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# The Impacts of Using a Safety Compliance Standard in Highway Design

## **Cover Page Footnote**

This work was supported by a grant from the U.S. Dept. Transportation, University Transportation Centers Program.

# The Impacts of Using a Safety Compliance Standard in Highway Design\*

Paul J. Ossenbruggen\*\*

## Introduction

To test and evaluate safety objectively, I propose a compliance standard for the highway design process. The principle of individual lifetime risk is used to establish the standard.<sup>1</sup> A highway design location, or site  $s$ , is defined to be operating at an acceptable risk when an individual's chance of being involved in a fatal crash over a lifetime of motor vehicle travel at  $s$  is equal to or less than 1 in 1000, or  $\theta^* = 10^{-3}$ . Site  $s$  is defined "hazardous" if it fails to meet this criterion.

The highway design process as presented here uses a design algorithm derived from basic concepts of:

- highway design, or level of service (*LOS*) considerations;
- risk analysis, the principle of individual lifetime risk; and
- statistical modeling.

The algorithm will be described and a case study will demonstrate how it is applied and the design algorithm will be critiqued.

## Overview of the Design Algorithm

The design algorithm is formulated as a constrained optimization model using well-established principles of traffic flow and accepted highway design guidelines:

Objective: maximize  $\bar{u}$  (1)

Constraint set: subject to  $\pi \leq \bar{\omega}$  (fatal) (2)

$\pi_I \leq \bar{\omega}_I$  (injury) (3)

$\pi_P \leq \bar{\omega}_P$  (property damage) (4)

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\* This work was supported by a grant from the U.S. Dept. Transportation, University Transportation Centers Program.

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<sup>1</sup> Paul J. Ossenbruggen, *A Method of Identifying Hazardous Location Using the Principle of Individual Lifetime Risk*, 9 Risk 83 (1998).

The objective is to maximize average operating speed  $\bar{u}$  because speed is considered to be a most important *LOS* measure in design. "Speed and travel time are fundamental measurements of traffic performance of the existing highway system, and speed is the key variable in the redesign or design of new facilities."<sup>2</sup> "Except for local streets where speed controls are included intentionally, every effort should be made to use as high a design speed as practical to attain a desired degree of safety, mobility and efficiency while under the constraints of environmental quality, economics, esthetics and social and political impacts."<sup>3</sup>

A design is considered safe when the safety compliance constraint set is satisfied. That is, the predicted crash probabilities resulting in fatality  $\pi$ , injury  $\pi_i$  and property damage  $\pi_p$  are less than or equal to the corresponding compliance probabilities for fatality  $\varpi$ , injury  $\varpi_i$  and property damage  $\varpi_p$ .

In this paper, logistic regression is used to calibrate a crash prediction model for injury  $\pi_i$ . The data set is comprised of police accident reports, traffic volume and speed records for a five-year period at eight different, undivided two-lane highways in urban and rural Connecticut. The constrained optimization model for injury is:

$$\text{maximize } \bar{u} \quad (1)$$

$$\text{subject to } \pi_i \leq \varpi_i. \quad (3)$$

The model development, discussion and case study are focused on this model. Each of the  $\bar{u}$ ,  $\varpi_i$  and  $\pi_i$  models are presented in turn.

### The Average Operating Speed Model

The objective to maximize average operating speed  $\bar{u}$  is the concept used by highway designers. Average operating speed  $\bar{u}$  is a function of the free-mean speed  $u_f$  measured in miles per hour, traffic flow  $v$  and highway capacity  $c$  measured in vehicles per hour. It is calculated as:

$$\bar{u} = 0.5 \cdot u_f \left( 1 \pm \sqrt{1 - v/c} \right) \quad (5)$$

The equation is derived from Greenshield's linear speed-density model and flow-density-speed relationship.<sup>4</sup>

<sup>2</sup> Adolf D. May, *Traffic Flow Fundamentals* 116 (1990).

<sup>3</sup> Am. Ass'n State Highway & Transportation Officials (AASHTO), *A Policy on Geometric Design of Highway and Streets* 62 (1994).

Figure 1  
Average Operating Speed for  $u_f = 60$  mph

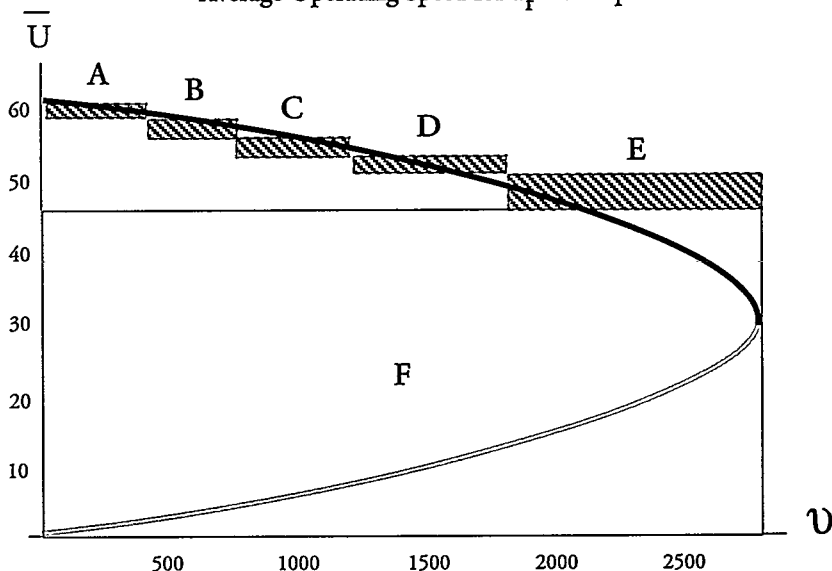


Figure 1 shows that the average operating speed  $\bar{u}$  model with  $u_f = 60$  mph and  $c = 2,800$  vph does a nice job of representing the Highway Capacity Manual<sup>5</sup> LOS letter rating system for a two-lane, undivided highway under ideal traffic conditions. An ideal condition is passenger cars traveling at an average operating speed of no less than 60 mph on level terrain with a 100% passing zone and with a 50/50 directional traffic flow split. A 50/50 split means that there are an equal number of passenger cars in each lane.

An ideal two-lane, undivided highway has a bidirectional flow capacity of  $c = 2,800$  vph. If one or more conditions are not met, then the capacity is reduced. Adjustments are made for grades  $> 3\%$ , directional distributions other than a 50/50, heavy vehicle usage, lane widths  $< 12$  ft, and shoulder widths  $< 6$  ft.

<sup>4</sup> See Nicholas A. Garber & Lester A. Hoel, *Traffic and Highway Engineering*, 184-85 (1997). Greenshield's Model is  $\bar{u} = u_f \left( 1 - \frac{k}{k_j} \right)$  where  $k$  = traffic density

in vehicles per mile (vpm) and  $k_j$  = jam density (vpm). The flow-speed-density relationship is  $v = u \cdot k$ .

<sup>5</sup> Transportation Research Board, *Highway Capacity Manual (HCM)*, Special Report 209, National Research Council (1994).

*Design Optimization:* For optimization, free-flow speed  $u_f$  is used as the control variable. A solution satisfying optimization model conditions, maximizing  $\bar{u}$  subject to:  $\pi_i \leq \omega_i$ , is designated as optimum solution  $u_f^*$ . For design,  $u_f$  is used as a design specification and  $u_f^*$  refers to a design specification that satisfies the objective function and the safety compliance constraint.

The free-mean speed  $u_f$  is a function of driver sight distance as determined by horizontal and vertical roadway curvature, right-of-way dimension, lane and shoulder widths — in other words, the highway's geometric alignment.<sup>6</sup> The design specification  $u_f$  affects average operating speed  $\bar{u}$  and highway capacity  $c$ . For example, the average speeds for two highways designed for  $u_f = 60$  and 45 mph given the same traffic flow  $v = 1,000$  vph are  $\bar{u} = 54$  and 49 mph, respectively.

*Highway Capacity:* Using Greenshield's linear speed-density relationship, highway capacity is calculated as:

$$c = u_f \cdot k_j / 4 \quad (6)$$

where vehicles  $k_j =$  jam traffic density measured in vehicles per mile (vpm). Given  $c = 2,800$  and  $u_f = 60$  mph, the jam density for ideal conditions is estimated to be  $k_j = 187$  vpm. This value of jam density  $k_j$  is assumed to be the same for all design specifications  $u_f$  for both ideal and non-ideal traffic conditions.

For example, given  $k_j = 187$  vpm for the two design specifications above, highway capacities are calculated to be  $c = 2,100$  and 2,800 vph for  $u_f = 45$  and 60 mph, respectively.<sup>7</sup> Furthermore with  $k_j$  assumed

<sup>6</sup> In this paper, specific values of the geometric factors will not be given. The important point is that the value of  $u_f$  can be achieved by specifying one or more geometric alignment factors. For example, if  $u_f < 60$  mph, then a particular value of  $u_f$  can be obtained by specifying a lane width  $< 12$  ft or by specifying a combination of lane width  $< 12$  ft and shoulder width  $< 6$  ft. Of course, other combinations can also lead to the desired value of  $u_f$ .

<sup>7</sup> The Highway Capacity Manual uses adjustment factors to adjust the capacity  $c$  for non-ideal conditions. Consider a design specification  $u_f = 45$  mph. An

to be a constant, the highway capacity formula reduces to a linear function,  $c = 48.5 \cdot u_f$ .

*Speed Maximization:* Given a traffic flow  $v$ , the solid line speeds in Figure 1 are calculated as:  $\bar{u} = 0.5 \cdot u_f \left(1 + \sqrt{1 - v/c}\right)$ ; and the broken line speeds are calculated as:  $\bar{u}_f = 0.5 \cdot u_f \left(1 - \sqrt{1 - v/c}\right)$ .

Since  $\bar{u} > \bar{u}_f$  and the design objective is to maximize  $\bar{u}$ , the solutions given by  $\bar{u}_f$  are not of interest for design optimization. Furthermore, since  $c = 48.5 \cdot u_f$ , the objective function is written as the function of the control variable  $u_f$  exclusively:

$$\text{maximize } \bar{u} = 0.5 \cdot u_f \left(1 + \sqrt{1 - v/48.5 \cdot u_f}\right) \quad (7)$$

This function draws attention to the fact that the  $u_f$  specification directly affects the highway speed and capacity.

*Safety Considerations:* Driver convenience and speed are often sacrificed by reducing the average operating speed  $\bar{u}$ . Theoretically, this can be achieved through (1) the geometric alignment of the highway and through (2) speed limit control.

*Geometric Alignment and Traffic Calming:* Highway designers and planners are under pressure to construct a high-speed highway system because there is an insatiable worldwide desire for mobility.<sup>8</sup> The construction of "big roads", that is, wide, straight roads with geometric alignment to maximize driver sight distance are favored.

The design algorithm puts less emphasis on mobility (reducing congestion and delay)<sup>9</sup> for purposes of improved highway safety and

adjustment factor for narrow lanes and restricted shoulder width is  $f_w = 0.75$ . The capacity is  $c = 2,800 \cdot f_w = 2,100$  vph.

<sup>8</sup> Andreas Schafer & David Victor, *The Past and Future of Global Mobility*, *Scientific American*, October 1997, at 58.

<sup>9</sup> Congestion and delay often accompanied with a long waiting line are a possibility when the traffic flow  $v$  approaches the highway capacity  $c$ . In Figure 1 where  $u_f = 60$  mph and when  $v$  approaches  $c = 2,800$  vph, the average operating speed is about

more emphasis on controlling speed through geometric design. It permits the use of narrow highway lanes and reduced sight distance to control speed by forcing drivers to slow down.

According to traffic-calming advocates, "The idea that bigger roads increase people's mobility" is a myth.<sup>10</sup> They claim that straight, wide roads encourage speed and greater risks. They show that measures to force drivers to slow down are effective to control speed and reduce injury and fatal crashes in cities and residential areas. They claim that the toll can be reduced more than 40% with traffic-calming methods.

The design algorithm and the traffic-calming methods have a similar goal, but the means differ. Traffic-calming measures are generally employed in residential communities where quality of life from high-speed traffic is threatened. Roads are calmed by employing geometric alignment and other techniques, such as speed tables, chicanes, neck-downs and interrupted sight lines. These dramatically reduce the average operating speed, so much so, as to cause traffic diversion. Of course, this is achieved through purposeful design. Traffic encourages motorists to find alternate roadways; therefore, both speed  $\bar{u}$  and traffic volume  $v$  are reduced.

The geometric alignment methods contemplated for the design algorithm reduce speed less dramatically without causing traffic diversion. Furthermore, introducing traffic diversion into the design algorithm would greatly complicate the mathematics.

*Speed Limit Control:* This is often employed to reduce speed  $\bar{u}$  at sites where crashes occur due to excessive speed or where excessive speed is considered a hazard. For example, it is not uncommon to observe a highway with a posted speed limit  $s_p = 30$  mph with a design specification  $u_f = 60$  mph. Clearly, the speed restriction would be unnecessary if the highway was safe at operating speeds that approach the design specification speed  $u_f$ . The signage is an attempt to control

one-half  $u_f$  or  $\bar{u} \cong 30$  mph. Under the same condition for  $u_f = 45$  mph,  $v$  approaches  $c = 2,100$  vph at  $\bar{u} \cong 23$  mph. Either equation 5 or 7 can be used to calculate  $\bar{u}$  for  $u_f = 45$  mph.

<sup>10</sup> Citizens Advocating Responsible Transportation (CART), *Traffic Calming, The Solution to Urban Traffic and a New Vision for Neighborhood Livability*, Sensible Transportation Options for People (STOP) 12 (1989).



the average operating speed  $\bar{u}$  for a highway design specification deemed to be too fast and hazardous for site  $s$ .

In the context of design optimization, this design specification given in the example does not satisfy the safety compliance constraint  $\pi_f \leq \varpi_f$  for  $u_f = 60$  mph. It is an infeasible solution; therefore,  $u_f^* \neq u_f = 60$  mph. The speed limit control method, while used in practice, is not applicable to the philosophy or methods espoused. The aim of this paper is to design a safe highway the first time, and to avoid the use of corrective traffic control schemes and costly roadway reconstruction at hazardous sites.

### The Allowable Safety Limit Model

The allowable limit for fatality crashes  $\varpi$  is determined from the individual lifetime risk model,<sup>11</sup>

$$\theta = 1 - \exp(-70 \cdot \eta \cdot \varpi) \quad (8)$$

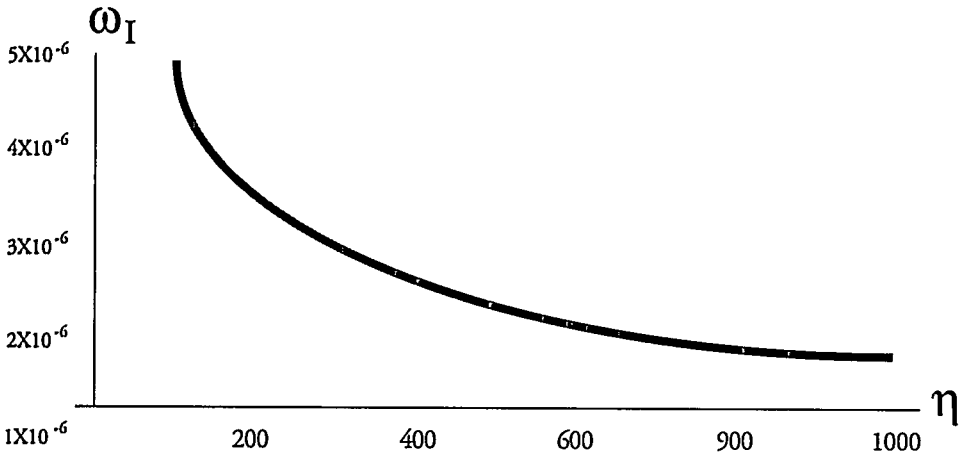
and the assignment of an acceptable lifetime risk  $\theta = 10^{-3}$ . The annual exposure  $\eta = 664$  trips per year per person. Given these assignments, the allowable limit for fatal crashes is calculated to be  $\varpi = 2.2 \times 10^{-8}$ . Given that one in about 55 serious injury crashes is fatal, the allowable limit for injury crashes is  $\varpi_f = 1.2 \times 10^{-6}$ . The assignment of  $\varpi_p$  is based on property damage costs and is independent of individual lifetime risk considerations and outside the scope of this paper.

Figure 2 shows the effect of annual trip exposure  $\eta$  on  $\varpi_f$  for a constant acceptable risk equal to  $\theta = 10^{-3}$ . If incentives to travelers to reduce the annual individual trip exposure  $\eta$  can be found, then  $\varpi_f$  can be relaxed. In other words, the allowable limit of  $\varpi_f = 1.2 \times 10^{-6}$  can be increased. In the U.S. exposure  $\eta$  is increasing among a growing driver population. If an acceptable lifetime risk equal to  $\theta = 10^{-3}$  is to remain constant over time, then  $\varpi_f$  should be decreased to account for the increased individual exposure to highway risk.

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<sup>11</sup> See Ossenbruggen, *supra*, note 1 at 86. The model was derived from basic principles of probability using the geometric and Poisson distributions. A premature death is considered to be a person who dies before the age of 70 years. According to the National Personal Transportation Survey, in 1990 a person made an average of 664 trips per year.

Figure 2  
 Allowable Limit for an Individual Lifetime Risk  $\theta = 10^{-3}$



### An Injury Crash Prediction Model

Traffic volume counts, police accident reports and other descriptive materials for eight sites in Connecticut from 1990-94 formed a data set for model calibration and validation. Each site listed in Tables 1 and 2 are continuous traffic counting stations grouped by posted speed limit  $s_p$ . The characteristics given under the headings of Land Use, Traffic Control and Geometric Design Factors in Table 2 show a variety of adjacent land use and roadside activity and highway designs located in rural and urban areas in Connecticut.

*Exploratory Data Analysis:* Such analysis is an intuitive and effective means to identify patterns and trends in data and often helps to identify statistically significant factors prior to performing model calibration and validation testing.

The total, injury and fatal crashes and estimates of the probability of crashes causing property damage  $\tilde{\pi}_p$  and injury  $\tilde{\pi}_i$  are given in Table 1. The probability  $\tilde{\pi}_i$  is estimated as the ratio of the number of injury crashes to trip count.<sup>12</sup> Similarly,  $\tilde{\pi}_p$  is estimated as the ratio of the number of property damage crashes to trip count. For example, the estimates for Hebron are  $\tilde{\pi}_i = 10/1.3 \times 10^6 = 7.7 \times 10^{-6}$  and  $\tilde{\pi}_p = (23 - 10)/1.3 \times 10^6 = 10. \times 10^{-6}$ , respectively.

<sup>12</sup> The five-year trip count is treated as a measure of highway risk exposure.

Table 1  
Five -Year Trip Volume and Crash Counts

Site <i>s</i>	Posted Speed $s_p$ (mph)	Trip Volume ( $10^6$ )	Crash Total	Counts Injury	Fatal	$\tilde{\pi}_P$ ( $10^{-6}$ )	$\tilde{\pi}_L$ ( $10^{-6}$ )
Darien	35	17.7	56	25	0	1.8	1.4
Killingly	35	8.1	34	17	0	2.1	2.1
Hebron	35	1.3	23	10	0	10.0	7.7
Totals		27.1	113	52	0	2.3	1.9
Waterford	40	4.7	72	21	0	11.0	4.5
Kent	40	3.4	19	9	0	2.9	5.6
Colebrook	40	4.6	7	4	1	2.9	1.5
Totals		12.7	98	34	1	5.0	2.7
E. Windsor	45	11.0	6	0	0	0.5	0.0
Clinton	45	7.1	6	4	0	0.3	0.6
Totals		18.1	12	4	0	0.4	0.2

Table 2  
Site Characteristics

Site <i>s</i>	$s_p$ (mph)	Land Use		Traffic Control			Geometric Design		
		Pop. ( $10^3$ )	Road Class	Heavy Vehicles (%)	No. Signals	No. Stop Signs	Lane Width (feet)	Shoulder Width (feet)	Sight Distance (feet)
Darien	35	50-200	UPA	3	0	0	11	2	1,500
Killingly	35	< 5	RMA	2	0	0	12	4	1,350
Hebron	35	< 5	RPA	3	1	0	12	8	1,350
Waterford	40	50-200	UPA	1	1	0	12	3	1,500
Kent	40	< 5	RMA	1	0	0	12	1	1,350
Colebrook	40	< 5	RMA	1	0	0	11	2	1,200
E. Windsor	45	> 200	UPA	2	0	0	12	5	1,500
Clinton	45	< 5	RPA	1	0	0	12	1	1,500

UPA = Urban Principal Arterial, RMA = Rural Minor Arterial, RPA = Rural Principal Arterial

If  $\tilde{\pi}_i$  estimates are used to rank sites, Hebron is the most hazardous location because it has the largest  $\tilde{\pi}_i$  value in Table 1. Darien and East Windsor with the two highest traffic volumes and Darien and Waterford with the maximum number of injury and total crash counts are relatively safe when comparing their  $\tilde{\pi}_i$  values to other sites. The trip count, a measure of exposure to highway risk at a site *s*, plays a critical role in the  $\tilde{\pi}_i$  calculation and also in the site ranking.

*Speed Limit Control:* When comparing speed limit groups, the total injury and fatal crashes and the values of  $\tilde{\pi}_I$  and  $\tilde{\pi}_P$  in Table 1 are the greatest at sites where the most restrictive speed limit controls are used. Highway risk is greater at sites with posted speed limits of 35 and 40 mph than at sites with the least restrictive speed limit of 45 mph.

*Land Use and Roadside Activity:* Table 2 shows a diverse set of land use, traffic control and geometric design characteristics for sites in each speed limit group. However, nothing seems to stand out to explain why some have greater crash probability. Sorting the data in different ways and using contingency tables and scatter plots was revealing.

The contingency table, Table 3, suggests that time-of-day and LOS rating may be important explanatory variables. Comparing  $\tilde{\pi}_I$  and  $\tilde{\pi}_P$  values in the two time-of-day categories by the same LOS rating show a pattern that suggests that there is a greater chance of crashes during dusk than at any other time of day.

Table 3<sup>13</sup>  
Contingency Table of Five-Year Traffic Volume and Crash Counts

<i>Time-of-day</i>	LOS	<i>Traffic Count (10<sup>6</sup>)</i>	<i>Counts Total</i>	<i>Injury</i>	$\tilde{\pi}_P$ (10 <sup>-6</sup> )	$\tilde{\pi}_I$ (10 <sup>-6</sup> )
Dawn/Day/Night	A	6.1	85	41	7.2	6.7
	B	17.9	62	17	2.5	1.0
	C	7.7	17	4	1.7	0.5
	D	4.6	12	6	1.3	1.3
	E	6.4	16	4	1.9	0.6
Dusk	A	3.0	12	8	1.3	2.7
	B	2.0	8	3	2.5	1.5
	C	0.6	3	3	0.0	5.0
	D	0.5	1	1	0.0	2.0
	E	1.2	7	3	3.3	2.5

Comparing  $\tilde{\pi}_P$  and  $\tilde{\pi}_I$  values by LOS rating within the dawn, day and night category and to a lesser degree within the dusk category suggests that the probability of being in a crash is dependent on LOS rating. Travelers experiencing driving conditions rated as LOS A and

<sup>13</sup> The  $\nu/c$  ratios for each site were calculated using the highway geometric design characteristics given in Table 2. The data for each site were then sorted by LOS rating and then combined to form this table.

B, are more likely to be involved in a crash than at poorer LOS ratings. This suggests that the average operating speed  $\bar{u}$  is related to the crash probability.

Logit scatter plots, which are not shown in this paper, suggest location  $s$ , posted speed limit  $s_p$ , time-of-day  $t$  and shoulder width may be significant explanatory variables. A logit is calculated as the natural logarithm of the ratio of the number of injury crashes to trip count or  $\log[\hat{\pi}_I]$ . The scatter plot for shoulder width suggests that shoulder widths of three feet or more tend to reduce the probability of a crash resulting in injury.

The exploratory data analyses suggest that:

- LOS rating or vehicular speed is an important factor in explaining the number of crashes;
- a posted speed limit has little effect in minimizing highway risk; and
- time-of-day and shoulder width may be important factors in predicting crash probability.

*Modeling Calibration Results:* LOS rating, expressed as capacity utilization  $v/c$ , posted speed limit  $s_p$ , time period  $t$  and the characteristics listed in Table 2 were introduced as candidate variables in logistic regression model calibration and testing. The method of maximum likelihood was used to calibrate models and to estimate the variances and covariances of their model parameters. Models were tested using the likelihood-ratio (Wilk's statistic) and Wald tests.<sup>14</sup>

The following prediction model<sup>15</sup> satisfied validation testing:

$$\pi_I = \frac{\exp[-8.34 - 0.12 \cdot s_p - 0.34 \cdot t - 1.36 \cdot v/c]}{1 + \exp[-8.34 - 0.12 \cdot s_p - 0.34 \cdot t - 1.36 \cdot v/c]} \quad (9)$$

The time period variable  $t$  is a discrete variable where  $t = -1$  for dusk and  $t = 1$  for dawn, day or night (D/D/N). The variables  $s_p$  and  $v/c$

<sup>14</sup> David W. Hosmer & Stanley Lemeshow, *Applied Logistic Regression* 25(1989); Alan Agresti, *Categorical Data Analysis* 112 (1990).

<sup>15</sup> Kopl Halperin, *A Comparative Analysis of Six Methods for Calculating Travel Fatality Risk*, 4 Risk 14 (1996). Traffic engineers report fatality rates in the number of fatalities per vehicle miles traveled (VMT). VMT is considered to be an inappropriate measurement for public health hazards.

The crash prediction and lifetime risk models are site specific derived and calibrated on point measurements of crash and trip counts.

are continuous variables with ranges of  $35 \leq s_p \leq 45$  mph and  $0 \leq v/c \leq 1$ , respectively. All model parameters are significant at  $\alpha = 5\%$ .

Shoulder width, which showed promise in the exploratory data analyses, when treated as a continuous variable was insignificant at  $\alpha = 5\%$ . When introduced as a discrete variable, it proved to be a significant variable; however, the model was considered unsuitable for the general concepts presented in this paper.

*Model Properties:* For purposes of crash prediction and highway design, a model should minimally be a function of variables reflecting the travel demand, land use, roadside activity and geometric design features at  $s$ . The crash prediction model  $\pi_I$  satisfies these requirements with the following variables serving various purposes:

- $v$ , a travel demand input parameter;
- $s_p$ , a surrogate land use and roadside activity variable;
- $c$ , a principle design variable;
- $v/c$ , a measure of design performance *LOS*; and
- $t$ , signifying that crash probability is a function of time-of-day.

*Travel Demand:* The effect of travel flow  $v$  on  $\pi_I$  is most easily seen in Figures 3 and 4, the major difference being the designation of  $s_p$ . Comparing the *dusk* plots indicates that the probability of a crash is larger at a site with  $s_p = 35$  mph than the one with  $s_p = 45$  mph. The same holds for the *dawn, day and night* plots.

The safety compliance constraint,  $\pi_I \leq \overline{\omega}_I$ , is satisfied for all traffic volumes  $v$  except at dusk for  $v < 500$  vph for site designation  $s_p = 45$  mph shown in Figure 3. The safety compliance constraint is violated at dusk for all  $v$  and for dawn, day or night when  $v < 1,500$  vph for site designation  $s_p = 35$  mph. Clearly, the highway risk is greatest at a site designated  $s_p = 35$  mph than at a site designated  $s_p = 45$  mph shown in Figure 4.

Figure 3  
Probability of injury-causing crash at site  $s_p = 45$  mph

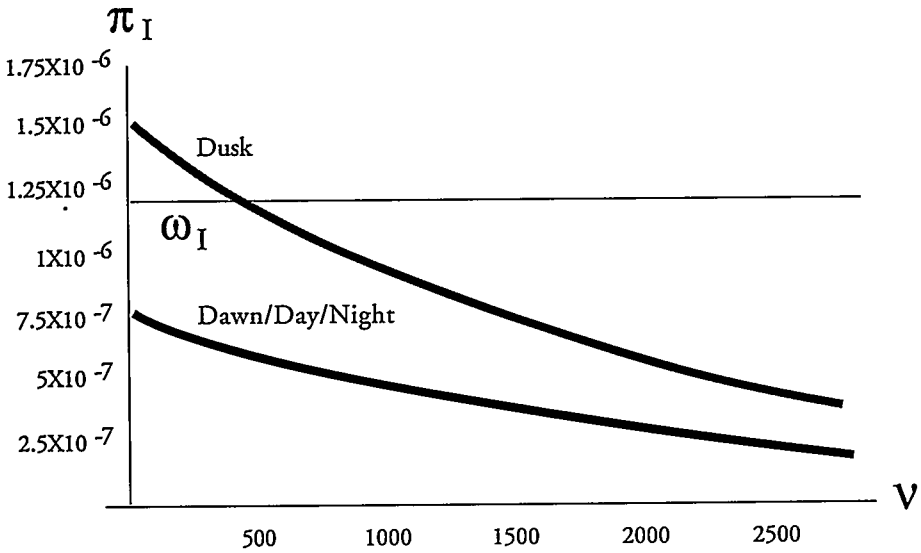
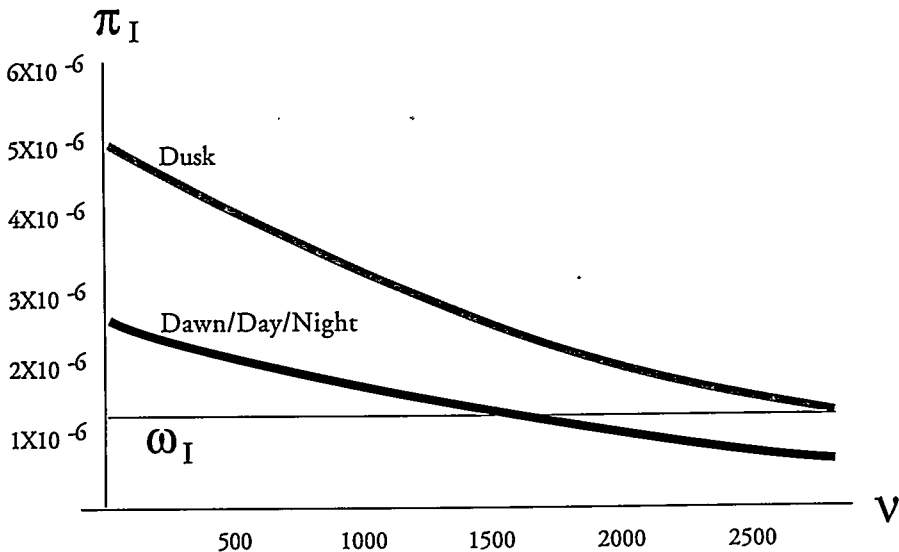


Figure 4  
Probability of injury-causing crash at site  $s_p = 35$  mph



*The Surrogate Land Use and Roadside Activity Variable:* The variable  $s_p$  indicates how hazardous it is to drive at a site  $s$  and is considered as a site-characteristic variable.

Interpreting  $s_p$  as a traffic control measure leads to the claim that the probability of an injury crash will decrease by increasing the posted speed limit. This is a naive claim. Speed limits are imposed to reduce the probability of crashes. The only meaningful interpretation is that more restrictive speeds are imposed at more hazardous sites. Sites posted at 35 mph have greater risks than ones posted at 40 or 45 mph.

The  $s_p$  variable is not considered as a traffic-control measure in the crash-prediction model. In fact, the model suggests that a posted speed limit is ineffective to improve highway safety. This is consistent with the findings of a study of raising and lowering posted speed limits on 83 comparison sites over increments of 5, 10, 15 and 20 mph. The signs had no practical significance in controlling speed.<sup>16</sup>

*LOS:* As the  $v$  increases, both average operating speed  $\bar{u}$  and crash probability  $\pi_I$  decrease. This result suggests that a loss in *LOS* is coupled with an improvement in highway safety. Stated another way, it suggests that  $\bar{u}$  and  $\pi_I$  are positively correlated. Simply stated: *Faster speed is associated with greater highway risk.*

*Time-of-day Considerations:* An individual is not exposed to the same travel volumes each hour of the day,  $h = 1, 2, 3, \dots, 24$ . Traffic flow varies by hour of the day. A key point in the design algorithm considers this range of hourly traffic volume  $v_h$  exposure with the use of marginal and condition probabilities.

The constrained optimization model for injury written as a function free-flow speed  $u_f$  becomes:

$$\text{maximize } \bar{u} = \sum_h p_h \cdot \bar{u}_h \quad (10)$$

$$\text{subject to } \pi_I = \sum_h p_h \cdot \pi_{I|h} \leq \bar{\omega}_I \quad (11)$$

where the conditional probability for average operating speed given hour  $h$  is:

$$\bar{u}_h = 0.5 \cdot u_f \left( 1 + \sqrt{1 - v_h / 48.5 \cdot u_f} \right), \quad (12)$$

the conditional crash probability for injury given hour  $h$  is:

<sup>16</sup> Federal Highway Administration, *Effects of Raising and Lowering Speed Limits on Selected Roadway Sections* (FHWA-RD-92-84, Jan. 1997).



$$\pi_{lh} = \frac{\exp\left[-8.34 - 0.12 \cdot s_p - 0.34 \cdot t_h - 0.028 \cdot \frac{v_h}{u_f}\right]}{1 + \exp\left[-8.34 - 0.12 \cdot s_p - 0.34 \cdot t_h - 0.028 \cdot \frac{v_h}{u_f}\right]}, \quad (13)$$

and  $P_h$  = probability that an individual is traveling in hour  $h$ . The values of  $P_h$  are estimated to be the ratio of the hourly to daily traffic

counts,  $P_h = \frac{v_h}{\sum_h v_h}$ . The summation can be interpreted as the average daily traffic (*ADT*).

Consequently, the design process using marginal probabilities considers all hours of the day, incorporating, e.g., the effects of high speed on risk  $\pi_I$  and high volume on *LOS* as measured by  $\bar{u}$ .

*Odds*: Since the crash probabilities  $\pi_I$  are small numbers and can be difficult to comprehend, the odds are summarized in Table 4. In the case of time-of-day, odds =  $\pi_I(A)/\pi_I(B) = \pi_I(t = -1)/\pi_I(t = 1)$  where  $t = 1$  (dusk) and  $t = -1$  (dawn, day or night) where  $\frac{v}{c}$  and  $s_p$  are assigned the same values for  $t = 1$  and  $t = -1$ . The odds of an injury-causing crash is twice as great at dusk than during dawn, day or night.

Table 4 — Odds

	B	A	Odds = $\frac{\pi_I(A)}{\pi_I(B)}$
<i>Time-of-day</i>	$t = -1$	$t = 1$	2.0
<i>Land Use and Roadside Activity</i>	$s_p = 45$ mph	$s_p = 40$ mph	1.8
		$s_p = 35$ mph	3.3
<i>Operating Speed</i>	$\bar{u} = 45$ mph	$\bar{u} = 50$ mph	1.3
		$\bar{u} = 55$ mph	1.8
		$\bar{u} = 60$ mph	2.7

In the case of land use and roadside activity, the odds of a crash resulting in injury is 3.3 times greater at site  $s_p = 35$  mph than at site  $s_p = 45$  mph. Clearly, a highway site posted at 35 mph will be expected to pose the greatest design challenges.

Since average operating speed is a most important *LOS* measure and it is a function of  $\frac{v}{c}$ ,  $\bar{u}$  was used in the odds table with capacity  $c = 2,800$  vph. The odds of a crash resulting in an injury is 2.7 times greater at  $\bar{u} = 60$  mph (*LOS A*) than at  $\bar{u} = 45$  mph (*LOS D*).

### Case Studies

The design specification  $u_f$  affects  $c$ ,  $\pi_I$ ,  $\bar{u}$ ,  $\frac{v}{c}$  and the *LOS* rating. Assigning it is critical in design optimization. Here, graphs of  $\pi_I$  are plotted as functions of  $u_f$  with travel demand  $v$  constant.

Figure 5 contains  $\pi_I$  plots for sites designated  $s_p = 45$  mph for traffic volumes  $v = 400$  and 2,000 vph at dusk and at dawn, day and dusk. For simplicity, the subscript  $h$  is not shown. Figure 6 contains  $\pi_I$  plots for sites designated  $s_p = 35$  mph for the same traffic volumes and times-of-day as Figure 5. Figure 5 shows that the safety compliance constraint,  $\pi_I \leq \omega_I$ , is satisfied for a wide range of  $u_f$  values at almost any time of day and at both traffic volumes, but Figure 6 shows that  $\pi_I \leq \omega_I$  is satisfied for a narrow range of conditions.

Figure 5  
Probability of injury-causing crash at 45 mph site

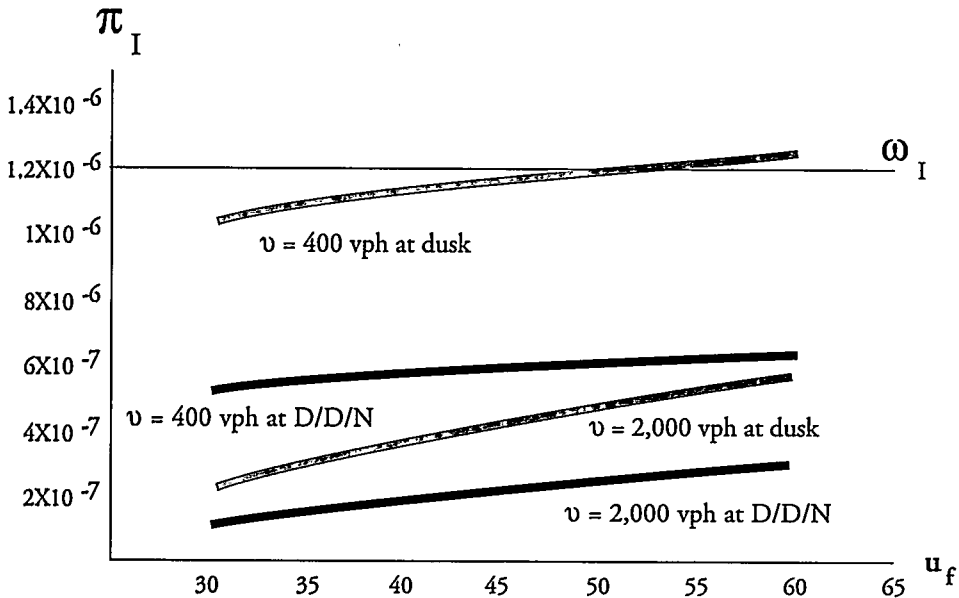


Figure 6  
Probability of injury-causing crash at 35 mph site

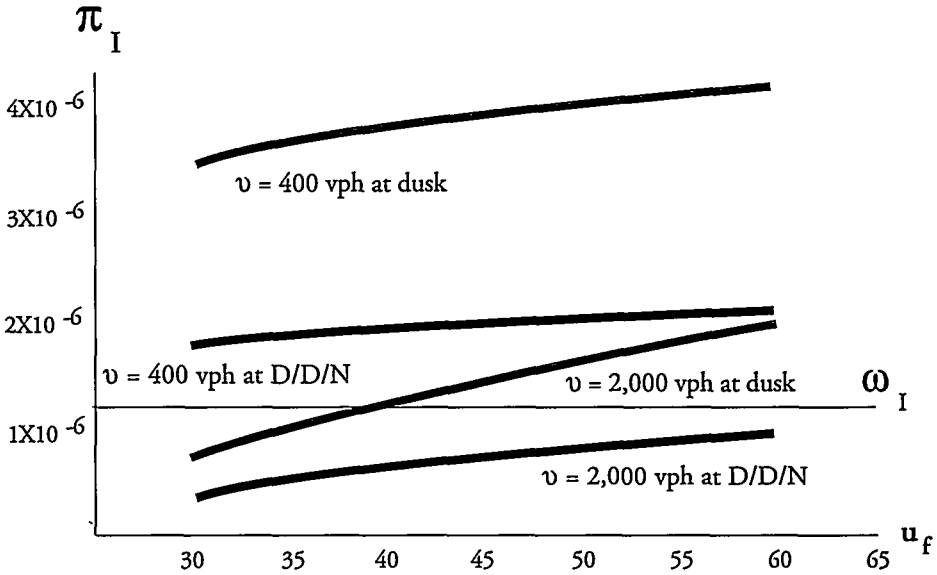


Table 5 contains case study results for sites designated  $s_p = 35, 40$  and  $45$  mph. In each case, the same annual trip count of  $5.8 \times 10^6$  or  $ADT = 16,000$  trips per day is assumed. The hourly traffic volume  $v_h = 400$  vph is assumed for all hours of the day except for a two-hour dawn period and for a two-hour dusk peak period. During these two-hour periods,  $v_h = 2,000$  vph. The  $\pi_I$  and  $\bar{u}$  values are calculated as marginal probabilities given by equations 12 and 13. The candidates for  $u_f^*$  are given under column heading  $u_f$ .

*Sites  $s_p = 40$  and  $45$  mph:* Inspection of case study results show that optimal solutions are obtained for sites designated  $s_p = 40$  and  $45$  mph. That is, *maximum  $\bar{u} = 52$  mph subject to:  $\pi_I \leq \bar{\omega}_I$*  is achieved.

*Sites  $s_p = 35$  mph:* The four candidate solutions were non-optimal.

Table 5  
Case Study Results (annual trip count =  $5.8 \times 10^6$ )

Site $s_p$	$u_f$ (mph)	$\pi_I$	$\bar{u}$	$u_f^*$	Comments
45	60	$0.5 \times 10^{-6}$	52	Yes	$u_f^* = 60$ mph
40	60	$1.0 \times 10^{-6}$	52	Yes	$u_f^* = 60$ mph
35	60	$1.8 \times 10^{-6}$	52	No	$\pi_I > \bar{\omega}_I$
	50	$1.6 \times 10^{-6}$	41	No	$\pi_I > \bar{\omega}_I$
	40	$1.4 \times 10^{-6}$	—	No	$\pi_I > \bar{\omega}_I, v_h > c$
	30	$1.1 \times 10^{-6}$	—	No	$v_h > c$

The reasons are:

- safety non-compliance because  $\pi_I > \bar{\omega}_I$ ;
- traffic congestion and delay because  $v_h > c$ ; and
- a combination of these.

Reducing the free-flow speed  $u_f$  is marginally effective in reducing  $\pi_I$ . Inspection of any of the  $\pi_I$  plots in Figure 6 shows that their slopes are slight. As a result, design specifications  $u_f < 60$  mph reduce  $\pi_I$  values to a relatively small degree.

Consider design specification  $u_f = 40$  mph where  $\pi_I > \bar{\omega}_I$  and  $v_h > c$  are cited as reasons for non-optimality. The safety constraint is non-compliant when  $\pi_I$  is calculated as a marginal probability, even though the site meets the safety compliance constraint during dawn and dusk. Figure 6 shows  $\pi_{lh} = \bar{\omega}_I$  for  $v_h = 2,000$  vph at dusk and  $\pi_{lh} < \bar{\omega}_I$  for  $v_h = 2,000$  vph at dawn. A significant portion of the ADT, however, occurs during the day and night when  $v_h = 400$  vph and  $\pi_{lh} > \bar{\omega}_I$ . Also, the capacity at  $u_f = 40$  mph is  $c = 1,870$  vph; therefore,  $v_h > c$  during the dawn and dusk. This design specification is also unacceptable because of traffic congestion and delay.

*Relaxing the Allowable Limit:* An option that remains is to increase the allowable limit  $\bar{\omega}_I$  by reducing an individual's exposure  $\eta$  to

highway risk. Suppose at site  $s_p = 35$  mph, an alternative is found to reduce individual exposure from  $\eta = 664$  to 400 trips per person per year. The allowable limit is increased from  $\overline{\omega}_f = 1.2 \times 10^{-6}$  to  $2 \times 10^{-6}$  as shown in Figure 2. Now, the design specification  $u_f = 60$  mph satisfies the safety compliance constraint; thus  $u_f^* = 60$  mph!

Given the heavy reliance on private motor-vehicles in our daily lives, many trips are ones of necessity not choice. The most mundane tasks, such as buying a newspaper or a loaf of bread, require a trip to the store by automobile. Convenience stores are outlawed by local zoning ordinances in many suburban communities. Through coordinated transportation and land use planning efforts, both individual exposure to the private motor-vehicle  $\eta$  and traffic volume  $v$  can be reduced. In addition to promoting highway safety, attractive alternatives, like public transportation, pedestrian and bicycle friendly communities, have far reaching social, public health and environmental benefits.

### Discussion

This paper flags a new outlook that the concept of individual lifetime risk can bring to highway design. An algorithm was structured as a constrained optimization problem, with an objective to maximize average operating speed subject to a safety compliance constraint. Case studies were analyzed using the design algorithm and its models, average operating speed, allowable safety limit and crash prediction models. Results, exploratory data analysis, and individual models, individually and collectively, are insightful to the highway design process. For example, the crash prediction model gives insights as to why a design may not satisfy the safety compliance constraint at a site; and when this insight is introduced into the larger framework of constrained optimization, this additional information gives further insights as to how an optimal design can be achieved.

The analyses and case study results suggest that highway risk:

- is highly dependent on adjacent land use and roadside activity;
- is marginally reduced by geometric alignment;
- may be inversely proportional to posted speed limits;
- may be reduced if alternatives can divert motorists; and
- may be more effectively ranked and communicated using individual lifetime risks, odds and crash probabilities.

The crash prediction model, average operating speed and allowable safety limit models, fundamental to the design process using a safety compliance standard, have shortcomings, but no model is “perfect” or solution without criticism. No model can incorporate the driver’s multifaceted demands, the neighborhood, and views of various organizations concerned with transportation service, environment, public health and financing. Even when used for design optimization, the models cannot address most of these demands.

The crash prediction model is the least useful. It cannot address issues associated with fatal crashes. That limitation, use of a surrogate variable for land use and roadside activity, and the questionable result suggesting that “reducing highway risk by geometric alignment is marginally effective”, all directly link to limited data. Constructing a data set for model calibration poses three difficulties.

The eight Connecticut sites were needed for high-quality annual traffic counts and speed data. Without that, the potential significance of *LOS* rating and time-of-day as explanatory variables would not have been discovered. It was unknown that the chosen sites would only show but one fatal crash in five years despite annual traffic of 50 million. Thus, model calibration had to be limited to injury-causing crashes.

Second, posted speed limit, a surrogate variable, does not describe site characteristics. Adding more sites with other land use characteristics would be beneficial. Recall that population and road class were the only data available. Yet, adding sites and land use characteristics must assure that the data can provide reliable annual traffic counts, critical to measuring highway risk exposure.

Third, the suggestion that “reducing highway risk by geometric alignment is marginally effective” was obtained by extrapolation. Specifically, crash predictions and average operating speeds calculated for design specifications  $u_f = 30$  and 40 mph for site designation  $s_p = 35$  mph in Table 5 are suspect. Sites used in this study have highway design speeds of 50 mph and greater. The highways have good sight distances and adequate lane and shoulder widths as given in Table 2. The model calibration did not include data for highway designs of 30 and 40 mph; therefore, the predictions for these highway design speeds are not supported by observation. A data set consisting of sites with

highway design speeds of 40 mph and less and sites where traffic-calming measures have been used is desirable. Models calibrated with this data set will clarify whether or not geometric alignment is an effective method in reducing highway risk and speed. The predictions given by the crash prediction model for design specifications  $u_f = 30$  and 40 mph seem larger than results given by CART.<sup>17</sup>

Regardless of imperfections, the overall benefits of the crash prediction model outweigh its shortcomings. Especially, when it is introduced into constrained optimization model, its benefits and the potential usefulness of the design algorithm for highway design are demonstrated. The crash prediction model, in its current stage of development, is considered to be a concept model.



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<sup>17</sup> See *supra* note 10.

