy, so numerical measures of effectiveness (MOEs) were also used to help determine the strategies that reduce the crash risk the most.

The primary MOEs in this study are the overall risk change index (ORCI) for the rear-end and lane-change crash risks. These measures denote the change in the rear-end and lane-change crash risk, respectively, between any particular test case and the base case. The MOEs are calculated in the following manner. First, the crash risk is calculated for each 5-min period at every location. Second, the crash risk at each location is averaged over the entire simulation length (3-h simulation = thirty-six 5-min crash risk values) at every location. Next, a plot of the average crash risk value versus location was created for the base case and the test case. The area between the two crash risk curves represents the ORCI. This measure is shown more clearly in Equation 1. A positive value of the ORCI indicates that the overall change across the network is an increase in the crash risk, whereas a negative value shows a decrease in the crash risk (improved safety conditions).

$$\text{ORCI} = \sum_{l}\left[\frac{1}{T}\sum_{t=1}^{T}\left(\text{risk\_profile}\right)_{tl}\right]_{\text{base}} - \sum_{l}\left[\frac{1}{T}\sum_{t=1}^{T}\left(\text{risk\_profile}\right)_{tl}\right]_{\text{Test}} \tag{1}$$

where $(\text{risk\_profile})_{tl}$ is the average rear-end crash risk at time t and Station 1, and T is the number of time periods in the simulation run (36).

The other measure of effectiveness that is considered in this study is the overall network travel time. The travel time was included as a measure of effectiveness to take into account the operational effects of changing the speed limit in real time.

SPEED DIFFERENCE

One of the most important factors in the crash risk models is the variance of speed between the station of interest and the nearest station upstream. This measure is referred to as the speed difference. The speed difference is defined as the difference between the 5-min average speed at the station upstream and the 5-min average speed at the station of interest and is described by Equation 2.

$$\text{speeddiff} = \text{avgspeed}_E - \text{avgspeed}_F \tag{2}$$

Loop detector stations exist every ½ mi on the network and are numbered in order. The station of interest is always referred to as Station F, and upstream stations take on relative symbols, such as Station E located a ½ mi upstream and Station D located 1 mi upstream from the station of interest. Stations downstream also take on relative symbols, such as Station G located a ½ mi downstream and Station H located 1 mi downstream from Station F.

The measure of speed difference is the primary stimulation needed to implement VSL in this study. However, there is a question as to how great a speed difference demands the implementation of variable speed limits to reduce the crash risk and homogenize average speeds. Exploratory analysis was performed to determine the critical speed difference at which VSL should be implemented, according to its effect on crash risk. Two locations were chosen at which to examine the effects of speed difference on the crash risk measures. It was shown in both locations that a speed difference of 7 mph between stations is the critical point at which crash risks begin to increase substantially. Therefore, a critical speed difference of 7 mph was used as the threshold for variable speed limit implementation throughout this study. For more information about the procedure to determine the critical speed difference, the reader is referred to Cunningham (*17*).

HOMOGENEOUS SPEED ZONES

Previous VSL studies have typically used sets of fixed distances over which VSL would be applied (*6, 7, 9*). A fixed distance is hard to justify as the best option, however, considering the dynamic characteristics of traffic flows on freeways. For instance, a backward-forming queue may necessitate decreased speeds upstream and increased speeds downstream. It may do no good to increase speed limits 2 mi downstream if the queue is, say, 5 mi long. For this study, then, a dynamic distance was considered for VSL application. As far as the authors know, this is the first time a dynamic distance has been considered for VSL application.

To include a dynamic application distance in the VSL strategy, the concept of homogeneous speed zones was introduced. A homogeneous speed zone is a collection of similar, contiguous segments of road, based on average speed, and is distinguished from other homogeneous segments. The similarity of these groups is determined by the difference in their average speeds between stations. The entire network, then, is made up of a small number of homogeneous speed zones described by an average speed at the borders of each zone rather than 70 different stations with 70 average speeds. This concept is illustrated in Figure 1*a*. Initially, the highway is split into ½-mile stations. The station number and the distance covered between each station are shown, as well as the 5-min average speed for each station.

The process of defining homogeneous speed zones involves taking the difference of the 5-min average speeds at each station and the station upstream. This measure was earlier referred to as the speed difference. If the speed difference at a station is less than the speed zone threshold, then that station is considered to be part of the same homogeneous speed zone as the next station upstream. However, if the speed difference at the station of interest is greater than the threshold, then that station of interest becomes the first station in a new speed zone. The speed zones threshold in this example was a speed difference of 5 mph. As shown in Figure 1*a*, the freeway, which was once arbitrarily defined by 13 separate ½-mi segments, is now considered three homogeneous speed zones with defined lengths, as well as beginning and ending average speeds. The defining speed zone difference, or the speed difference threshold for the homogeneous speed zones, was considered as a factor in this study to properly analyze the sensitivity of this measure.

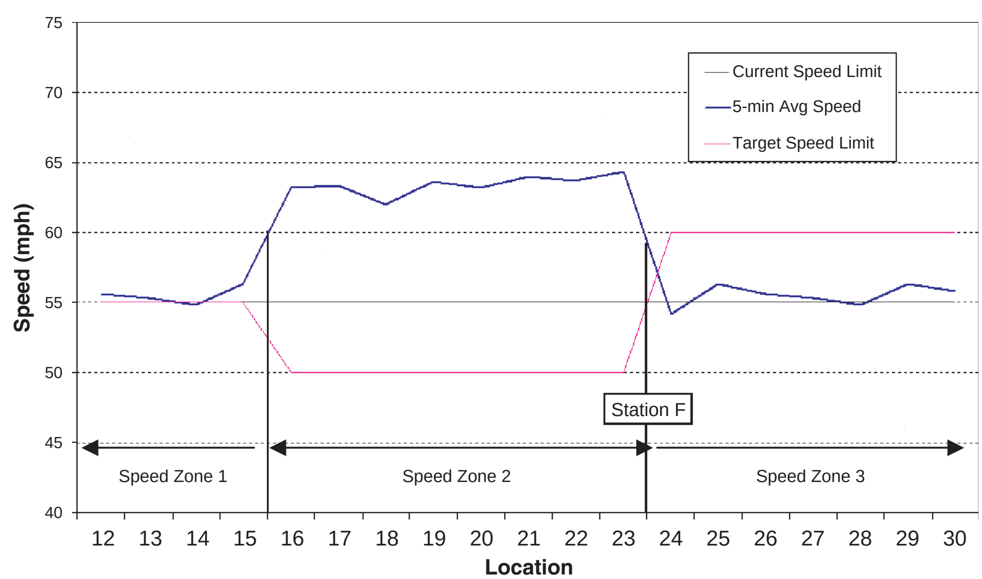
EXPERIMENTAL DESIGN

Several important factors are necessary for properly describing a VSL implementation technique. These factors include the implementation strategy, the defining speed difference of the speed zones, the speed zone multiplier describing the spatial extent over which

Distance	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Station	12	13	14	15	16	17	18	19	20	21	22	23	24
Average Speed	70	72	71	69	64	63	65	66	72	70	69	71	73

Distance	2.0				2.0				2.5				
Station	12	13	14	15	16	17	18	19	20	21	22	23	24
Average Speed	(70, 69)				(64, 66)				(72, 73)				

(a)



(b)

FIGURE 1 (*a*) Typical freeway layout according to stations and homogeneous speed zones and (*b*) VSL implementation on I-4.

speed limits would be changed, and the minimum time period for which the change in speed limits would be sustained. For clarity, the speed zone multiplier is the fraction by which the length of a speed zone is multiplied to give a value to the dynamic distance over which variable speed limits are implemented.

Multiple approaches for deciding the implementation strategy of VSL are available. Some of these approaches include setting the target speed limit to some all-encompassing “safe speed,” setting the target speed limit equal to the average speed of vehicles downstream, or changing the speed limits in specific areas by some predetermined amount. The latter was chosen as the best approach for this study. This provides for smooth speed limits that are divisible by 5 mph and an implementation strategy that is dynamic to the location on the freeway. The minimum and maximum speed limits that can be posted on the freeway must also be considered. I-4 has a minimum speed limit threshold of 40 mph, although this speed limit is posted nowhere on the freeway. The minimum speed limit in the study section is 50 mph. Therefore, the maximum that speed limits can be reduced is 10 mph. A 10-mph reduction in speeds ought to be set as a maximum reduction anyway. It is not reasonable to change the speed limits in an area by 15 mph, especially when the area is traveled heavily by familiar commuters. Two strategies of upstream speed limit lowering were tested. A decrease of 10 mph and a decrease of 5 mph in the upstream speed limits will be investigated in the experimental design.

Another strategy should also be included, that of lowering upstream speed limits and simultaneously raising downstream speed limits. The maximum allowable speed limit on most roadways is the design speed. The design speed ensures adequate stopping sight distance, adequate radii on horizontal curves, and reasonable superelevations, among other factors. Therefore, the posted speed limit should never exceed the design speed on the freeway. The minimum design speed occurs throughout the downtown segment of I-4 at 60 mph. The speed limit in that area is set at 50 mph. Therefore, there should be no increase in speed limits that exceed 10 mph. Increasing speed limits by 10 mph, however, may also be a bit extreme. To allow for reasonable recommendations from this study, the third strategy implemented a decreasing of upstream speed limits by 5 mph coupled with a simultaneous increasing of downstream speed limits by 5 mph.

The next factor must define the threshold speed difference for homogeneous speed zones. Because the threshold speed difference for variable speed limit implementation is 7 mph, as discussed in the previous section, the threshold for speed zones must be less. A natural choice is 5 mph. This would separate the segments with different speed limits into speed zones, and any disruptions greater than 5 mph within a speed limit section also would be separated. However, it may be more desirable to have more defined speed zones, based on a lower threshold. A threshold of 2.5 mph could also be used so that the speed zones that are defined have average speeds within 2.5 mph of each other. When this threshold is exceeded, a new speed zone is defined. Based on this, then, the speed zone threshold variable will have two levels: 5 mph and 2.5 mph.

On the basis of the speed zones, the spatial extent of the VSL implementation can be defined by a multiplier. The idea behind homogeneous speed zones was to capture the stations with similar traffic conditions and change them all together. Therefore, a necessary multiplier for the speed zone is 1.0. If a given speed zone is 5 mi long and it is determined that VSL should be implemented in that area, then a multiplier of 1.0 would allow the entire 5 mi to be effected. It may also be of interest to see the effects that implementation only over a fraction of the speed zone has. In this case, 0.5 is an ideal multiplier. For the scenario previously mentioned, a multiplier of 0.5 would allow speed limits to be changed over the 2.5 mi closest to the location of interest, whereas the remainder of the speed zone is unchanged. The speed zone multiplier, then, will consist of two levels: 1.0 and 0.5.

Finally, the minimum time period for VSL to be extended will be discussed. Previous studies found time periods of 5 min and 10 min to work best (*6*). Longer time periods should be avoided, because their effects may be overstated or exaggerated. Also, to create a treatment that is as dynamic as possible, the most adaptable factors should be used. In this case, the minimum feasible time is 5 min, because the speed differences are analyzed only every 5 min. To maintain highly adaptable time periods, then, the minimum time periods for VSL implementation were investigated at 5 min and 10 min and extended if needed.

Considering these four variables, one at three levels and three at two levels, there are 24 possible treatment combinations. A treatment is referred to here as a unique combination of factors. Each combination of factors is given a unique treatment ID, as shown in Table 1.

Figure 1*b* shows a schematic of how Treatment 17 might work, for instance. In the example, speeds are fairly homogeneous from Stations 12 through 15. At Station 16, however, the 5-min average speed jumps from 56 mph up to approximately 63 mph. This negative speed difference of 7 mph constitutes the creation of a new speed zone, Speed Zone 2, which begins at Station 16. High average speeds are maintained until Station 24, where the speeds go back down to approximately 55 mph. This speed difference also constitutes a new speed zone, Speed Zone 3, which begins at Station 24. When evaluating the speed differences at every station for VSL implementation, Station 24 had a high positive speed difference (10 mph), which warrants VSL application. On the basis of Treatment 17, the speed limit for the whole speed zone upstream (Speed Zone 2) was lowered by 5 mph, and the speed limit for the whole speed zone downstream (Speed Zone 3) was raised by 5 mph. These speed limits would be maintained for 5 min and then reevaluated.

This experimental design of 24 treatments was applied to three different volume loading blocks: 60%, 80%, and 90%. The first scenario refers to free-flow conditions on the freeway, the second scenario refers to conditions approaching congestion before the peak period, and the last scenario refers to congested conditions observed during the peak period.

RESULTS

60% Loading

Each of the 24 treatments was applied to the 60% loading scenario. It was found that the first 16 treatments, which operate by decreasing the speed limits upstream by 10 mph or 5 mph, do not positively affect the overall rear-end or lane-change crash risk on the network. However, positive results were observed when the downstream speed limits were increased and the upstream speed limits were decreased simultaneously by 5 mph. The best results were observed when a liberal speed zone multiplier was used (5 mph) and VSL was implemented over the whole network (multiplier = 1.0). Treatments 17 and 18 represent these values, and they have a time period of 5 min and 10 min, respectively. Using a time period of 10 min was found to be slightly more profitable than using 5 min. Figure 2 shows a segment of significant benefit in rear-end crash risk. Treatments 17 and 18 are shown to reduce the crash risk the greatest, with Treatment 18 being slightly better than Treatment 17.

TABLE 1 Layout of Experimental Design

Treatment ID	Speed Change Implementation	Speed Zone Threshold (mph)	Speed Change Distance	Speed Change Time Period (min)
1	–10	5	Speed zone	5
2	–10	5	Speed zone	10
3	–10	5	½ speed zone	5
4	–10	5	½ speed zone	10
5	–10	2.5	Speed zone	5
6	–10	2.5	Speed zone	10
7	–10	2.5	½ speed zone	5
8	–10	2.5	½ speed zone	10
9	–5	5	Speed zone	5
10	–5	5	Speed zone	10
11	–5	5	½ speed zone	5
12	–5	5	½ speed zone	10
13	–5	2.5	Speed zone	5
14	–5	2.5	Speed zone	10
15	–5	2.5	½ speed zone	5
16	–5	2.5	½ speed zone	10
17	–5/+5	5	Speed zone	5
18	–5/+5	5	Speed zone	10
19	–5/+5	5	½ speed zone	5
20	–5/+5	5	½ speed zone	10
21	–5/+5	2.5	Speed zone	5
22	–5/+5	2.5	Speed zone	10
23	–5/+5	2.5	½ speed zone	5
24	–5/+5	2.5	½ speed zone	10

The differences in crash risk were tested for statistical significance using *t* tests. As shown in Table 2, Treatments 17 and 18 outperformed every other treatment in rear-end and lane-change crash risk reduction. The bolded values indicate where the crash risk reduction was significant. It should also be noted that where crash risk migration appears to be prevalent in the upstream reduction of speed limits (Treatments 1–16), Treatments 17 to 24 appear to be completely resistant to the effects of crash risk migration in the free-flow scenario.

Travel time analysis was also performed for Treatments 17 and 18. The total network travel time, measured in vehicle-hours traveled, was reported directly by PARAMICS in the simulation output. This value was calculated by summing the travel time of each vehicle over

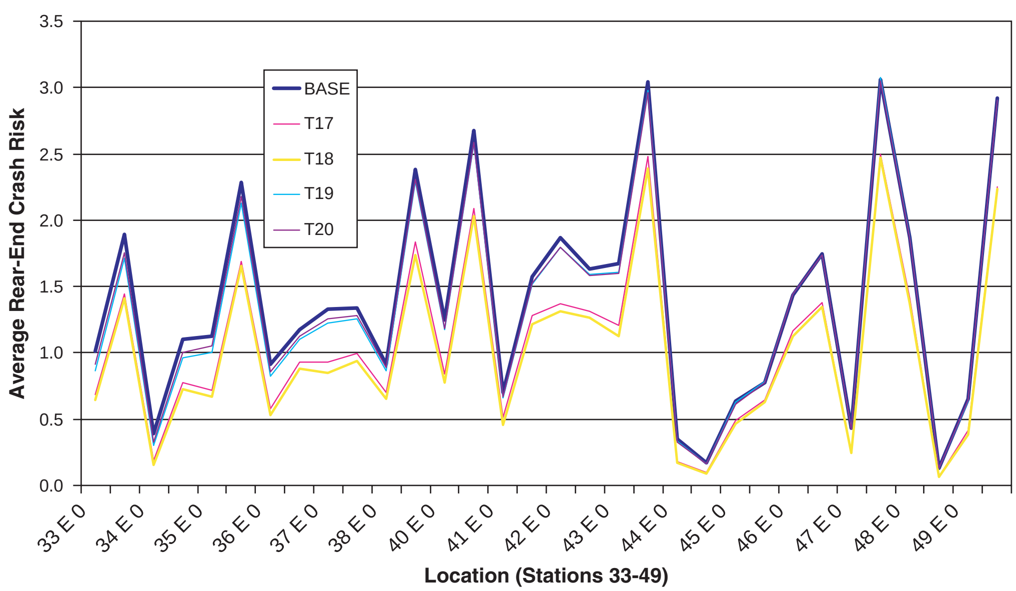


FIGURE 2 Average rear-end crash risk versus location for Treatments 17 to 20.

TABLE 2 Summary of Rear-End and Lane-Change ORCI Values for 60% Loading, Treatments 17 to 24

	Test Case ID							
	T17	T18	T19	T20	T21	T22	T23	T24
Rear-end ORCI			1.894	1.540	9.256	8.336	–0.118	0.446
No. stations affected	16	16	16	16	16	16	16	16
Lane-change ORCI				8.564			6.082	5.710
No. stations affected	38	38	38	38	38	38	38	38

the entire network and the whole simulation time. In the free-flow scenario, Treatments 17 and 18 actually reduced the network travel time. Treatment 17 reduced the travel time by 0.4%, and Treatment 18 was found to significantly reduce the travel time by 0.8%.

80% Loading

The 24 treatments were also applied to the 80% loading scenario, which simulated conditions approaching congestion. Again it was observed that simply decreasing the speed limits upstream by 5 or 10 mph did not effectively reduce the crash risk. It was also seen that VSL affected the corridor in positive and negative segments in the 80% loading scenario. Typically, the negative segments occurred upstream, and the positive segments occurred downstream. The upstream, negatively affected segments were the results of crash migration, which was found to be much more prevalent in conditions approaching congestion. The best treatment, Treatment 19, had the highest cumulative ORCI in the rear-end and lane-change crash risk analyses. Treatment 19 used the technique of simultaneously reducing speed limits upstream and increasing speed limits downstream. It defined homogeneous speed zones by a 5 mph threshold and applied speed limit changes over half the speed zone distances for a minimum time period of 5 min. Its high ORCI values are due in part to its superiority in resisting the effects of crash migration.

In fact, the treatments that implemented speed limit changes over half the speed zone distance (Treatments 19, 20, 23, and 24) were found to resist the effects of crash migration much better than other treatments. This is shown clearly in Figure 3, where the difference in rear-end crash risks for Treatments 17 to 24 from the base case were compared for in the 60% and 80% loading scenarios. The light yellow cells represent a positive change (improvement) in crash risk from the base case, and the dark grey cells represent a negative change in crash risk from the base case, which is evidence of crash migration. It can be seen that in the 60% loading scenario, these treatments are fully resistant to crash migration. However, in the 80% loading scenario, the effects of crash migration appear to be stronger, and only the treatments mentioned earlier resist them well.

Table 3 shows the rear-end and lane-change cumulative ORCI values for Treatments 17 to 24. The cumulative ORCI is simply the sum of ORCI values across affected sections. For example, Treatment 17 was affected negatively from Stations 14 to 31 but positively from Stations 33 to 44. These segments were analyzed separately, but the ORCI values from each segment were summed to obtain the cumulative ORCI. In this way, the cumulative ORCI value takes into account the negative effects of crash migration when comparing multiple treatments.

It is shown in the table that none of these treatments were found to significantly reduce the rear-end crash risk in the 80% loading scenario, but all of them were able to significantly reduce the lane-change crash risk. Treatment 19 had the highest cumulative ORCI value for both crash risks and in no way significantly affected the crash risk in a negative way. It is, therefore, the best treatment for conditions approaching congestion.

Travel time analysis was also performed for Treatment 19 in the 80% loading scenario. It was found that Treatment 19 increased the network travel time by approximately 60 vehicle-hours traveled, which is an increase of less than 0.4%. This small increase is deemed acceptable for the beneficial trade-off with safety.

90% Loading

In the congested scenario, no treatments were found to have a positive ORCI in the rear-end or lane-change crash risk analyses. This is because of the nature of the traffic flow in the 90% loading scenario. In congested situations, the speeds of vehicles are mostly determined by the traffic conditions as opposed to the speed limit. Varying the speed limit, therefore, will not have the desired effect, because the vehicles are subject more to the congestion than to the speed limit. These findings concur with past studies on variable speed limits in congested situations, such as Abdel-Aty et al. (*7*).

CONCLUSION

This study found that the implementation of variable speed limits successfully reduced the rear-end and lane-change crash risks at low-volume traffic conditions (60% and 80% loading conditions). In every case, the most successful treatments involved the lowering of upstream speed limits by 5 mph and the raising of downstream speed limits by 5 mph. In the free-flow condition (60% loading), the best treatment (Treatment 18) involved the more liberal threshold for homogeneous speed zones (5 mph) and the more liberal implementation distance (the entire speed zone) for a minimum time period of 10 min. Treatment 18 was actually shown to reduce the network travel time by a fraction of a percent. It was also shown that this particular implementation strategy (lowering upstream, raising downstream) is wholly resistant to the effects of crash migration in free-flow conditions.

In the condition approaching congestion (80% loading), the best treatment (Treatment 19) again involved the more liberal threshold for homogeneous speed zones (5 mph) but the more conservative implementation distance (half the speed zone) and the minimum time period of 5 min. This particular treatment arises as the best because of its unique capability to resist the effects of crash migration in the 80% loading scenario. It was shown that the treatments implementing more than half the speed zone (Treatment 19, Treatment 20, Treatment 21, Treatment 22) were more robust against crash migra-

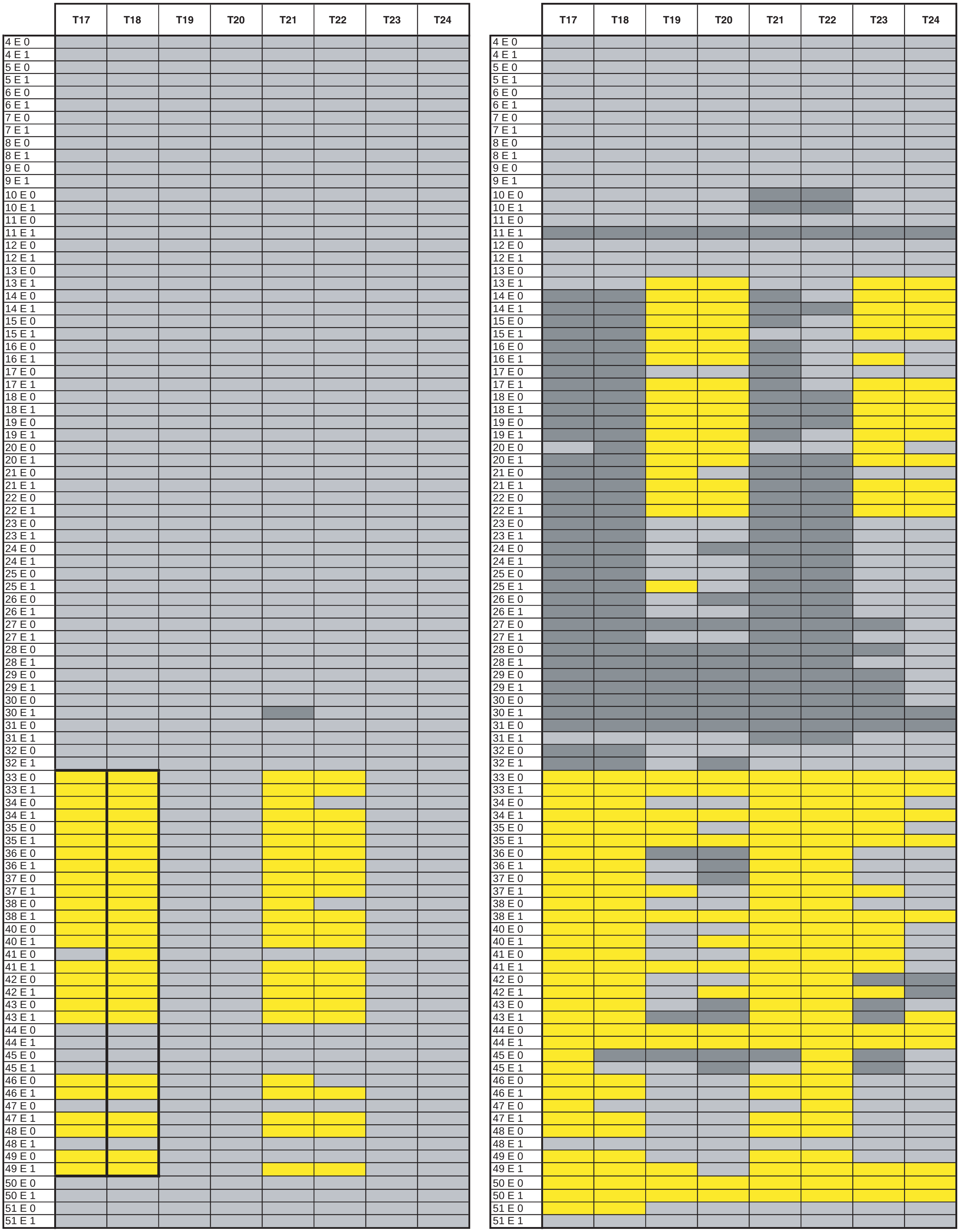


FIGURE 3 Comparison of difference in rear-end crash risk for 60% and 80% loading scenarios.

TABLE 3 Summary of Rear-End and Lane-Change Cumulative ORCI Values for 80% Loading, Treatments 17 to 24

	Test Case ID							
	T17	T18	T19	T20	T21	T22	T23	T24
Rear-end ORCI	1.017	0.406	1.196	0.858	0.327	0.869	1.112	0.770
No. stations affected	29	29	29	29	29	29	29	29
Lane-change ORCI								
No. stations affected	36	36	36	36	36	36	36	36

tion than the other treatments. Treatment 19 exemplified the greatest benefit in reduced sections and the greatest resistance to crash migration in other sections. This treatment was found to increase the network travel time by less than 0.4%, which was deemed acceptable.

Finally, no treatment was found to successfully reduce the rear-end and lane-change crash risks in the congested traffic condition (90% loading). This is attributed to the fact that, in the congested state, the speed of vehicles is subject to the surrounding traffic conditions and not to the posted speed limit. Therefore, changing the posted speed limit does not affect the speed of vehicles in a desirable way. However, it was shown that the effects of crash migration are more prevalent in the congested situation than in the previous conditions, confirming that the effects of crash migration increase as traffic volume increases. It was also confirmed that the treatments implementing speed limit changes upstream and downstream over half the length of the speed zones, although they were unable to effectively reduce the rear-end and lane-change crash risks, were more resistant against the effects of crash migration than other treatments. Although this study shows considerable benefits of VSL for the safety and efficiency of freeways, there are still substantial issues that need to be addressed including traffic law enforcement and drivers acceptance.

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