TRB

TRANSPORTATION RESEARCH RECORD

Journal of the Transportation Research Board, No. 1922

Safety

Older Drivers; Traffic Law Enforcement; Management; School Transportation; Emergency Evacuation; Truck and Bus; and Motorcycles

12-Safety

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Transportation Research Record 1922

Contents

Foreword

The 2005 series of the *Transportation Research Record: Journal of the Transportation Research Board* consists of approximately 770 papers selected from 2,600 submissions after rigorous peer review. The peer review for each paper published in this volume was coordinated by the sponsoring committee acknowledged at the end of the text; members of the sponsoring committees for the papers in this volume are listed on page ii. Many of these papers were presented at the TRB 84th Annual Meeting in January 2005, and draft versions were included in the Annual Meeting Compendium of Papers CD-ROM.

Additional information about the *Transportation Research Record: Journal of the Transportation Research Board* series and the peer review process appears on the inside back cover. TRB appreciates the interest shown by authors in offering their papers, and the Board looks forward to future submissions.

Measurement Conversion Factors

To convert from the unit in the first column to the unit in the second column, multiply by the factor in the third column.

Customary Unit	*SI Unit*	*Factor*
Length		
inches	millimeters	25.4
inches	centimeters	2.54
feet	meters	0.305
yards	meters	0.914
miles	kilometers	1.61
Area		
square inches	square millimeters	645.1
square feet	square meters	0.093
square yards	square meters	0.836
acres	hectares	0.405
square miles	square kilometers	2.59
Volume		
gallons	liters	3.785
cubic feet	cubic meters	0.028
cubic yards	cubic meters	0.765
Mass		
ounces	grams	28.35
pounds	kilograms	0.454
short tons	megagrams	0.907
Illumination		
footcandles	lux	10.76
footlamberts	candelas per square meter	3.426
Force and Pressure or Stress		
poundforce	newtons	4.45
poundforce per square inch	kilopascals	6.89

Temperature

To convert Fahrenheit temperature (°F) to Celsius temperature (°C), use the following formula:
°C = (°F – 32)/1.8

SI Unit	*Customary Unit*	*Factor*
Length		
millimeters	inches	0.039
centimeters	inches	0.394
meters	feet	3.281
meters	yards	1.094
kilometers	miles	0.621
Area		
square millimeters	square inches	0.00155
square meters	square feet	10.764
square meters	square yards	1.196
hectares	acres	2.471
square kilometers	square miles	0.386
Volume		
liters	gallons	0.264
cubic meters	cubic feet	35.314
cubic meters	cubic yards	1.308
Mass		
grams	ounces	0.035
kilograms	pounds	2.205
megagrams	short tons	1.102
Illumination		
lux	footcandles	0.093
candelas per square meter	footlamberts	0.292
Force and Pressure or Stress		
newtons	poundforce	0.225
kilopascals	poundforce per square inch	0.145

Temperature

To convert Celsius temperature (°C) to Fahrenheit temperature (°F), use the following formula:
°F = (°C × 1.8) + 32

Abbreviations Used Without Definitions

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials (known by abbreviation only)
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ISO	International Organization for Standardization
ITE	Institute of Transportation Engineers
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
SAE	Society of Automotive Engineers
TCRP	Transit Cooperative Research Program
TRB	Transportation Research Board

Stops for Cops

Impaired Response Implementation for Older Drivers with Cognitive Decline

Matthew Rizzo, Qian Shi, Jeffrey D. Dawson, Steven W. Anderson, Ida Kellison, and Thomas Pietras

Response to an emergency vehicle requires detection and recognition of an object in peripheral vision, situation recognition, and a rapid response to execute a safety maneuver to decrease the potential for crashing into the vehicle or striking people situated near it. To investigate situation awareness and response to a roadway emergency in at-risk elderly drivers, 149 licensed older drivers were tested with a battery of visual and cognitive tests and in a driving simulator scenario in which drivers encountered a police car on the shoulder of the road. Forty-eight drivers (mean age of 73.5) had cognitive impairments caused by mild to moderate Alzheimer's disease, and 101 (mean age of 69.3 years) were neurologically normal. Results showed that compared with controls, drivers with Alzheimer's reacted more slowly ($P = 0.0008$)—with abrupt decelerations resulting—or failed to steer clear of the police car ($P = 0.0036$). Several older drivers stopped in the middle of the road. Poorer scores on neuropsychological tests of perception, attention, memory, and executive function predicted slower first reactions and increased the risk of inappropriate and potentially unsafe reactions. These results provide evidence that cognitive errors leading to unsafe driver behaviors can be tested safely in a simulator. The findings suggest that there is decreased situation awareness or poor executive control over response implementation in older drivers with cognitive decline, possibly at the level of selecting one of several possible learned evasive motor actions.

Safe driving performance requires the continuous coordination of several cognitive processes, including attention, perception, memory, and executive functions (decision making and implementation) (*1*). These processes are impaired in populations of drivers with cognitive disorders, increasing the risk of driver errors and motor vehicle crashes (*2–6*). Special concerns are often raised about fitness to drive in Alzheimer's disease (AD) (5), a progressive age-related cognitive disorder associated with neurofibrillary tangles, extracellular plaques, and neuronal loss in the brain.

Relationships between driver performance and safety errors can be represented by an imaginary iceberg (*7, 8*). Visible errors ("above the waterline") generally are associated with car crashes that result in fatality, serious injury, mild injury, or, most often, property damage only. Hidden errors ("below the waterline") are driver behaviors that theoretically are related to crashes and occur more frequently. Objective measurements of driver behavior in a range of traffic situations can reveal critical and hidden relationships between low-frequency, high-severity and high-frequency, low-severity driver errors (*9*). Better understanding of these relationships could improve predictions of driver safety in at-risk drivers with cognitive impairments and in normal drivers.

This study assessed how cognitively impaired elderly drivers respond to an emergency vehicle (a police car) parked by the side of the road, a relatively common (below the waterline) traffic situation that challenges driver perception, attention, memory, and executive functions. Typical state regulations indicate that the driver should decrease speed and steer clear when approaching and passing the stopped emergency vehicle, even on a two-lane bidirectional highway, as long as there is no oncoming traffic (*10*). Inappropriate reactions to stopped vehicles can lead to unsafe car–car and car–pedestrian interactions. These mechanisms and underlying cognitive factors were objectively addressed by using the following methods and materials.

METHODS

Subjects

Participants included 149 legally licensed elderly drivers participating in a larger study of driving performance in at-risk older drivers with cognitive impairments. Forty-eight participants recruited from a registry in the Alzheimer's Disease Research Center of the Department of Neurology, University of Iowa (mean ± SD age of 73.5 ± 8.9 years) had probable AD of mild to moderate severity. The diagnosis of AD was based on standard NINCDS-ADRA (National Institute of Neurological and Communicative Disorders and Stroke and the Alzheimer's Disease and Related Disorders Association) diagnostic criteria (*11*). One-hundred-one control participants without dementia were volunteers in the local community (mean ± SD age of 69.3 ± 6.6 years). All participants held a current, valid state driver's license, although some had reduced driving activities because of self- or family-imposed restrictions. AD subjects were older than control subjects ($P = 0.0055$, Wilcoxon rank sum test). All subjects participated in a battery of visual and cognitive tests and in a driving simulator scenario that tested driver response to a police car parked by the side of the road.

Vision Assessment

Letter acuity was measured separately in each eye and with both eyes open (OD/OS and OU) by using the Early Treatment Diabetic Retinopathy Study chart (*12*). Acuity was measured at far and near

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distance. Contrast sensitivity was assessed OD/OS and OU with the use of the Pelli chart (*13*). This test provides a measure of low to medium spatial frequency sensitivity (i.e., near the peak of the contrast sensitivity function).

Cognitive Assessment: Approach and Procedures

All participants were studied with a battery of standardized off-road neuropsychological procedures aimed at cognitive functions essential to the driving task. Clinicians who performed the cognitive assessment were masked to driving performance measures and hypotheses. The following standardized tests are included in the battery:

- Attention
  – Useful field of view (*14, 15*);
- Memory
  – Benton visual retention test (BVRT),
  – Complex figure test, recall version (CFT-Recall),
  – Rey auditory verbal learning test (Rey AVLT);
- Visuoperceptual and visuomotor functions
  – Judgment of line orientation (JLO) test (*16*),
  – Structure from motion (*17–19*),
  – WAIS-III block design (*20*),
  – CFT-Copy (*21*),
  – Grooved pegboard test;
- Executive functions
  – Wisconsin card sorting test (*22*),
  – Trail-making test (TMT), Parts A and B (*23*),
  – Controlled oral word association (COWA) test (*24*),
  – Overall cognitive function (COGSTAT).

A composite index of cognitive ability in each driver was calculated, similar to the work of Rizzo et al. (*25*). This composite measure, COGSTAT, was calculated by assigning standard T scores (mean = 50, SD = 10) to each of eight tests from the cognitive assessment battery: JLO, CFT-Copy, Blocks (WAIS-III), CFT-Recall, BVRT, Rey AVLT, TMT-B, and COWA.

A combination of techniques is desirable to assess patient mobility (*26*). This research used the "get up and go" test (2 min) and the functional reach test (2 min). These tests are theoretically based, well normed, sensitive to medical interventions, correlated with real-world outcomes, and easy to administer (*27–35*).

Driving Simulator Assessment

The effects of cognitive impairment on driver errors were studied safely and under strictly controlled conditions in the synthetic environment provided by a driving simulator. Driving simulation offers several advantages over the use of driving records and state road tests in assessments of driver fitness. Simulators provide the best means with which to replicate the road conditions under which driver decisions are made, and simulations are safe, without the safety risks of the road or test track.

In this study, a high-fidelity driving simulator known as SIREN (*36*) was used to assess response to an emergency vehicle. SIREN creates an immersive, real-time virtual environment for assessing at-risk drivers in a medical setting (*36*). SIREN comprises a 1994 GM Saturn, embedded electronic sensors, miniature video cameras for recording driver performance, a sound system and surrounding screens (150° forward field of view, 50° rear field of view), four LCD projectors with image generators, an integrated host computer, and another computer for scenario design, control, and data collection. A tile-based scenario development tool (DriveSafety, Fort Collins, Colorado) provided multiple road types and allowed roadways to be populated with different vehicles that interact with the driver and each other according to experimental needs.

Training

A warm-up and training phase lasting about 5 min preceded the experimental drive and was sufficient for adapting to vehicle controls (*37*). A research assistant familiarized the driver with the vehicle controls. A simulator operator communicated with the driver by intercom to monitor the driver for signs of discomfort or fatigue. Before the experiment was started, each driver was familiarized with the simulator by driving on a simulated two-lane highway.

Response to Emergency Vehicle

All drivers participated in a scenario that required response to an emergency vehicle (Figure 1). Response to an emergency vehicle such as a police car parked by the roadside requires detection and recognition of an object in peripheral vision, situation recognition, and a rapid response to execute a safety maneuver to decrease the potential for a crash with the vehicle or running over people situated near it.

Electronic Measures

Experimental performance data were digitized at 30 Hz and reduced to means, standard deviations, or counts for each virtual road segment. Simulator output includes steering wheel position (in degrees), normalized accelerator and brake position (i.e., scale of pedal depression from 0% to 100%), speed (mph), and other variables.

FIGURE 1 Over-shoulder view shows setup of driver in SIREN approaching police car parked by side of virtual road.

Video Data

Driving performance was captured (at 30 Hz) by using miniature cameras to record the scene observed by the driver and provide a backup record of the driver's lane tracking and to evaluate a subject's gaze in regions of interest in the car and on the virtual road. Synchronization of the digital and video data facilitates the inspection of artifacts and allows for review of potential driver safety errors.

Calculation of Dependent Measures

The region of interest about the police car was about 50 s, including approach and departure from the car. This included approximately 20,000 digital observations for each of the 149 drivers. There were three types of reaction to the car: (*a*) accelerator pedal release, (*b*) brake pedal application, and (*c*) lane position change. Figure 2 gives a schematic drawing of the police car scenario and reference points for calculating five dependent measures.

A main outcome of this experiment was the first reaction time. This continuous measure described how quickly (in seconds) the driver reacted to the police car by exerting control over the vehicle pedals or lane position. This reaction was reckoned from a point 500 m from the police car.

The second main outcome of the experiment was the occurrence of an inappropriate reaction, a binary variable. The optimal response when a driver encounters a police car parked on the right shoulder of a two-lane highway with no oncoming vehicles is to slow down and deviate to the left, away from the police car. Furthermore, the reaction must be smooth and continuous. That is, multiple sequential applications of the brake pedal, accelerator pedal, or both, or weaving back and forth across lanes, are indecisive and inappropriate reactions that are potentially unsafe (see Figure 3).

The other outcome measures were (*a*) the relative speed decrease during the reaction section, a continuous measure defined by an equation (maximal speed minus the minimal speed/maximum speed); (*b*) the amount of room (proportion of total available) left for a police officer or bystander (see Figure 2); and (*c*) inappropriate deceleration, a binary measure defined by failure to slow down or by inappropriate slowing (stopping on the road).

Statistical Analyses

Comparisons Between Groups

These analyses addressed all five driving simulation outcome measures, as well as the off-road laboratory measures. Crude comparisons relied on the Wilcoxon rank sum test for continuous driving outcome and laboratory outcome measures and Fisher's exact test for binary outcome measures. Age-adjusted comparisons relied on multiple linear regression for continuous driving outcome and laboratory outcome measures and on multiple logistic regression for binary outcomes.

Identifying Significant Predictors of Two Main Outcome Measures

Associations between individual lab measures and the first reaction time measure in the driving simulator scenario relied on Spearman correlation tests. Multiple linear regression was used to assess these associations and adjust for age. Associations between individual lab measures and the inappropriate reactions used univariate logistic regressions. Multiple logistic regression was used to assess these associations and adjust for age.

Model Building

Model building involved applying backward elimination linear or logistic regression variable selection procedures among all lab measures (including age) on the two main outcome measures.

RESULTS

Comparisons Between Groups

Table 1 shows that drivers with AD performed significantly worse on all off-road lab measures, even after adjustment for age. Table 2 shows that drivers with AD had slower response to the police car,

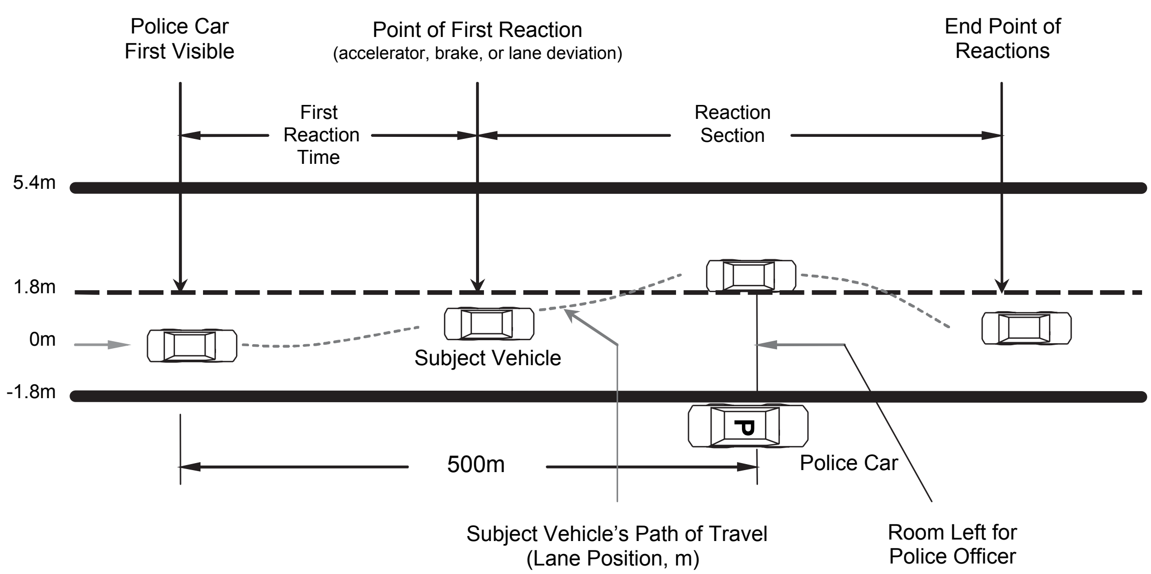

FIGURE 2 Response to emergency vehicle parked roadside.

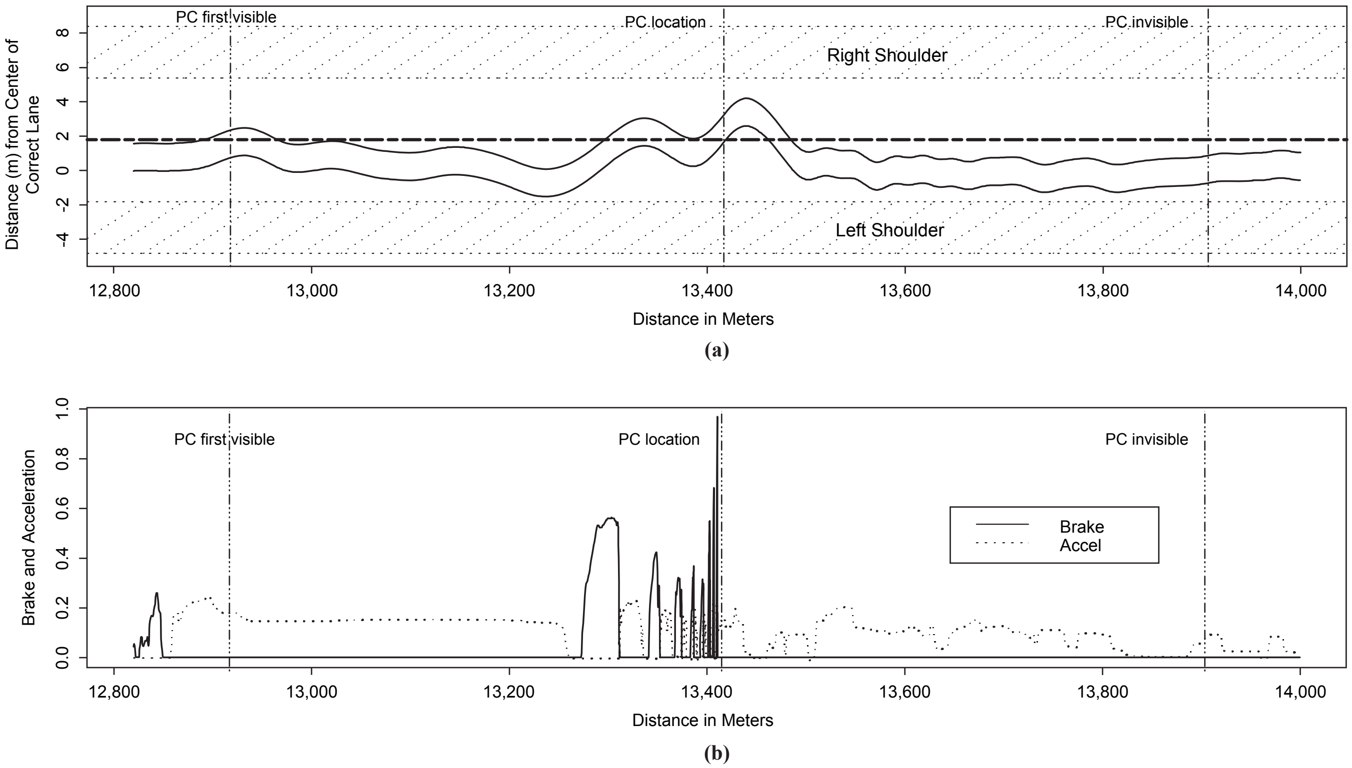

FIGURE 3 Inappropriate reactions in two different drivers: (*a*) inappropriate lane deviation and (*b*) inappropriate acceleration or brake reaction (0 = no application, 1 = full application).

were more likely to react unsafely, and reduced speed less, even after age adjustment. They were more likely to stop on the road unsafely or to fail to slow down, before adjustment for age.

Identifying Significant Predictors of Two Main Outcome Measures

A variety of off-road visual, cognitive, and mobility factors correlated significantly with the two main outcome measures. Tables 3 and 4 show univariate predictors of the first reaction time and of inappropriate reaction, including adjustments for age.

Model Building

Residual plots and normality tests were used to assess goodness of fit for the multiple linear regression model of first reaction time. All the necessary assumptions (e.g., normality, independence, constant variance) were met. Results showed that the first reaction time in the police car scenario was predicted simultaneously by the score (number of errors) on the Benton visual retention test (visual memory) and by far visual acuity. Inappropriate reactions were predicted by near and far visual acuity. The Hosmer and Lemeshow goodness-of-fit test (P-value = 0.5026) indicated the multiple logistic regression models fit the data well. (High P-values indicate good fit.

DISCUSSION OF RESULTS

This study showed that elderly drivers with mild to moderate AD had slower responses to the police car than did elderly controls. The drivers with AD also were more likely to make inappropriate reactions than were the controls. Univariate predictors of worse driving performances included impaired visual acuity, disordered attention, memory executive functions, overall cognitive decline, and motor slowing. Impairments of far visual acuity and visual memory were the most important factors among all the explanatory variables for slower firs reaction time. Impairments of far and near visual acuity were the most important predictors for the likelihood of making inappropriate reactions.

Response to an emergency vehicle in this study can be understood within a general framework in which the driver (*a*) perceives and attends to stimulus evidence (e.g., through visual inputs) and interprets the traffic situation; (*b*) formulates a plan based on the particular driving situation and relevant previous experience or memory; and (*c*) executes an action (e.g., by applying the accelerator, brake, or steering controls). The response is safe or unsafe as a result of errors at one or more stages in the driving task. The outcome of the behavior provides a source of potential feedback for the driver to take subsequent action.

Attention, Memory, Decision Making, and Driving Behavior

The risk of human errors in complex systems, such as a driver operating a motor vehicle, increases with deficits of attention, perception, and response selection (which depends on decision making and memory) and implementation (which depends on executive functions). Psychomotor factors and general mobility are also relevant (*38*). Individuals with deficits in these abilities are more likely than normal drivers to commit errors that cause motor vehicle crashes and injuries.

TABLE 1 Comparison of Measures Between Groups

	AD Group	Control Group		
Sample Size	48 12F, 36M	101 49F, 52M		
	Mean (SD)	Mean (SD)	Crude *p*-Value	Age-Adjusted *p*-Value
Demographics				
Age	73.46 (8.91)	69.33 (6.61)	0.0055	NA
Education (years)	14.71 (3.19)	15.55 (2.54)	0.0900	NA
Cognitive, visual, executive function and mobility measures				
CFT-Copy	26.78 (6.64)	31.45 (4.05)	<0.0001	<0.0001
JLO	22.43 (5.07)	25.05 (4.17)	0.0004	0.0016
WAIS-III block design	20.16 (11.77)	37.67 (9.83)	<0.0001	<0.0001
BVRT error	9.57 (3.47)	5.27 (2.58)	<0.0001	<0.0001
CFT-Recall	8.56 (4.34)	14.93 (5.50)	<0.0001	<0.0001
Rey AVLT	3.24 (2.72)	9.85 (3.04)	<0.0001	<0.0001
COWA	29.08 (13.18)	38.48 (11.61)	<0.0001	<0.0001
TMT-A	59.74 (34.58)	36.80 (12.54)	<0.0001	<0.0001
TMT-B	163.37 (77.40)	84.71 (36.15)	<0.0001	<0.0001
Contrast sensitivity	1.64 (0.21)	1.82 (0.15)	<0.0001	<0.0001
Far visual acuity	0.03 (0.15)	–0.06 (0.13)	0.0002	0.0040
Near visual acuity	0.06 (0.06)	0.02 (0.04)	<0.0001	0.0002
SFM	55.96 (245.3)	10.30 (2.55)	0.0017	0.0113
UFOV	1305.5 (291.9)	693.92 (222.2)	<0.0001	<0.0001
WCST preservative error	30.32 (24.32)	15.65 (12.48)	<0.0001	0.0002
WCST categories completed	3.43 (2.35)	5.15 (1.62)	<0.0001	0.0001
Grooved pegboard test	127.37 (48.46)	89.22 (18.40)	<0.0001	<0.0001
Get Up and Go test	11.23 (2.38)	8.83 (2.61)	<0.0001	<0.0001
Functional reach	11.56 (2.69)	13.36 (2.43)	0.0001	0.0046
COGSTAT	286.43 (57.18)	391.70 (41.82)	<0.0001	<0.0001

M = male, F = female.
Crude *p*-values were based on Wilcoxon rank sum test; age-adjusted *p*-values were based on multiple linear regression. Grooved pegboard test scores were calculated by taking the averages of the scores of both hand tests.
SFM = structure from motion, UFOV = useful field of view, WCST = Wisconsin card sorting test.

TABLE 2 Comparison of Driving Outcomes Between Groups

Dependent Variable	Group	Mean (SD) or *N* (%)	Crude *p*-Value	Age-Adjusted *p*-Value
First reaction time (continuous, sec.)	AD Control	12.08 (4.17) 9.70 (4.46)	0.0008	0.0091
Inappropriate reaction (binary)	AD Control	38 (79.17%) 54 (53.47%)	0.0036	0.0146
Relative speed decrease (proportion)	AD Control	0.47 (0.28) 0.58 (0.21)	0.0046	0.0030
Room left for a police officer (proportion)	AD Control	0.46 (0.20) 0.51 (0.20)	0.2215	0.4910
Inappropriate deceleration (binary)	AD Control	6 (12.50%) 3 (2.97%)	0.0315	0.2160

Mean (SD) includes means and standard deviations of continuous dependent variable and proportions for each group.
N (%) includes numbers (percentages) of subjects with worse values of binary outcomes within each group.
Crude *p*-values were based on Wilcoxon rank sum test (for continuous variables) and Fisher's exact test (for binary variables).
Age-adjusted *p*-values were based on multiple linear regression (for continuous variables) and multiple logistic regression (for binary variables).

TABLE 3 Univariate Predictors of First Reaction Time

Predictor Variable	r_s	Crude p-Value	Age-Adjusted p-Value
CFT-Copy	−0.18	0.0321	0.0700
JLO	−0.18	0.0358	0.0465
WAIS-III block design	−0.21	0.0128	0.0345
BVRT error	0.24	0.0033	0.0153
Rey AVLT	−0.26	0.0013	0.0307
COWA	−0.15	0.0768	0.0278
TMT-A	0.17	0.0413	0.4165
TMT-B	0.27	0.0011	0.0039
Contrast sensitivity	−0.20	0.0144	0.0856
Far visual acuity	0.16	0.0576	0.0250
UFOV	0.21	0.0110	0.0456
Get Up and Go test	0.21	0.0145	0.3808
Grooved pegboard test	0.17	0.0402	0.0679
WCST preservative error	0.20	0.0165	0.3243
WCST categories completed	0.30	0.0004	0.0070
COGSTAT	−0.27	0.0027	0.0080

r_s stands for estimated Spearman correlation coefficient.
Crude p-values were based on Spearman correlation tests; age-adjusted p-values were based on multiple linear regressions.
Grooved pegboard test scores were calculated by taking the averages of the scores of both hand tests.

Attention, memory, and decision making depend on overlapping neural systems to help with executive selection and scheduling of competing behavior choices and action plans (*39, 40*). Decision making requires evaluation of immediate and long-term consequences of planned actions and is often included with impulse control, insight, judgment, and planning under the rubric of executive functions (*41–45*). Impaired decision making is a critical factor in driver errors that lead to vehicle crashes (*2*). Causes include acquired brain lesions (due to stroke, trauma, or neurodegenerative impairment) affecting prefrontal areas, antisocial personality disorder, effects of drugs and alcohol (*46–49*), and fatigue (*50*). Executive functions strongly interact with working memory (the process of brief storage of information until it is available for use) and attention (operating on contents of working memory) (*51–53*) and are a key determinant of driver strategies and tactics. Driver strategies include a sequence of trips or stops (for gas, food, directions, or naps) and evaluation of traffic and weather risks. Tactics include adapting to speed changes near a school and making go or no-go decisions on whether to pass through intersections or cross traffic. Drivers with impaired decision making may also show impairments of impulse control, which is related to decision making (*54, 55*). Impulsiveness can be perceptual, cognitive, or motor (*56*). Cognitive impulsiveness reflects an inability to evaluate the outcome of a planned action and may give the appearance of failure to perceive or evaluate risk.

TABLE 4 Univariate Predictors of Inappropriate Reaction

Predictor Variable	OR	Crude p-Value	Age-Adjusted p-Value
WAIS-III block design (5-unit increase)	0.82	0.0053	0.0137
Rey AVLT	0.86	0.0008	0.0138
TMT-B (30-units increase)	1.22	0.0495	0.1605
Far visual acuity (0.02-unit increase)	1.12	0.0001	0.0014
Near visual acuity (>0 vs. 0)	3.11	0.0016	0.0163
UFOV (100-unit increase)	1.12	0.0234	0.0960
Get Up and Go test	1.19	0.0147	0.0876
Functional reach	0.87	0.0446	0.2881
WCST preservative error	1.03	0.0237	0.0508
WCST categories completed	0.79	0.0277	0.0643
COGSTAT (50-unit increase)	0.73	0.0394	0.0880

ORs were defined as the ratios of odds of making inappropriate reactions among the subjects with higher scores of cognitive, visual, or motor tests, relative to those among the subjects with lower scores.
Crude p-values were based on logistic regressions; age-adjusted p-values were based on multiple logistic regressions.

The scenario used in this study investigated situation awareness and response to a roadway emergency in at-risk drivers. Results showed that compared to controls, drivers with cognitive impairments reacted more slowly. This led to more abrupt decelerations, or failed to steer clear of the police car. Unsafe reactions were predicted by standard visual and neuropsychological measures of perception, attention, memory, and executive function. Several impaired drivers stopped in the middle of the road. The results provide evidence that cognitive errors leading to unsafe driver behaviors can be tested safely in a simulator. The findings suggest that there is decreased situation awareness or poor executive control over response implementation in older drivers with cognitive decline, possibly at the level of selecting one of several possible learned evasive motor actions. The pattern was different from what was observed in at-risk younger drivers.

Situation awareness and risk taking in the police car task were studied in 42 younger drivers between ages 21 and 42. These included 12 MDMA (3,4-methylenedioxymethamphetamine) and marijuana users (MDMA/THC), 15 marijuana users (THC), and 15 non-drug-using controls. Participants were asked to abstain from drug use on the day of testing. Although driving performance on uneventful segments did not differ between groups, the non-drug users slowed to a safer speed as compared to drug users during some of the events. Results showed that the drug users passed the police car parked on the shoulder of the highway at significantly higher speeds compared to the non-drug users. (These same at-risk younger drivers also crashed at higher speeds than non-drug-using younger drivers in an intersection incursion scenario.) In the study, the elderly drivers with AD passed the police car at a marginally higher speed than the elderly controls (37.29 mph for AD versus 34.85 mph for controls, $P = 0.0589$).

These results suggest that younger and older drivers can fail the same task for different reasons. A slower speed provides the driver with a greater margin of safety when suddenly required to respond to an unexpected event—for example, the opening of a door of the passed police car (Lamers, C. T. J., M. Rizzo, A. Bechara, and J. G. Ramaekers, unpublished.). Slowing and giving clearance to emergency vehicles on the roadside is a posted regulation in some jurisdictions (*57*). However, overcompensation (e.g., stopping on the roadway), as demonstrated by some of the older drivers, is maladaptive and risks a collision with moving cars. The errors in the older drivers can be interpreted as caused by declines in visual speed, attention, and cognition, including executive functions. In the study of the younger drivers, no low-level visual impairments or attentional impairments were found, but there were differences in executive function tests in the drug users related to the apparent risk taking and acceptance (*58*). MDMA and THC use was also associated with impaired perception of heading from optical flow (*59*). The findings are relevant to the risk in older and younger drivers who may take a variety of prescription medications (*60*), including cholinesterase inhibitors in AD.

AD and Driving Behavior

AD is the most common cause of abnormal cognitive decline in older adults (*61*). There is considerable evidence that the progressive disease process begins years before clinical diagnosis. Brain autopsies in 98 older drivers who died in vehicle crashes showed that 52 (53%) had sufficient neuritic plaques to fulfill standard neuropathological criteria of CERAD, the Consortium to Establish a Registry for Alzheimer's Disease, suggesting (20%) or indicating (33%) AD (*62*). That none of these drivers carried a diagnosis of AD while family members were often unaware of a problem (*63*) raises the concern that the first manifestation of AD may sometimes be a fatal crash. At-risk behaviors in these drivers must be detected to exert appropriate interventions before it is too late.

For this reason, Duchek et al. (*64*) administered road tests to 21 older adults with very mild AD and 29 with mild AD (scores of 0.5 and 1, respectively, on the Clinical Dementia Rating Scale) and 58 older drivers without dementia. With a driving expert's help, Duchek et al. rated drivers as safe, marginal (small to moderate crash risk, e.g., from driving too slowly), or unsafe (substantial crash risk, e.g., caused by ignoring a traffic light or stopping without a reason). Nearly half the drivers with mild AD failed the first driving test. Only 14% of those with very mild AD and 3% of controls failed. Most drivers with AD were judged to be unsafe at the initial testing or follow-up testing after 2 years. All groups were judged to decline in situations that depend on more complex cognitive skills involved in driving, such as awareness of the driving environment and decision making. Clinicians would benefit the most from tests that are correlated with driving abilities within mildly impaired individuals (*65*). However, subjective scoring and variations on traffic and road conditions increase the variability of road test scores, reducing the strength of relationships with neuropsychological test scores (*66*).

Reger et al. conducted a meta-analysis of 27 studies of drivers with dementia to examine relationships between driving abilities and neuropsychological functions (mental status and general cognition, attention and concentration, visual spatial skills, memory, executive functions, and language (*66*). When studies that used a control group were included, the relationship between cognitive measures and on-road or off-road driving measures was significant for all test domains. When studies that used a control group were excluded, moderate mean correlations were observed for visual spatial skills and on-road or off-road measures and for mental status with non-road tests. Reger et al. suggested caution when using neuropsychological tests for driving recommendations. Visual spatial defects should trigger evaluation of other risk factors but alone are not sufficient to recommend driver restriction (*66*).

CONCLUSIONS

Cognitive errors leading to unsafe driver behaviors can be tested safely in a simulator. By studying specific driving maneuvers in detail, it is possible to observe more closely how cognitive impairments relate to mechanisms of driver error. In this study, a variety of neuropsychological tests predicted driver performance and error, perhaps because any driving scenario (including response to a police car) depends on multiple cognitive domains, as do many neuropsychological tests, although one ability or another may predominate. The findings suggest that there is decreased situation awareness or poor executive control over response implementation in older drivers with cognitive decline, possibly at the level of selecting one of several possible learned evasive motor actions. The pattern in at-risk older drivers was different from what was observed in at-risk younger drivers.

In anticipation of clinical observational and interventional trials of driving (e.g., use of drugs, new vehicle designs, and driver-assist or warning devices), attempts must be made to standardize testing of driving, including driving simulator scenarios. The results can be used to maximize predictions of risk in studies of different populations of at-risk drivers at different institutions. These new developments should be incorporated into updated evaluation guidelines on driving in dementia, such as those of the American Academy of Neurology (*67*) and the American Medical Association and the National Highway Transportation Administration (*68*).

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Driver Identification of Landmarks and Traffic Signs After a Stroke

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A study was done to assess the ability for visual search and recognition of roadside targets and safety errors during a landmark and traffic sign identification task in drivers with stroke, that is, drivers who have had a stroke. Visual search for roadside targets during automobile driving can compete for a driver's cognitive resources and may impair driving, especially in drivers with cognitive impairment caused by stroke. Thirty-two drivers with stroke and 137 neurologically normal older adults underwent a battery of visual, cognitive, and motor tests and were asked to report sightings of specific landmarks and traffic signs along a segment of an experimental drive. The drivers with stroke identified significantly fewer landmarks and traffic signs and showed a tendency to make more at-fault safety errors during the task than did control subjects. Roadside target identification performance and safety errors were predicted by scores on standardized tests of visual, cognitive, and motor function. Drivers with stroke are impaired in a task of visual search and recognition of roadside targets whose demands on visual perception, attention, executive functions, and memory probably increased the cognitive load and worsened their driving safety.

Safe automobile driving requires a driver to perform multiple competing tasks and attend to a host of objects and ongoing events while simultaneously monitoring traffic with central and peripheral vision to avoid roadway hazards (*1*). Impairments in visual acuity and field increase crashes and traffic violations (*2*). However, drivers with certain neurological conditions may fail to perceive critical roadside targets and dangers even in the absence of a measurable field defect on standard perimetry or diminished visual acuity (*1*).

Stroke can affect processing of visual sensory cues and may produce attentional decline (*3, 4*) and inability to recognize landmarks (*5, 6*). These deficits can impair driver processing of safety-relevant visual information in a roadway environment (*7–9*). This includes difficulty perceiving landmarks and traffic signs that provide key information about a driver's route and about upcoming road hazards and safety regulations. To address this real-world problem, an instrumented vehicle (IV) was used to test the hypothesis that drivers with stroke, that is, drivers who have had a stroke—have impairments on a landmark and traffic sign identification task (LTIT). Whether impaired drivers would commit more safety errors under the influence of the cognitive load imposed by the LTIT, placing them at greater risk for a potential crash, also was tested. Finally, whether LTIT performance and safety errors could be predicted by visual and cognitive measures sensitive to decline in stroke was tested.

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METHODS

Subjects

Subjects were 32 participants with stroke in the chronic phase (>6 months after the stroke) with stable neurologic deficits and 147 neurologically normal control participants. Most of the strokes were ischemic. Thirteen stroke subjects had purely right-hemispheric lesions, 14 had purely left-hemispheric lesions, and lesion location was bilateral or nonhemispheric (i.e., in the cerebellum or brainstem) in the remaining five. Visual field testing by using a standard perimetry technique (frequency doubling perimetry visual field instrument, Zeiss/Humphrey Systems, Dublin, California, and Welch Allyn, Skaneateles Falls, New York) showed that no participant had a hemianopia. Stroke subjects comprised 20 men and 12 women, and controls comprised 74 men and 73 women. The male majority in the stroke group can be explained by two factors. First, among non-Hispanic white adults (96.6% of Iowa population), the stroke prevalence is 2.2% for men and 1.5% for women (*10*). Second, women in this age group had not traditionally been the main driver in the family and, probably, more readily relinquished their driving privileges once they developed stroke. The normal control group had slightly longer education (in years, mean ± SD = 14.0 ± 2.7 and 15.6 ± 2.6 for stroke and normal controls, respectively, $P = 0.0017$).

Participants with stroke were recruited from a registry in the Behavioral Neurology Division of the Department of Neurology, University of Iowa, Iowa City. Participation was completely voluntary, and the informed consent clearly expressed that the information regarding the subject's driving abilities would be kept confidential to the extent permitted by law and released only with the subject's written approval. No subject had active confounding medical or psychiatric conditions. Medical history and examination as well as computed tomographic and magnetic resonance imaging scans of the brain were obtained to identify stroke and exclude other lesions caused by demyelinating, traumatic, or neoplastic disease. No stroke subject received driving rehabilitation programs or used adaptive equipment. Control participants were recruited from volunteers in the local community. All participants held a current, valid state driver's license and were still driving, although some had reduced driving activity because of self- or family-imposed restrictions. Criteria for exclusion included alcoholism, depression, vestibular disease, and motion sickness. Informed consent was obtained in accord with institutional and federal guidelines for the safety and confidentiality of human subjects.

TABLE 1 Cognitive, Motor, and Visual Test Battery (*29, 30*)

Abbreviation	Full Name of the Test	Measured Function
AVLT	Rey Auditory Verbal Learning Test (*11, 12*)	Anterograde verbal memory
Blocks	Block Design subtest from the WAIS-R (Wechsler Adult Intelligence Scale-Revised) (*13*)	Nonverbal intellect
BVRT	Benton Visual Retention Test (*14*)	Visual perception, visual memory, and visuoconstructive abilities
CFT-Copy	Rey-Osterreith Complex Figure Test, copy version (*15*)	Visuoconstructional ability
CFT-Recall	Rey-Osterreith Complex Figure Test, recall version (*15*)	Nonverbal anterograde memory
COWA	Controlled Oral Word Association Test (*16*)	Executive functions using a verbal task
CS	Contrast sensitivity (*17*)	Low to medium spatial frequency sensitivity
FVA	Far visual acuity (*18*)	
NVA	Near visual acuity (*18*)	
JLO	Judgment of line orientation (*19*)	Visual perception and spatial abilities
SFM	Perception of 3-dimensional structure-from-motion and of motion direction (*20*)	
TMT-B	Trail-Making Test Subtest B (*21*)	Executive functions using a nonverbal task
UFOV	Useful field of view (correlates with increased crash risk in a simulated driving scenarios and real life crashes) (*22, 23*)	Speed of visual processing, divided attention, and selective attention
GUG	Get-Up-Go (*24, 25*)	Mobility
FR	Functional reach (*26*)	Mobility
PEG-R, PEG-L	Grooved pegboard task, using right or left hand (*27, 28*)	Motor dexterity

Cognitive and Visual Battery

All participants were tested on the same batteries of cognitive, motor, and visual tasks (Table 1; *11–30*). These tests were chosen as representative of key cognitive domains that are important for safe driving, as in previous work (*31*). Figure 1 shows an information-processing model of the role of perception, attention, memory, decision making, and feedback in driving (*31*). Measured functions included verbal memory (AVLT-Recall) and visual memory (CFT-Recall, BVRT), executive functions (TMT-B, COWA), visual perception (FVA, NVA, CS, JLO), visual attention (UFOVTOT), visuoconstructional abilities (CFT-Copy, Blocks), and overall cognitive function (COGSTAT). COGSTAT, a composite measure calculated by assigning standard T scores (mean = 50, SD = 10) to each of the eight tests from the neuropsychological assessment battery (COWA, CFT-Copy, CFT-Recall, AVLT-Recall, BVRT, Blocks, JLO, and TRLB-T), was chosen as a gauge of overall cognitive impairment, as in previous work (*31, 20*). Standardization of these scores allowed us to generate an equally weighted composite score due to homogeneity of variance of each test score. Motor functions were tested by using the "get up and go" (GUG), functional reach, and grooved pegboard (PEG) tests.

Experimental Drive

The experimental drive was conducted aboard an instrumented vehicle known as the Automobile for Research in Ergonomics and Safety (ARGOS), a mid-sized 1995 Ford Taurus station wagon with an automatic transmission and with hidden instrumentation and sensors (*32–35*). Data on steering wheel position, normalized accelerator and brake pedal position, lateral and longitudinal acceleration, and vehicle speed were obtained at 10 Hz. A driver's lane tracking and visual scanning activity of the environment were videotaped at 30 Hz.

The experimenter sat in the front passenger seat to score the on-road performance and operate the dual controls in case of emergency. The experimental drive lasted approximately 45 min and started after the

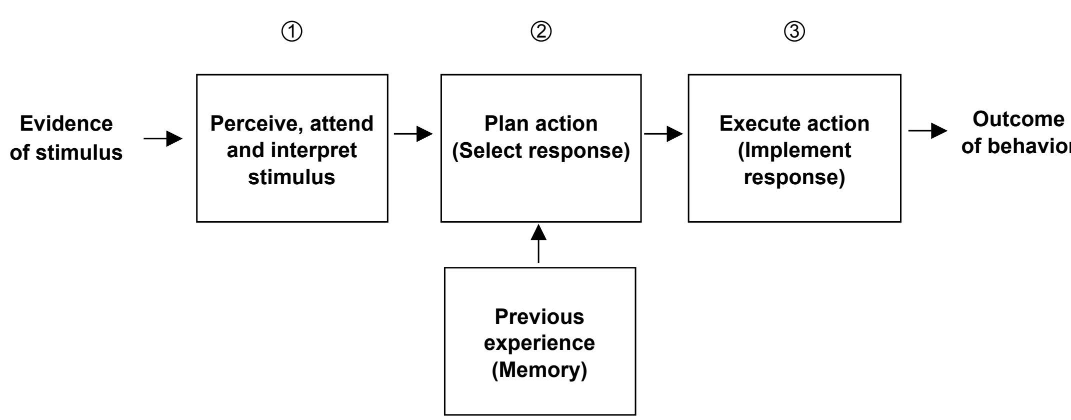

FIGURE 1 Information processing model for understanding driver error.

driver acclimated to ARGOS. Specifically, the road test in ARGOS was preceded by driver screening at curbside to test several fundamental requirements for driving, including locating the vehicle's controls and signals, inserting the key in the ignition, starting the car, shifting from park to drive, driving forward 20 m, and stopping. No participants failed the screening protocol. The experimental drive consisted of on-task (e.g., while performing LTIT) and no-task segments. Road testing was carried out only during daylight and in good weather on specific roads surrounding Iowa City.

Landmark and Traffic Sign Identification Test

LTIT was administered as part of a sequence of on-the-road tasks in ARGOS. Drivers were asked to look for and report verbally on traffic signs and restaurants (a highly ubiquitous type of roadside landmark) along a 1-mi commercial segment of a four-lane divided highway approximately 1 min before these stimuli started to appear. These targets were classified as high-saliency or low-saliency stimuli as based on ratings and detection rates of drivers tested in pilot studies. For example, a speed limit sign is a high-saliency stimulus detected by nearly all subjects, whereas a small and low mile marker is of lower saliency and is missed by some normal subjects. Likewise, a restaurant situated right on the road in its own building is a high-saliency stimulus, whereas a deli within a grocery store, or a restaurant whose sign or building can be seen afar but is not on the route is considered a lower-saliency stimulus. There were 16 road signs (11 high-saliency) and 13 restaurants (six high-saliency) along the route. Dependent measures were percentage of landmarks and traffic signs identified and number of at-fault safety errors, such as erratic steering, lane deviation, shoulder incursion, stopping or slowing in unsafe circumstances, and unsafe intersection behavior during the task.

Statistical Analysis

The stroke and control groups were compared for demographic, visual, cognitive, and LTIT outcome measures by using the Wilcoxon rank sum test. Fisher's exact test was used to compare the proportion of drivers in each group who had good outcomes (defined as equal to or better than the median of the control group for a particular variable), that is, who identified more than 60% of all targets and less than 80% of high-saliency targets and made no at-fault safety errors. Demographic factors and cognitive, motor, and vision test scores of drivers with stroke who had good outcomes versus those with stroke who had bad outcomes were compared.

Because motor impairment is one of the most common and clinically identifiable complications of stroke, the effects of right- and left-sided upper extremity motor impairment (defined as PEG performance on that side worse than 150 s) and gait impairment (define as GUG worse than 12 s) on LTIT outcomes were analyzed by comparing impaired versus nonimpaired subjects within the stroke group in these functions.

Multiple linear regression models were used to adjust the LTIT outcome comparisons for age, gender, visual acuity, and previous familiarity with the route, and Spearman correlation coefficients were calculated for the LTIT outcome measures with the cognitive and visual tests. Stepwise linear regression was used to identify predictors of total landmark and traffic sign identification percentage and ordinal logistic regression for at-fault safety errors. First, the individual components of COGSTAT were used to model for cognitive predictors, and this was followed by modeling for visual variables that was adjusted for general cognitive performance by using COGSTAT in the model. This showed the overall effect of the main two classes (cognitive and visual) of variables.

RESULTS

Drivers with stroke identified a significantly smaller percentage of restaurants and traffic signs during LTIT than did the neurologically normal controls (Table 2). The difference between the groups on LTIT performance persisted after adjustment for familiarity with the neighborhood, education, visual acuity, and gender, although the level of significance was reduced some after adjustment for familiarity. The stroke group also showed a tendency to commit more at-fault safety errors during the task (Table 2). After adjustment for familiarity, group difference in at-fault safety errors became significan (Table 2). Stratification by familiarity revealed that among subjects who were not previously familiar with the neighborhood where the experimental drive was conducted, the stroke group committed more at-fault safety errors ($P = 0.0160$, Wilcoxon rank sum test), whereas there was no significant difference between the groups in the familiar subcategory ($P = 0.2535$).

On a straight segment of the drive with no task load, there was no difference between the groups in basic vehicular control measured by

TABLE 2 Outcome Measures of LTIT Expressed as Means, with Comparison by Wilcoxon Rank Sum

	Stroke ($N = 32$) 9 familiar	Control ($N = 147$) 115 familiar	*p*-Values Crude	*p*-Values Adjusted for Familiarity
Landmark identification %				
All	32.7(17.8)	45.0(16.4)	0.0001	0.0153
High saliency only	51.0(21.1)	66.4(21.0)	0.0001	0.0135
Traffic sign identification %				
All	49.4(22.4)	70.7(17.5)	<0.0001	<0.0001
High saliency only	57.3(25.8)	82.4(17.7)	<0.0001	<0.0001
Total LTIT identification %				
All	41.9(15.1)	59.0(13.8)	<0.0001	<0.0001
High saliency only	55.2(18.6)	77.0(15.5)	<0.0001	<0.0001
# at-fault safety errors	0.8(1.0)	0.4(0.8)	0.0641	0.0066

SD of steering wheel position (degrees), number of large (>6°) changes in steering wheel position per minute, and SD of mean speed (data not shown).

The stroke group performed worse than the control group on several neuropsychological, motor, and visual tests, showing mild to moderate cognitive and visual perception and processing deficits compatible with their heterogeneous lesion localization (Table 3). Four stroke subjects could not perform PEG with the left hand because of hemiparesis. Measures of verbal (AVLT-Recall) and nonverbal (BVRT) memory, executive function (TMT-B, COWA), visual perception (FVA, NVA, CS, JLO), visual attention (UFOVTOT), visuoconstructional abilities (CFT-Copy, Blocks), and overall cognitive function (COGSTAT) and motor tests correlated significantly (Spearman coefficients) with the outcome measures of LTIT (Table 4). Regression analyses showed that TMT-B ($P = 0.0002$), JLO ($P = 0.0043$), and AVLT-Recall ($P = 0.0063$) predicted total landmark and traffic sign identification percentage for all targets. TMT-B ($P = 0.0013$), JLO ($P = 0.0003$), AVLT-Recall ($P = 0.0210$), and COWA ($P = 0.0497$) predicted identification rate of high-saliency (easy-to-recognize) targets. The GUG measure of mobility was the only predictor ($P = 0.0011$) of a higher number of at-fault safety errors.

TABLE 3 Comparison of Stroke and Control Groups

	Stroke	Control	
Sample Size	32	147	
	20 M, 12 F	74 M, 73 F	
	Mean (SD)	Mean (SD)	*p*-Values
Demographics			
Age	60.9 (12.8)	65.1 (11.5)	0.1067
Education (years)	14.0 (2.7)	15.6 (2.6)	0.0017
Cognitive tests			
COGSTAT	352 (59)	403 (46)	<0.0001
AVLT-Recall	8.2 (4.4)	10.2 (3.1)	0.0205
BVRT (error)	7.0 (4.4)	4.5 (2.6)	0.0004
CFT-Recall	15.0 (5.9)	15.9 (5.8)	0.5683
JLO	23.7 (4.5)	25.8 (3.9)	0.0109
Blocks	28.7 (12.7)	40.0 (10.8)	<0.0001
CFT-Copy	30.1 (3.8)	31.8 (3.8)	0.0066
TMT-B	120 (67)	81 (41)	0.0001
COWA	29.1 (10.0)	39.1 (11.1)	<0.0001
Visual tests			
NVA	.048 (.085)	.021 (.041)	0.0160
FVA	–.084 (.114)	–.085 (.126)	0.7908
CS	1.80 (.15)	1.84 (.15)	0.1154
SFM	11.8 (3.5)	10.3 (2.8)	0.0221
UFOVTOT (msec)	916 (401)	634 (245)	0.0004
Motor tests			
PEG-R (sec)	102.8 (38.6)	81.5 (18.8)	0.0010
PEG-L (sec)	109.7 (45.5)	89.1 (23.2)	0.0002
GUG (sec)	11.1 (3.7)	8.6 (2.3)	0.0001
FR (inches)	11.8 (3.7)	13.4 (2.7)	0.0494

M = male, F = female.

TABLE 4 Significant Spearman Correlation Coefficients for Outcome Measures of LTIT

	Variables	R_s	*p*-Values
Total LTIT identification % (all targets)	TMT-B	–0.34	<0.0001
	UFOVTOT	–0.33	<0.0001
	Blocks	0.32	<0.0001
	AVLT-Recall	0.30	<0.0001
	COGSTAT	0.30	<0.0001
	CS	0.30	<0.0001
	PEG-L	0.28	0.0001
	PEG-R	0.28	0.0002
	FVA	–0.26	0.0006
	JLO	0.25	0.0006
	COWA	0.25	0.0007
	BVRT	–0.20	0.0088
	CFT-Copy	0.17	0.0258
	FR	0.17	0.0299
	NVA	–0.16	0.0303
At-fault safety errors	GUG	0.19	0.0117
	CFT-Copy	–0.15	0.0452

The proportion of subjects with good outcomes for target identification was higher in the control group ($P < 0.0001$) (Figure 2). Within the stroke group, several tests on the battery could distinguish between subjects with good and bad outcomes. The stroke subjects who identified >60% of all targets scored better on COWA ($P = 0.0036$, Wilcoxon rank sum test) than those stroke subjects who identifie ≤60% of targets. The stroke subjects with no safety errors performed better on CFT-Recall ($P = 0.0120$) and GUG ($P = 0.0325$) than those stroke subjects with safety errors.

Subjects with right hemispheric strokes performed more at-fault safety errors ($P = 0.0299$) and tended to identify fewer targets ($P = 0.0743$ for all targets, $P = 0.0999$ for high-saliency targets). Stroke subjects with poor right PEG performance (>150 s) identified a smaller percentage of targets compared to stroke subjects with normal R-PEG performance, for both all and high-saliency targets, $P = 0.0064$ and $P = 0.0028$, respectively. The L-PEG-impaired group did not differ from the nonimpaired group in LTIT outcomes. Stroke subjects with

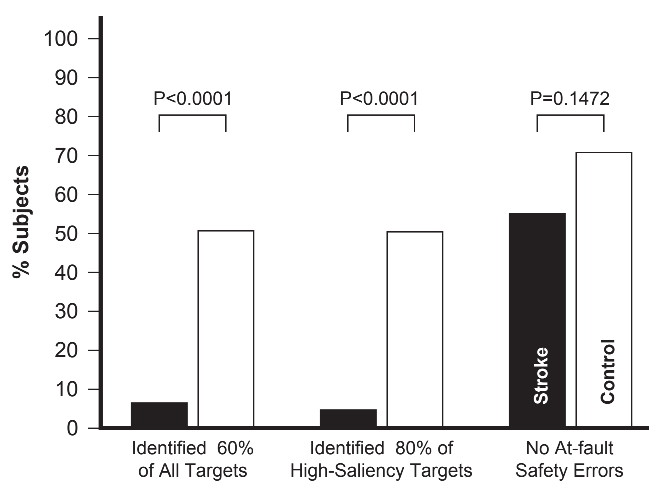

FIGURE 2 Proportion of subjects with good outcomes in stroke and control groups.

poor GUG performance (>12 s) committed more at-fault safety errors ($P = 0.0116$) than stroke subjects with normal gait.

DISCUSSION OF RESULTS

The findings in this study support the hypothesis that drivers with stroke perform worse than neurologically normal drivers on an LTIT. These group differences could not be explained by age, gender, or education differences between the groups (Table 2). Performance on standardized tests of visual perception, visual processing and attention, verbal and visual memory, executive functions, visuoconstructional abilities, and motor functions (Table 3) correlated with the LTIT outcome measures (Table 4) consistent with its demands on vision and cognition. These demands probably increased the cognitive load during driving, which may explain the higher number of at-fault safety errors measured in those drivers who have limited cognitive resources, especially those unfamiliar with the neighborhood in which the test was performed.

Driving performance and safety errors in driving with stroke is an active research topic (*7, 8, 36–38*) because stroke is a relatively common medical problem (*10*). As of 2002, 4,600,000 stroke victims were living in the United States (*10*). Although stroke can occur at all ages, it is more prevalent in older individuals (*10*). The number of drivers who have had a stroke is increasing because of general tendencies of aging in the United States (*39*). Impaired visual search and object recognition (traffic signs and landmarks in this paper) can be seen in strokes affecting various regions in the brain (*3–6*). No known study has tested landmark and traffic sign identification in stroke subjects during actual driving, as in the present study.

JLO predicted LTIT performance indicating that higher-order visual processes are important in performing this task of visual search and object recognition during driving. The finding that AVLT-Recall predicted LTIT underscores the role of verbal memory in the identification of the traffic signs because U.S. road signs are mainly word (rather than symbol) based (*40*).

The LTIT requires executive control over attention to switch between the tasks of searching for landmarks and controlling the vehicle on the road. Regression modeling showed that the TMT-B and COWA are independent predictors of LTIT performance. COWA measures verbal fluency and is a test of executive functions and language. TMT depends on cognitive flexibility to switch attention between two competing tasks (tracking numbers and letters) and is considered a task of executive function (*29*).

The results of regression modeling showed that gait impairment predicted at-fault safety errors, and stroke subjects with gait impairment committed more safety errors. Decreased dexterity measured by PEG correlated with poor identification of all targets and also high-saliency targets. There were no stroke subjects with right hemiparesis in the sample, perhaps because patients with involvement of the dominant hand in conjunction with a greater likelihood of communication difficulties caused by aphasia are less likely to drive after a stroke. Motor impairments on the PEG and GUG contributed to decline in LTIT performance. These tests are theoretically based, well normed, correlated with real-world outcomes, and easy to administer (*24–26*), and the results suggest that they can help to assess aspects of driver safety risk after a stroke.

Both verbal (e.g., COWA, AVLT-Recall) and nonverbal impairments correlated with the LTIT performance (Table 4), indicating the importance of both language-related and visuospatial impairments for the successful completion of the task, implicating structures within the left and right hemispheres. Of note, subjects with right-hemisphere lesions made more at-fault safety errors and showed a tendency to identify fewer targets, consistent with the predominant role of the right hemisphere in visual perception and visuospatial abilities, which are critical for the LTIT performance.

Prior familiarity with the neighborhood was a mitigating factor in this study as in previous studies on route following in Alzheimer's disease and stroke (*34, 35*). It was found that drivers with stroke who reported previous familiarity with the area of town in which the LTIT was conducted committed a similar number of at-fault safety errors as the controls, whereas "unfamiliar" stroke subjects committed more safety errors than unfamiliar controls. This could be explained by the greater cognitive burden of performing LTIT on unfamiliar roads in stroke subjects with reduced cognitive capacity. This finding suggests that graded licensure policies that allow driving in a familiar neighborhood can be considered in stroke subjects.

A subset of drivers with stroke performed similarly well on all LTIT, and some made no safety errors, suggesting that some individuals with stroke remain fit drivers and may be allowed to continue to drive (*41*). The current findings indicate that the approach to assessments of driver fitness in at-risk drivers with cognitive impairment can benefit from introduction of controlled challenges of vision, perception, memory, and attention during driving scenarios implemented in an instrumented vehicle. Future work in this area must address the relationship between relatively high-frequency, low-severity safety errors, such as were measured in ARGOS, and low-frequency, high-severity incidents, such as injurious crashes in epidemiologic records.

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Use of Video Intervention to Increase Elders' Awareness of Low-Cost Vehicle Modifications That Enhance Driving Safety and Comfort

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Use of vehicle modifications might enable older drivers to stay on the road safely and provide a more gradual pathway between driving and driving cessation. Although adaptive features for automobiles have long been known by the disabilities community, they have not been well known by professionals in the network of aging programs and services or by elders themselves. This study examined the use of a video intervention to increase elders' awareness of low-tech vehicle features. It was hypothesized that participants would have an increased awareness after viewing a video and would take steps toward using the features. The 23-min video was shown to 157 drivers age 70+ at seven Councils on Aging and senior centers in Massachusetts. The median age was 79, and 11% were age 85 and older. The participants completed pre- and postvideo questionnaires, and 127 of the participants (81%) were surveyed by phone approximately 2 months later. Familiarity significantly increased for 10 of 13 demonstrated features. On average, participants had taken two of five follow-up steps: 85% had read handouts, 63% had discussed features with family or friends, 20% had looked for features in stores or on the Internet, 9% had tried features, and only 2% had contacted a professional. Eleven percent of the telephone interviewees had purchased features. The video served the intended purpose of increasing awareness of vehicle modifications. Although some change was noted, 2 months may not have been sufficient time to observe change for the majority of the participants. Moreover, it is not known to what extent the participants may draw on this knowledge in the future, if and when they perceive a more direct need.

Driving is a complex skill requiring precise coordination of numerous physical, visual, and cognitive abilities. Some of these abilities tend to decline with aging (*1–3*). Reductions in height, strength, and flexibility, for example, can make it difficult to see over the steering wheel, press on the pedals, and turn one's head to look for traffic in adjacent lanes. Reduced peripheral vision also makes it more of a challenge to see vehicles in adjacent lanes, and increased sensitivity to glare, including longer recovery time, may result in reduced ability to see the road. Moreover, sensory and tactile changes can increase sensitivity to pressure from the safety belt, and slower processing of information can reduce the ability to judge distances accurately.

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Although older drivers may have some driving challenges because of these functional limitations, for safety on the road, older drivers are "not a significant risk to others" (*4*). Rather, they are a risk to themselves. This demographic age group has the highest fatality rate per mile driven, other than young (under age 25) drivers (*5*). Johns Hopkins University and the Insurance Institute for Highway Safety attribute this high fatality rate to elders' frailty (*4*). The seriousness of the situation has resulted in the American Medical Association designating the safety of elder drivers a public health issue (*5*).

One means of addressing this public health issue is to find ways to compensate for functional deficits attributed to aging that may impair safe driving. Although some older drivers may have to stop driving, others may benefit from using various features designed to keep drivers safely on the road (*6, 7*). Although helpful features have been on the market for some time and have long been known by the disabilities community, such features are not well known to the general public, to professionals in the network of aging programs and services, or to elders themselves.

The specific goal for this study is to examine one method, the development and use of a video intervention, to increase elders' awareness of low-tech features that may be helpful in driving. These features are designed to help compensate for the effects of functional deficit commonly occurring in aging and may be beneficial for people of all ages with either long-term or temporary impairments. It was hypothesized that elders who viewed the video would have increased awareness of the demonstrated features and would take steps toward using one or more of the features.

METHOD

To determine the content of the video, the researchers first consulted with experts in driver rehabilitation and van conversion and with occupational therapists. Several databases were searched. The discussions with experts and the literature search yielded numerous functional deficits and consequent driving challenges noted as commonly occurring in the later years of the lifespan as well as more than 50 features that might alleviate the challenges. The process of choosing the features is fully described elsewhere (*8*). The researchers developed a number of criteria to help them reduce the features to a manageable number for the video demonstration. Features would generally have to be currently available, affordable, easily understood, comfortable and convenient to use, common, and effective and safe in the opinion of the experts or as evidenced in the literature.

Three definitions are important for terms that will be used throughout this paper:

- Functional deficits—problems or impairments that prevent normal functioning, for example, reduced peripheral vision caused by changes in the eye.
- Driving challenges—problems or difficulties with driving caused by functional deficits, for example, increased difficulty seeing vehicles in adjacent lanes and parking because of reduced peripheral vision.
- Features—products or items that can be used to address the driving challenge caused by the functional deficit, for example, convex side- and rearview mirrors that enable the driver to see to the side and rear without depending on peripheral vision. The researchers intentionally used the term “feature” rather than “vehicle modification because they believed that the latter term was overly associated with disabilities. And the term “modification” itself may be limiting in reaching the general aging population. The reader, however, may think of features and modifications as interchangeable (*8*).

Sixteen professionals in transportation, safety, law enforcement, aging, and related fields were asked to complete rating tools on 18 driving challenges and 30 features that may help to alleviate challenges; 10 of those asked participated. In addition, a focus group of 11 drivers age 70+ was invited to respond to a list of 12 proposed features. The experts’ responses on the features varied, and some outside the rehabilitation field were hesitant to complete the form. Several of the focus group members had limited knowledge of any of the features. These responses were consistent with the researchers’ assumptions of the general public’s lack of awareness of these features.

Additional discussions with experts before, during, and after film ing resulted in the following 13 features, addressing 11 driving challenges, demonstrated in the video (some of these features are illustrated in Figure 1):

- Visor extender reduces glare from the sun;
- Convex side-view mirrors help to eliminate blind spots;
- Convex rearview mirror helps to eliminate blind spots;
- Seat cushion raises the driver for an obstruction-free line of sight;
- Pedal extenders put pedals closer to the driver so a short person can reach the pedals (cushion may raise legs so driver cannot reach pedals);
- Support handle helps for getting in and out of the car;
- Ceiling hand grip helps for getting in and out of the car (driver can use arm strength to move in and out of the car, especially if there is weakness in the lower body);
- Safety belt extender facilitates fastening of the safety belt by raising the receptacle (also good for large people);
- Ribbon for safety belt is used to pull the safety belt over the shoulder;
- Safety belt adjuster positions the safety belt for easier reach;
- Safety belt pad, soft cloth, or fleece covers a portion of the safety belt for a more comfortable fit
- Key extender is useful in inserting the key into the ignition and turning, for people with arthritic hands; and
- Trash bag or silk scarf is used as a seat cover for sliding in and out of the car.

The final 23-min video was titled *Keep Moving Longer: Features for Safe Driving*. A well-known Boston television health reporter was engaged to give the introduction and closing to the video. The central section of the video depicts a certified driver rehabilitation specialist

(a)

(b)

(c)

(d)

FIGURE 1 Sample vehicle features: (*a*) pedal extenders, (*b*) visor extender, (*c*) support handle, and (*d*) safety belt adjuster.

demonstrating 13 vehicle safety features to three older female drivers who themselves were facing some of the driving challenges that were mentioned. The video had four components: (*a*) introduction and closing by the celebrity health reporter; (*b*) demonstration of features by the certified driver rehabilitation specialist to three amateur actresses—two age 80 and older and one age 70 and older; (*c*) summary photos with voice-over of each feature; and (*d*) resource information including websites and toll-free numbers. Early versions of the video were shown to the focus group members for comment and to two experts in transportation. The video underwent numerous edits before finalization

Data Collection

In the next phase of this research project, the video was shown to 157 drivers age 70 and older at seven Councils on Aging and senior centers in eastern Massachusetts. The sites were recruited as community partners during the proposal stage of the research. All sites represented suburban communities with limited public transportation options. The participants completed questionnaires immediately before (pretest) and after (posttest) they watched the video, and 127 (81%) of the participants responded to a telephone follow-up survey approximately 2 months later. The loss of 30 participants between the video viewing and the telephone follow-up can be attributed to a variety of reasons: about half had indicated they did not wish to be recontacted; some could not be reached after several attempts; others declined the follow-up interview when telephoned, or the participant had no recollection of the event so the call was terminated by the researcher.

Results

Sample Description and Driving History

Participants ranged in age from 70 to 93 years. The median age was 79, and 11% were age 85 or older. Two-thirds were women (67%). This was a well-educated sample: two-thirds (68%) had some college education; 34% had college degrees. Participants reflected a range of incomes: 17% of those responding to this question reported incomes under $20,000; 36% reported incomes between $20,000 and $40,000; and 26% reported incomes above $40,000.

Table 1 presents participants' driving histories. Most participants reported driving five or more days per week. Although most participants reported driving modest distances, nearly a quarter reported driving in excess of 100 mi a week. Half the participants reported usually driving alone. When they do have passengers, however, these are most often friends (67%) or spouses (36%), and nearly a quarter also drive grandchildren. Women (82%) were more likely than men (35%) to report driving with friends (chi square = 31.9, $p < .001$). Driving alone was more common among older than younger participants ($t = 2.3$, $p < .05$).

This sample reported a range of types of cars driven. The most frequently reported manufacturer of cars driven was Toyota (21%), followed by Chevrolet (12%), Ford (12%), Buick (9%), Oldsmobile (8%), and Mercury (7%). Model years ranged from 1970 to 2004. The median model year was 1998; thus, 50% of the respondents had cars that were six or more years old. Nearly all participants reported that they were the primary drivers.

Nearly all (91%) participants reported always wearing safety belts. The few elders who did not wear their safety belts all the time offered the following comments: "I dislike restraints"; "I only go on short trips—3 mi or less and 30 mph or less"; "I'm short and the seat belt comes right across my neck"; "I don't think to buckle up all of the time—I know that's a very poor excuse!"

TABLE 1 Driving History of Participants

Driving Characteristic	Percentage
Driving frequency	
1–4 days/week	20
5+ days/week	80
Average weekly mileage	
<50 miles	37
50–99 miles	41
100+ miles	22
Usually drive alone	50
If passengers, relationship to driver (all that apply)	
Friends	67
Spouse	36
Grandchildren	24
Other relatives	20
Always wear safety belt	91
Recent traffic warnings or violations	5
Recent auto accident	10
Taken driving refresher course	19
Any personal driving concerns	67
Driving concerns voiced by others	6

N = 157

Ten percent reported accidents and 5% reported driving citations within the last 2 years. The following explanations about accidents were offered: "A large SUV drove into me"; "I was hit twice backing out of a parking space"; "I hit a motorcycle at an intersection"; "I was in the wrong lane trying to exit a rotary"; "While crossing an intersection, I was hit on the left side of my vehicle by a driver turning left."

The following comments about receiving warnings or citations were also provided: "I was stopped going through a traffic [red] light"; "I drove through a crosswalk while a pedestrian was entering"; "I misjudged the distance and happened to pull out in front of a police car—I was ticketed and later excused."

Nearly a fifth (19%) of the participants reported having previously taken driving refresher courses, such as those offered through AARP. (Nine more participants took such a class after seeing the video.)

Driving Concerns and Self-Imposed Driving Limitations

The majority of participants (67%) indicated being concerned with at least one of 14 factors related to driving that were listed on the pretest questionnaire: vision, memory, alertness, physical flexibil ity, hearing, reaction time, attention span, cognitive ability, medical conditions, medications, mental health, upper body strength, lower body strength, and potential for driving crashes. On average, participants indicated concern with 2.3 (SD: 3.1) of these factors (range from 0 to 14). The most frequently mentioned concerns were vision (especially night vision) (36%), reaction time (28%), crash potential (24%), hearing (23%), and alertness (22%). Several participants noted additional factors of concern beyond the 14 that were specificall included: night driving when encountering (the glare from) blue xenon headlights approaching from the opposite direction; drivers who use cell phones while driving; backing up; white lines that are not repainted by the town—especially curbs at dangerous intersections; and road rage.

Participants were asked to indicate personal restrictions they placed on their driving. The majority of respondents (64%) reported placing one or more restrictions on their driving (see Table 2.) An average of 2.2 (SD: 2.4) restrictions were reported from among those who placed any restrictions on their driving. More than 40% of the participants reported restricting their driving in bad weather and at night. In addition to the list of 11 restrictions provided from the literature, respondents offered additional self-imposed restrictions: "I try not to drive long distances—6 or 7 h, for example"; "I survey the situation to make sure that parking will not be a problem." Despite driving concerns, most participants reported feeling very safe (88%) and very comfortable (94%) driving their cars.

TABLE 2 Personal Driving Restrictions

Driving Restriction	Percentage
Do not place any restrictions on my driving	24
Do not drive in bad weather	43
Do not drive after dark	41
Do not drive in the city	24
Do not drive in unfamiliar places	24
Do not drive in rush hour traffic	21
Do not drive during high glare times (dawn and dusk)	17
Do not make certain left-hand turns	16
Do not parallel park	13
Do not drive with young children in the car	12
Do not back up	6
Do not drive without a passenger to copilot	1

$N = 157$

Postvideo Results

The first indicator that learning about auto safety and comfort features is important to elders was that the number of older drivers who attended the presentations exceeded the stated recruitment goal of 100 elders, as noted in the original grant proposal. Most participants (77%) indicated the video was "very informative," and 21% reported the video was "somewhat informative," whereas only 2% reported learning "nothing I did not already know." Participants were asked to indicate what information they found most interesting. Almost a quarter of the respondents, 24%, noted the convex mirrors, followed by 20% who indicated the support handle and the visor extender. Thirteen percent responded that all the features were interesting to learn about. Moreover, discussions following the viewing and completion of survey instruments were typically lively, with participants raising important driving safety concerns. Viewing the video provided the opportunity to talk about these concerns.

As mentioned, 127 (81%) of the initial participants were surveyed by telephone approximately 2 months after the presentations. Participants were asked to indicate how familiar they were with 13 items depicted in the video both before viewing the video and later as part of the follow-up telephone survey. Familiarity was rated as 0 (not familiar), 1 (somewhat familiar), and 2 (very familiar). The hypothesis was that most participants would not be familiar with the features before viewing the video. Table 3 presents pre- and posttest data on familiarity with the auto features. Familiarity significantly increased for 10 of the 13 items. Three items (convex side- and rearview mirrors and the safety belt adjustor) did not show increased familiarity. However, participants' questions and comments suggested that the participants equated convex mirrors with standard side- and rearview mirrors and that there was some confusion regarding safety belt adjusters, safety belt extenders, and the retractable safety belts that come standard on most cars. Thus, while familiarity scores did not change for the convex mirrors and safety belt adjustor, it is likely that after the video, participants understood the distinction between at least convex and standard mirrors. Thus, data confirmed the first hypothesis

TABLE 3 Change in Familiarity with Features

Feature	Pretest Mean	SD	Posttest Mean	SD	*t*-Value	Signif.
Convex side-view mirror	1.02	.86	1.14	.67	1.28	
Convex rearview mirror	0.89	.88	0.94	.58	0.42	
Seat cushion	1.05	.89	1.55	.58	6.48	***
Trash bag/silk scarf	0.09	.39	1.16	.58	17.30	***
Safety belt adjustor	1.09	.89	1.13	.66	0.44	
Safety belt extender	0.86	.89	1.06	.62	1.94	*
Ribbon on safety belt	0.12	.45	0.94	.64	12.08	***
Pedal extenders	0.33	.58	1.16	.49	13.73	***
Key extenders	0.12	.44	0.85	.63	11.31	***
Ceiling hand grip	0.83	.84	1.59	.60	9.56	***
Support handle	0.56	.83	0.93	.62	4.35	***
Safety belt pad	0.54	.79	1.20	.68	7.77	***
Visor extender	0.67	.81	1.39	.65	7.85	***

$N = 127$
* <.05, *** <.001

that the video would result in increased knowledge about auto safety and comfort features.

The second and third hypotheses were that viewing the video would result in increased consideration and use of auto safety and comfort features. Since there was little initial knowledge of the features, there was not a true baseline measure of consideration. Before viewing the video, few participants reported using the auto features shown on the video, with the exception of the seat cushion and the ceiling hand grip: each of these two items was used by about a quarter of the participants. Thus, consideration and use were examined in terms of actions participants reported taking after seeing the video.

The first indicator of participants' consideration of vehicle features was measured as follows. Immediately after viewing the video, participants were asked about the likelihood that they would take three actions: call a toll-free number, seek information on a website, and try one or more of the features shown on the video. Many expected to call one of the phone numbers (13% very likely and 43% somewhat likely); some anticipated seeking information through a website (14% very likely and 28% somewhat likely). Most participants indicated they were very likely (50%) or somewhat likely (34%) to try a feature. Then, after watching the video and completing the data collection instruments, the participants received handouts that depicted and described the 13 features, listed resources for locating the features (toll-free numbers and websites), and itemized a series of fiv follow-up steps.

At the 2-month follow-up interview, participants were asked whether they had taken a number of steps associated with the vehicle features seen in the video. Of those interviewed at this interval ($N = 127$), 85% reported they had read the handouts, and 63% had discussed the features with family or friends. Both of these activities were likely to increase familiarity with the features and might also indicate consideration. Other activities were more clearly indicators that elder participants were considering the features. Twenty percent had already looked for features in stores or on the Internet, and 9% had tried one or more features, whereas only 2% had contacted a professional for advice or information. The three participants who had contacted a professional had contacted their eye doctors. On average, participants had taken 1.8 of the five steps indicating feature consideration (SD: 1.0)

Eleven percent of the telephone interviewees indicated they had purchased one or more features since viewing the video: visor extender, convex side view mirrors, seat cushion, and support handle. One person had tried the plastic trash bag as a seat cover, and a few had attached a ribbon to their safety belts. Participants were also asked about further steps they were considering or planning regarding the features. Approximately 44% mentioned one or more steps they planned to take or were in the process of doing, typically associated with one or more features. The most commonly mentioned features that participants were planning to investigate were visor extenders (15%), convex mirrors (12%), safety belt extenders (6%), and support handles (5%).

Some participants stated they were having difficulty locating some items. Others, inspired by the video, were looking for items that addressed other needs, or items they thought would better address their personal circumstances (driving challenges and type of automobile). One participant had tried several mirrors but found that they compromised visibility.

Participants were asked to provide additional comments on auto safety features or other aspects of safe and comfortable driving. Some made requests for additional information and expressed driving concerns that went beyond the scope of this research. Several suggestions were made for vehicle design improvements: the need for extra steps for vans and SUVs; headrests that did not impede one's view when checking over the right shoulder and backing up; places for storing a cane or other adaptive devices; nonslip floor covering; easily accessible horn button. In addition, participants mentioned the need for more information regarding electronic alerting devices, tinted glass, and the recommended distance between the air bag and the driver.

DISCUSSION OF RESULTS

A limitation of this study was that all the study participants resided in a single geographic area—eastern Massachusetts. It is not known how generalizable these findings may be to other, nonsuburban communities, nor is it known how these findings might relate to a sample with less formal education. This sample was comparable for self-restrictions imposed and driving histories to those described in the literature. From that perspective, the findings may be generalizable to other drivers age 70 and older who reside in suburban areas and who have similar driving histories.

Although it was beyond the scope of this study to assess the individual components and quality of the video itself, participants overall seemed to like the video, and several asked if copies could be made available for viewing at other sites. Although amateur in its production (with the noted exception of being filmed by a professional videographer), this video served the research purposes and is likely to have a useful life generating conversations about safe mobility among elders beyond the duration of the study. Still it is worthwhile noting that one woman commented that she would learn better "by having the features there to touch and feel and maybe even try on [her] car." This concept is in development through the CarFit initiative of the AAA Foundation for Traffic Safety and the American Society on Aging.

There was great interest among the participants in the features that were demonstrated in the video. Although not all participants followed through on obtaining the features by the time of the telephone interview about 2 months later, many did express the intention of looking into one or more of the features in the future. Increased recognition by professionals in aging, health care, and transportation of the broad range of vehicle modifications that could benefit the general aging population and not just those elders who are formally acknowledged as "disabled" might result in elders being able to continue driving safely and also having a more gradual path between driving and driving cessation.

CONCLUSIONS

The video served the intended purpose of increasing elders' awareness of vehicle modifications that could enhance safety and comfort for older drivers. Equally important, the video provided an opportunity for older drivers to talk about challenges and concerns they have regarding their driving skills and to receive resource materials and referral information to address these concerns. Reinforcing the importance of the opportunity to talk, 63% of the participants interviewed by telephone had talked to family or friends about the features demonstrated in the video. Although 92% had taken at least one follow-up step, only 11% had actually purchased any of the features. However, many more (44%) were considering or planning to further investigate one or more of the features. Some change was noted, but 2 months may not be sufficient time to observe change for the majority of the participants. Moreover, it is not known to what extent the participants may draw on this knowledge in the future, if and when they perceive a more direct need.

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Investigation of Time into Red for Red Light–Related Crashes

Karl Zimmerman and James A. Bonneson

This paper describes a research effort to determine the time into red of 63 red light–related crashes. From these 63 crashes, the relationship of time into red and the red light–related crash type is explored, as are relationships to other factors. From the red light–related crashes investigated, it was found that red light–related crash type was related to time into red. Other factors were not found to be related to time into red, although the sample size and selection method prevented any definitiv findings. A brief discussion of the possible safety effectiveness of the all-red interval is also included. The time into red of red light–related crashes is related to the time into red of red light violations, so a characterization of red light violations and crashes is provided, along with suggested red light violation countermeasure classifications for each type of red light violation.

Red light violations have been widely recognized as a serious intersection safety problem. This paper focuses on one aspect of red light–related crashes: the amount of time after the start of red, or time into red, when red light–related crashes occur. This time into red may indirectly provide important clues about the nature of red light–related crashes. The time into red may also be helpful for identifying the most effective category of red light violation countermeasure to use in a particular situation. Finally, time into red may also help identify the likely effectiveness of some red light violation countermeasures. The use of an all-red interval was selected as an example of such a countermeasure.

BACKGROUND

Time into Red of Red Light Violators

Several studies investigated the amount of time after the start of red when red light violators enter an intersection. In one of the more recent studies, Bonneson et al. examined 541 signal phases in which at least one vehicle entered the intersection after the start of red (*1*). The results of this examination are shown in Figure 1 as a function of time into red. The median entry time was less than 0.5 s, and approximately 80% of drivers entered the intersection within 1.0 s after the start of red. This result roughly agrees with Farraher et al., who found that about 85% of red light runners enter the intersection within 1.5 s after the start of red (*2*).

Milazzo et al. noted that there were two common types of red light–related crashes: right-angle and left-turn-opposed (*3*). Unlike right-angle crashes, left-turn-opposed crashes are likely to occur soon after the start of red and possibly before the end of the all-red interval. This situation is especially true when the left-turn movement is permitted to turn through gaps in the opposing traffic stream. Drivers waiting to turn left while in the intersection at the end of the phase may unintentionally turn in front of an opposing through vehicle, believing that its driver will stop for the red indication. If this through driver violates the red indication, he or she may collide with the left-turning driver. This situation will not occur when protected-only left-turn phasing is provided.

Figure 2 illustrates the time of entry of the opposing left-turn and crossing-through movements that conflict with the subject through movement. The figure illustrates the flow rate of these two traffic movements for probability of having a headway less than 2.5 s. It is assumed that a red light violator is not able to avoid conflict with a stream of opposing left-turn or crossing-through vehicles that have headways of 2.5 s or less.

Both conflicting traffic movements identified in Figure 2 tend to be in queue as the subject phase ends. The probability associated with the crossing-through movement increases more gradually than the left-turn movement and reflects the longer start-up reaction time of the crossing-through driver to the change in the signal indication. The probability for the through movement decreases after about 20 s of red, reflecting the transition from queue service to random arrivals during green for the conflicting through movement. The probabilities shown in Figure 1 suggest that permitted left-turn vehicles clear the intersection within the first 3.0 s of red. Given a 1.0-s all-red interval, the probabilities also suggest that crossing-through vehicles will not start to enter until after about 4.0 s have elapsed.

The trends shown in Figure 2 are intended to reflect the observations made by Bonneson et al. (*1*) and Milazzo et al. (*3*). The probabilities shown and their time duration are highly dependent on the signal settings and traffic volumes. The values shown in Figure 2 are not the only possible values. However, all trends will tend to be similar to those shown in Figure 2.

Figure 3 illustrates how the time of violation and the time of entry combine to create the potential for a red light–related conflict. It represents the combination of Figures 1 and 2 for joint probability of a red light violation during a specific time interval and the probability of a conflicting vehicle entering the intersection during the same time interval. The trends in Figure 3 indicate that the potential for conflic is high in the first second of red. This conflict would be between a through vehicle violating the red and an opposing left-turn vehicle attempting to clear the intersection. The trend drops rapidly for the second and third seconds of red, reflecting the decreasing probability of violation and of left-turn presence. After the fourth second, the probability of conflict increases in a manner consistent with the probability of a through vehicle having a headway less than 2.5 s (as shown

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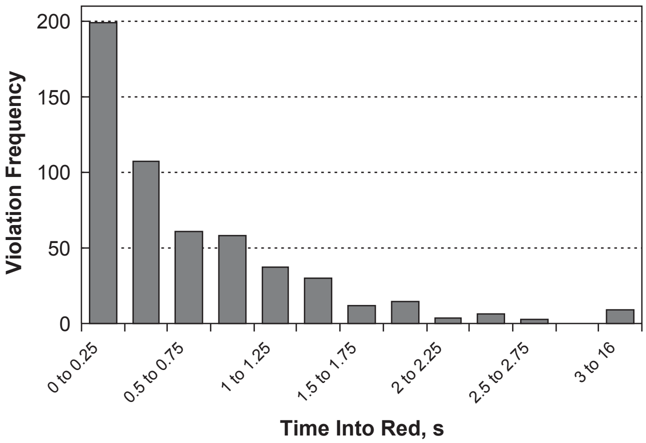

FIGURE 1 Frequency of red light violations as function of time into red.

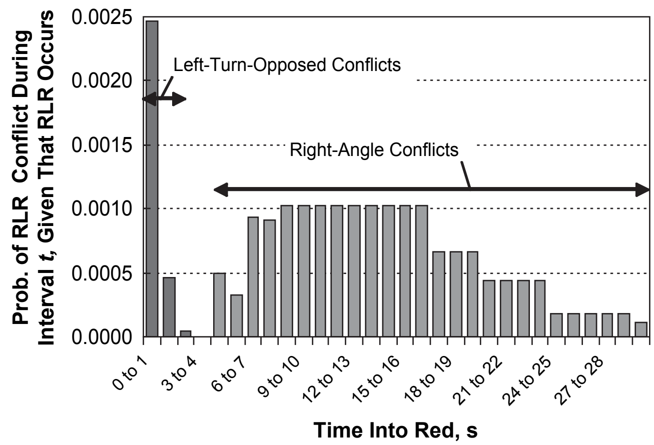

FIGURE 3 Probability of red light–related conflict as function of time into red.

in Figure 2). These trends suggest that the type of crash associated with red light violations is likely to be highly correlated with the time of the crash. If this assertion can be confirmed by using field data, crash type could form a basis for countermeasure selection.

Characterization of Red Light Violations

Bonneson et al. (*1*) and Milazzo et al. (*3*) provided characterizations of red light violations. Milazzo et al. developed a relationship among the legality of entering an intersection, the safety of that intersection entry, and whether the driver intentionally or unintentionally entered the intersection as a means for comparing red light violation events (*3*). Table 1 illustrates this relationship.

Milazzo et al. stated that Type B entries were assumed to be rare events and that camera enforcement may eliminate them entirely (*3*). Also, Type B entries were considered safe because conflicting traffic movements would have been unable to move by this time. Type C entries would be principally left-turn-opposed crashes, because cross-street traffic would not have time to enter the intersection. Type D entries would be primarily right-angle crashes. This characterization suggests that intentional red light violations occur only during the first 3 s of red.

Bonneson et al. offered characterizations of the red light violator for the purpose of identifying whether engineering or enforcement countermeasures were most appropriate for the particular red light violations that occur at a particular site (*1*). These characterizations are shown in Table 2.

The information in Table 2 suggests that two basic types of decisions are associated with a red light violation event. An avoidable red light violation event is committed by a driver who believes it is possible to safely stop but who decides to run the red indication anyway. In contrast, an unavoidable event is committed by a driver who either believes he or she is unable to safely stop and consciously decides to run the red indication or is unaware of the need to stop. Therefore, an avoidable red light violation event consists only of drivers who intentionally run the red, and an unavoidable red light violation event consists of drivers who could have either intentionally or unintentionally run the red indication.

The last column in Table 2 compares the characterizations used by Bonneson et al. (*1*) to those used by Milazzo et al. (*3*). The only common element between the two characterizations is whether the violation was intentional. The avoidable driver decision type is an intentional violation, which is defined by Milazzo et al. as either a Type B or a Type C entry (*3*). On the other hand, the unavoidable driver decision type is associated with any nonlegal entry type (i.e., entry types B, C, and D).

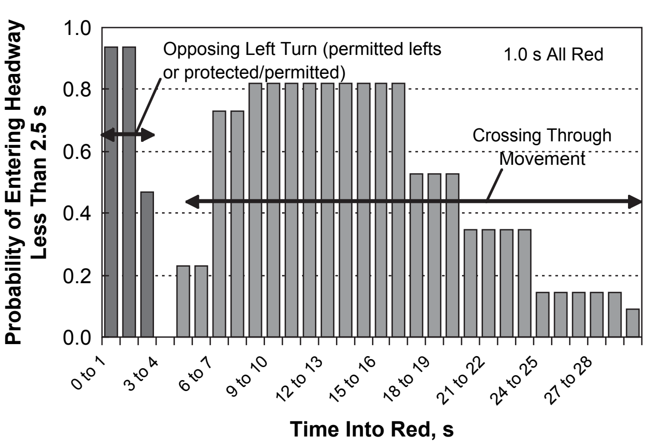

FIGURE 2 Probability of entering intersection as function of time into red.

Time into Red of Red Light–Related Crashes

Milazzo et al. investigated the relationship between time into red and crash type (*3*). To perform their investigation, they obtained 34 photographs of red light–related crashes taken by enforcement cameras. All photographs were obtained from Internet websites hosted by enforcement agencies. The crashes in the photographs were then classified by crash type. The right-angle crashes were further classified by their time into red. The findings of Milazzo et al. are given in Table 3.

The data in Table 3 are consistent with the trends shown in Figure 3 in three ways. First, right-angle crashes do not appear likely in the

TABLE 1 Relationship of Entry Type to Time into Red (3)

Entry Legal?[1]	Entry[2] Safe?	Entry Intentional?[3]	Typical Time into Red[4] No Earlier Than . . .	No Later Than . . .	Entry Type
Yes	Yes	Yes	Not applicable	Not applicable	A
No	Yes	Yes	0 s	1 to 3 s	B
	No	Yes	1 to 3 s	2 to 3 s	C
		No	> 3 s	> 3 s	D

[1]A legal entry occurs on green or yellow; by definition, Entry Type A is not a red light runner.
[2]A safe entry implies little or no chance of a crash with either a crossing vehicle or an opposing left turn vehicle.
[3]An intentional entry means that a driver realizes the signal is red but proceeds anyway. An unintentional entry means the driver is unaware of the signal and, therefore, the need to stop.
[4]The actual boundary times into red for each entry type depend on local conditions and signal timings.

TABLE 2 Relationship Between Countermeasure Category and Driver Decision Type

Driver Decision Type	Possible Scenario	Countermeasure Category: Engineering	Enforcement	Per Milazzo et al. (*3*)
Avoidable	Congested, cycle overflo	Less effective	Most effective	B, C
Unavoidable	Incapable of stop, inattentive driver	Most effective	Less effective	B, C, D

first 3 or 4 s of red. This implies that most cross-street vehicles are stopped at the start of green. Second, the median time into red of 1.9 s for left-turn-opposed crashes suggests that most of these crashes occur as a result of left-turn activity at the end of the phase. Third, the majority of red light–related crashes are of the right-angle type, agreeing with the findings of Bonneson et al. (*4*). In fact, Bonneson et al. found that there were five to six times as many right-angle crashes as left-turn-opposed crashes in their evaluation of 502 red light–related crashes in three Texas cities.

The reported average time into red of 6.0 s for the left-turn-opposed crashes compared to the median time of 1.9 s suggests that one left-turn-opposed crash occurred well into red. It is likely that this crash occurred because of driver inattention rather than a misjudged gap at the end of the permitted left-turn movement. Therefore, this one crash is probably unusual for a left-turn opposed crash, and the median time into red shown in Table 3 is probably more accurate for this crash type.

Potential Safety Benefit of All-Red Intervals

The time into red of red light–related crashes provides a clue to the potential benefit of the all-red interval. Several studies investigated the effect of the all-red interval on crashes (*5–9*). However, the finding reported in these studies are inconsistent. Research cited in an FHWA synthesis showed overall reductions in right-angle crashes ranging from 10% to 40% after an all-red interval was added. Zador et al. found that crash experience increased as the clearance interval decreased (*6*), although the clearance interval included both yellow and all-red intervals. Datta et al. found a significant crash reduction after all-red intervals were applied to traffic signals in Michigan (*7*). All three of these results argue that all-red intervals are effective at reducing intersection crashes.

Other research, such as that conducted by Polanis (*8*) and Roper et al. (*9*), found little or no safety benefit from the addition of all-red intervals. Most recently, a study by Souleyrette et al. for the Minnesota Department of Transportation concluded that all-red intervals were not effective at reducing crashes over time (*10*). Souleyrette et al. did find a short-term decrease in crashes after the all-red intervals were added, but this decrease lasted approximately 1 year. After 1 year, intersection crashes were at the same level they had been before an all-red interval was added, and after 5 years crashes had increased. Also, addition of the all-red interval reduced intersection capacity and increased delay, both of which can lead to a higher number of avoidable red light violations, as shown in Table 2.

TABLE 3 Red Light–Related Crash Summary Statistics (3)

Crash Type	Range of Time into Red (s)	Number of Crashes	Average Time into Red (s)	Median Time into Red (s)
Right-angle	0.0 to 2.9	0	No crashes	No crashes
	3.0 to 21.8	27	8.7	6.7
Left-turn-opposed	1.0 to 26.9	7	6.0	1.9
Overall		34	8.1	6.4

From this discussion, it is clear that the safety benefits of an all-red interval are uncertain. The time into red of red light–related crashes is critically important to determining the safety benefits of an all-red interval. Therefore, this paper addresses the potential safety benefit of an all-red interval on the basis of the time into red of red light–related crashes.

TIME INTO RED FOR RED LIGHT–RELATED CRASHES

The specific objectives of this research were to

- Identify when crashes occur after the start of red,
- Determine the effect of time into red on crash type and severity,
- Find factors that influence time into red, and
- Show how this information can be used to select countermeasures to prevent red light–related crashes.

Site Selection and Database Attributes

Initially, 100 photographs of crashes caused by red light violations, collected from three to five different intersections in two cities, was determined to be necessary to ensure statistical stability in any trends uncovered during this research. The data collected for each crash are shown in Table 4. A solid checkmark in Table 4 means that these data were always available, and a hollow checkmark indicates that these data were available for some photographs but not for others.

Nine agencies known to have red light enforcement cameras in operation were contacted and were asked for their participation. The agencies with the largest number of operating cameras were contacted first. Of these nine agencies, two in Arizona were selected.

An initial review of the crash records from Arizona indicated that three to five intersections would be insufficient to obtain the desired sample size. There were four main factors responsible for this:

- The presence of the cameras had significantly reduced red light–related crashes at those locations.
- The camera was not always present when a crash occurred.
- The camera did not always capture the actual crash even if it were present.
- Many of the cameras had been in service for less than 18 months, limiting the number of crashes that could be observed.

Because of these limitations, two approaches were used to increase the sample size as much as possible. First, all intersections in those cities that used red light camera enforcement were considered, increasing the number of intersections to 12. Second, crash photographs were obtained from other sources. These sources are given in Table 4 and are discussed more fully in the following sections. A total of 63 photographs were eventually obtained from all sources. Although this total was short of the 100-photograph sample size, it was the largest collection of photographs of red light–related crashes that had been assembled to date.

Data Sources

Arizona

Records of 27 red light–related crashes at 12 intersections were obtained from the two cities in Arizona. The distribution of these

TABLE 4 Database Attributes

Data Type	Attribute	Data Availability by Resource: Arizona[1]	Maryland	Milazzo et al. (*3*)	Other
Crash	Time into red of crash	✓	✓	✓	✓
	Speed of red-light-violating vehicle	✓	✓	✓	✓
	Date and time of crash	✓	✓	✓	✓
	Crash type (right-angle or left-turn-opposing)	✓	✓	✓	✓
	Severity	✓	—	—	—
	Number of injuries	✓ (hollow)[2]	—	—	—
Traffic control	Left turn phasing	✓	✓ (hollow)[3]	✓ (hollow)[3]	✓ (hollow)[3]
	Yellow interval duration	✓	✓	—	—
	All-red interval duration	✓	—	—	—
	Approach speed limit	✓	✓	—	—
Volume	Annual average daily traffic	✓	—	—	—
Geometry	Number of left, through, and right turn lanes on subject street	✓	✓ (hollow)[4]	✓ (hollow)[4]	✓ (hollow)[4]
	Number of left, through, and right turn lanes on cross street	✓	✓ (hollow)[3]	✓ (hollow)[3]	✓ (hollow)[3]
Total crashes: 63		27	18	7	11
Data source:		Crash report, agency files field survey	Photo	Photo	Photo

[1]Data from both cities in Arizona were in an identical format and were considered to be from a single source.
[2]Number of injuries not always known or reported.
[3]Left turn signal heads not always visible in camera field of view.
[4]All lanes not always visible in camera field of view.

TABLE 5 Distribution of Red Light–Related Crashes by Source and Type

Source[1]	Intersection Approaches	Number of Crashes: Left-Turn-Opposed	Right-Angle	Total
Arizona	12	16	11	27
Maryland	11	3	15	18
Milazzo et al. (*3*)	3	1	4	5
Other	9	2	11	13
Total:	35	22	41	63

[1]Data sources: Milazzo et al. (*3*); Other: Internet, magazine, and law enforcement personnel

crashes is given in Table 5. For legal reasons, no photographs of the crashes could be obtained from either city. Instead, city personnel reviewed the photographs and associated crash reports and documented their findings. The findings were then made available to the research team.

With a few exceptions, all the crash data identified in the third column of Table 4 were obtained with this approach. Severity information for each crash was limited to simply an indication of whether one or more persons involved in the crash were injured or killed. Levels of injury extent were not provided.

Maryland

The participating agency in Maryland provided 18 photographs of red light–related crashes at 11 intersections in central Maryland. The distribution of these crashes is shown in Table 5.

All the photographs indicated the time into red, vehicle speed, date and time of the crash, crash type, left-turn phasing, yellow duration, and approach speed limit. No information was available about crash severity, all-red duration, or traffic volume. Limited information about the approach geometry was available in some photographs. Ten of the 11 intersections were subsequently identified by using Internet-based street maps. From these identifications, aerial photographs were obtained and used to provide some of the missing geometric information about each intersection approach.

Milazzo et al.

The report by Milazzo et al. contained five photographs of red light–related crashes (*3*). These photographs were from the following sources:

- Two crashes in Charlotte, North Carolina (one intersection);
- One crash in Oxnard, California; and
- Two crashes in Washington, D.C. (one intersection).

The distribution of these crashes is given in Table 5.

The photographs in Milazzo et al. tended to contain less information than did the Maryland photographs. Specifically, they provided only data for time into red, vehicle speed, date and time, crash type, and left-turn phasing. No information was available about crash severity, yellow duration, all-red duration, approach speed limit, or traffic volume. Geometric information for each approach was obtained from a combination of the crash photograph and aerial photographs obtained from the Internet.

Other Crash Photographs

Thirteen other photographs of red light–related crashes were obtained from various sources, representing nine intersections. The locations of eight intersections were identified by using the photograph and the Internet. Three crash photographs were obtained from various red light–related Internet websites. Three other photographs were obtained from a prominent magazine. The remaining photographs were obtained from individuals affiliated with various law enforcement agencies. These 13 photographs include locations in Washington, D.C., North Carolina, and Australia. The distribution of these crashes is given in Table 5.

The data available in these other photographs varied. Most of them contained at least as much information as the Milazzo photographs. Some contained the same level of information as the Maryland photographs.

DATA SUMMARY AND ANALYSIS

This section describes the database of red light–related crashes collected and the analysis performed. Because of the sparse amount of data available for most of the crashes, the analysis opportunities were limited.

Database Summary

Selected database attributes are summarized in Table 6. Collectively, the statistics show that the data reflect a wide range of typical traffic control and volume conditions. They also show that with one exception, there is no practical difference between the conditions present in left-turn-opposed and right-angle crash photographs. The percent of injury crashes, the speed of the red light violator, the speed limit, and the average daily traffic volume are essentially the same for each crash type.

The one exception is the time into red of the crash. The median time into red for left-turn-opposed crashes is only 0.9 s, whereas the time

TABLE 6 Database Summary

Crash Type	Attribute	Statistic: Obs.	Average	Std. Dev.	Median	Minimum	Maximum
Left-turn-opposed	Time into red, s	22	0.9	0.6	0.9	0.1	3.1
	Yellow duration, s	19	4.0	0.2	4.0	3.5	4.5
	Percent injury crashes[1]	16	56	—	—	—	—
	Speed of violator, mph	21	32.8	10.1	32	16	52
	Speed limit, mph	19	40.8	3.8	40	35	45
	AADT, veh/d[1]	16	18,100	4,600	18,400	9,800	32,100
Right-angle	Time into red, s	41	14.1	12.0	8.9	0.6	44.2
	Yellow duration, s	33	4.3	0.4	4.0	3.9	5.0
	Percent injury crashes[1]	11	55	—	—	—	—
	Speed of violator, mph	40	33.5	7.8	32	17	55
	Speed limit, mph	26	40.4	6.0	40	35	55
	AADT, veh/d[1]	11	17,800	2,500	18,900	14,800	23,300

[1]Data from Arizona only

into red for the right-angle crashes is 8.9 s. These times are consistent with those reported by Milazzo et al. (*3*), as shown in Table 2. It could be argued that the similarity between the two results resulted because the two databases shared five common photographs. To test this argument, the five common photographs were deleted. The median time into red for both crash types remained the same after the deletion. Therefore, the findings of Milazzo et al. were independently verifie with these data.

The number of crashes for each crash type is given in the third column of Table 6 for the time into red attribute. These statistics indicate that right-angle crashes exceed left-turn-opposed crashes by a factor of two (= 41/22). This factor is much smaller than the range of "five to six" reported by Bonneson et al. in their analysis (*3*). However, of the 27 crash records obtained from Arizona, almost two-thirds (16) were left-turn-opposed. For the remaining 36 crashes from other sources, the right-angle crashes exceed left-turn-opposed crashes by a factor of 5 (= 30/6), more closely agreeing with the earlier research results. This finding does not suggest that there is any bias in the statistics shown in Table 6 or that the earlier studies incorrectly estimated the ratio of right-angle to left-turn-opposed to crashes. Instead, this finding suggests that the overrepresentation of left-turn-opposed crashes is probably caused by local conditions in Arizona.

Data Analysis

The relationship between time into red and each of the database attributes shown in Table 4 was investigated by using graphical techniques. As Table 6 indicates, the percent injury crashes, violation speed, speed limit, annual average daily traffic, and yellow duration were effectively the same for both crash types. Although a relationship may exist between time into red and one or more of these other factors, this database did not reveal them.

Figure 4 shows the frequency of crashes as a function of time into red. The trends in this figure confirm the tendency for left-turn-opposed crashes to occur in the first few seconds of red. With one exception, all right-angle crashes occurred after 5.0 s or more of red. Closer inspection of the one right-angle crash occurring earlier into red revealed that both colliding vehicles had violated their respective red indications nearly simultaneously.

Figure 4 also shows that the frequency of red light–related crashes tends to be highest in the first 5.0 s of red. After the first 5.0 s of red, the crash frequency declines, reaching a small and effectively random frequency after about 15 to 20 s of red. This pattern agrees with the expected trends shown in Figure 3. The high initial frequency of right-angle crashes is caused by discharging of the cross-street movement's queue. Red light violations during the cross street's queue clearance time have a high likelihood of conflicting with crossing vehicles. The potential for conflict after queue discharge is effectively constant because of the random nature of cross-street arrivals and red light violations as the time into red continues. The lack of crashes between 15 and 20 s into red is probably an artifact of the database rather than a significant occurrence.

The trends shown in Figure 4 are similar to the hypothesized trends shown in Figure 3. Figure 4 confirms the hypothesized effect of the joint probabilities of violation and conflicting vehicle presence on crash occurrence. From these trends, it is logical to assume that enforcement efforts (which primarily reduce violations in the first few seconds of red) are most likely to primarily reduce the frequency of left-turn-opposed crashes. In contrast, engineering countermeasures intended to improve driver attention or visibility are most

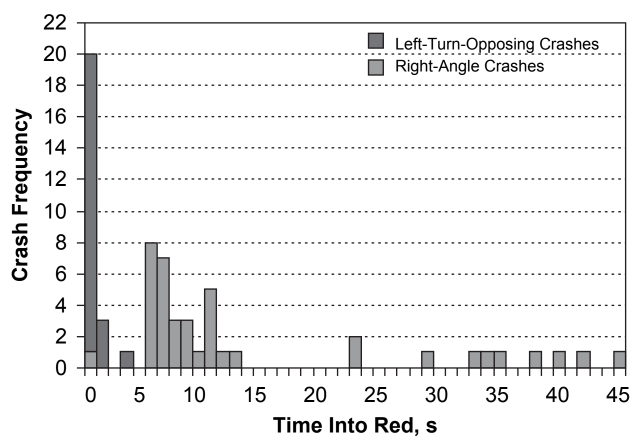

FIGURE 4 Crash frequency by time into red.

likely to reduce violations throughout the red and therefore reduce the frequency of right-angle crashes.

Possible Effect of All-Red Intervals on Intersection Safety

Figures 1 and 4 give an indication about the effectiveness of an all-red interval at improving intersection safety. Figure 1 shows that red light violations decrease to a minimal level after about 2 s of red. The observed crash distribution in Figure 4 indicates that the crashes that occur within the first few seconds of red are typically left-turn-opposed crashes. These crashes are triggered by the onset of red, rather than the length of the all-red interval, because left-turning drivers attempting to clear the intersection after the start of red are expecting opposing through vehicles to stop for them.

Figure 4 also shows that the frequency of right-angle crashes increases as the cross street's queue discharges, then decreases after queue discharge is complete on the cross street. The increase in crash frequency occurs at the same time that the red light violation frequency has decreased to a minimal level, and after any all-red interval would have expired. Although the number of right-angle crashes in the first few seconds of red probably would be reduced, Figures 3 and 4 indicate that these crashes are rare, and so preventing a rare crash type would not produce a significant safety benefit. Therefore, the observed trends in Figures 1, 3, and 4 suggest that the presence of an all-red interval would have a minimal effect on red light–related crashes of either type (left-turn-opposed or right-angle). This observation agrees with the results reported by Polanis (*8*), Roper et al. (*9*), and Souleyrette et al. (*10*) and is at odds with the other results presented here.

This research was not designed specifically to investigate the effects of an all-red interval on intersection safety. Therefore, it is possible that an all-red interval produces a safety benefit under certain specifi conditions. Further research is necessary to determine the conditions under which all-red intervals are beneficial to intersection safety.

Characterization of a Red Light Violation

From the results of this research, Table 2 can be expanded to include some additional information and the time into red of a red light–related crash. This expanded table is shown here as Table 7. The typical time into red of violations was added on the basis of the separation in time of left-turn-opposing and right-angle crashes found in this research.

The characterizations of red light violations shown in Table 7 provide a reasonably complete picture of red light violations and causes as well as suggest possible courses of action if red light violation countermeasures are desired. Engineering countermeasures may include increasing the yellow interval duration or providing back plates to increase signal conspicuity. Engineering countermeasures are best used in cases in which violations are characterized as unavoidable. Enforcement countermeasures may include active police presence at an intersection or red light enforcement cameras. Enforcement countermeasures are best used in cases in which violations are characterized as avoidable. Table 7 may be used by an agency to determine the most effective actions to take to reduce red light violations at a specific location or as part of a citywide or areawide program.

CONCLUSIONS

Sixty-three red light–related crashes were investigated to attempt to find a relationship between time into red of red light–related crashes, crash type, crash severity, and other intersection factors.

Crash type was related to time into red of a crash. Left-turn-opposing crashes occurred within the first 5 s of red, and right-angle crashes generally occurred after 5 s of red. There was an increase in right-angle crashes when the cross-street queue discharged, and then the right-angle crash frequency became effectively random as time into red continued to increase.

The findings of this research were used to illustrate the likely effectiveness of one red light violation countermeasure, the all-red interval. Increasing the all-red interval is likely to reduce the portion of right-angle crashes that occur in the first few seconds of red. However, right-angle crashes are relatively infrequent in the first few seconds of red, so increasing the all-red interval may not significantly reduce the total number of right-angle crashes.

By using the time into red of red light–related crashes, a characterization of red light violations can be created that includes crash type for violation causes. From this characterization, the most appropriate countermeasure category can be selected. Engineering countermeasures are best suited to situations in which violations occur because

TABLE 7 Characterization of Red Light Violation

Cause Category	Violation Type	Driver Intent[1]	Time of Violation	Most Likely Crash[2]	Countermeasure Category
Unnecessary delay[3]	Avoidable	Intentional	Any time during red	Right-angle	Enforcement (unless engineering can be used to reduce delay or eliminate signal)
Congestion, dense traffic			First few seconds of red	Left-turn-opposed	
Incapable of stop	Unavoidable				Engineering (to increase probability of stopping)
Inattentive		Unintentional	Any time during red	Right-angle	Engineering (to improve signal visibility or conspicuity)

[1]Intentional: driver sees red indication and decides to proceed anyway; unintentional: driver does not see red indication.
[2]Based on this research, "unnecessary delay" or "inattentive" violation causes may result in either left-turn-opposed or right-angle crashes because the cause does not depend on how long the red indication has been on.
[3]"Unnecessary delay" is from the driver's viewpoint.

the driver is unable to stop or is unaware of the need to stop. Enforcement countermeasures are best suited to locations where violations occur because of a driver's desire to avoid delay or where the intersection is congested.

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Multijurisdictional Safety Evaluation of Red Light Cameras

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The use of red light camera (RLC) systems has risen dramatically in the United States in recent years. The size of the problem, the promise shown by RLC systems in other countries, and the paucity of definitive U.S. studies have motivated a multijurisdictional U.S. study. The fundamental objective of this study, which was sponsored by FHWA, was to determine the effectiveness of the RLC systems in reducing crashes at monitored intersections as well as jurisdictionwide. Phase I involved the development of a detailed experimental design that included collection of background information, establishment of study goals, selection of potential study jurisdictions, and specification of statistical methodology. In Phase 2, an empirical Bayes before-and-after study used data from seven jurisdictions across the United States, with a total of 132 treatment sites. Effects detected were consistent in direction with those found in many previous studies—a decrease in right-angle crashes and an increase in rear-end crashes—although both effects are somewhat lower than those reported in many sources. The extent to which the increase in rear-end crashes negates the benefits for right-angle crashes is unclear and points to the need for an examination of the economic cost of crashes, which is the subject of a companion paper, to aggregate the effects on rear-end, right-angle, and other crash costs. That second paper seeks to isolate all factors that would favor the installation of RLC systems by using the aggregate economic benefit as the outcome variable. There were weak indications of a spillover effect, which point to a need for a more definitive, perhaps prospective, study of this issue.

After extensive use of red light camera (RLC) systems in other countries for more than a decade, their use has risen dramatically in the United States in recent years. This treatment is aimed at reducing a major safety problem at urban and rural intersections that is estimated to produce more than 100,000 crashes and approximately 1,000 deaths per year in the United States (*1*). The size of the problem, the promise shown from the use of RLC systems in other countries, and the paucity of definitive U.S. studies together established the need for a multijurisdictional U.S. study to determine the effectiveness of the RLC systems in reducing crashes at monitored intersections as well as jurisdictionwide.

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To meet this need, FHWA sponsored the study on which this paper is based. The work was conducted in two phases. In Phase I, a detailed experimental design was developed for a multijurisdictional study of red light camera programs. This included collecting background information from literature and other sources, establishing study goals, interviewing and choosing potential study jurisdictions; and designing a study. In Phase II, the design was implemented. This paper reports on the study design, data collection, and analysis that led to the results presented for the effects on target crashes.

A further study to aggregate the effects on the main impact types by using the economic cost of crashes examines the influence on RLC effectiveness of factors such as level of publicity and signalization variables and is described in a companion paper (*2*). A detailed description of all project efforts can be found in the FHWA final project report (*3*).

LESSONS LEARNED FROM REVIEW OF CRITICAL STUDIES

A literature review was made of 17 international studies judged to be critical to learn lessons that would guide the design of the new study. The studies reviewed varied widely in several areas, including impact type and severity, the entities studied (treated intersections, treated approaches, jurisdictionwide), the use and designation of comparison sites, the treatment type (cameras only, cameras plus warning signs, red light running and speed cameras), sample sizes, and study methodology (simple before-and-after, before-and-after with comparison group, chi-squared tests, statistical modeling, etc.).

The review found that estimates of the safety effect of red light running programs vary considerably. A summary of the more relevant study findings is provided in Table 1 (*4–18*), including a synopsis of the main difficulties. From this table, one might conclude that the results support a conclusion that red light cameras reduce right-angle crashes and could increase rear-end crashes. However, as can be seen from the last column, most studies are tainted by methodological difficulties that would render useless any conclusions from them. One difficulty—failure to account for regression to the mean—can exaggerate the positive effects, whereas another—ignoring possible spillover effects to intersections without RLCs—will lead to an underestimation of RLC benefits, more so if sites with these effects are used as a comparison group. (Spillover effect is the expected effect of RLCs on intersections other than the ones actually treated, caused by jurisdictionwide publicity and the general public's lack of knowledge of where RLCs are installed.)

A similar assessment of the literature was made independently in a recent meta-analysis (*19*). That work found, as expected, that the

TABLE 1 Summary of Literature Review

Reference	City	Camera Sites	Comparison/ Reference Group	Crash Type Studied and Estimated Effects (negative indicates reduction)		Comment
Hillier et al. (*4*)	Sydney, Australia	Installed at 16 intersections	16 signalized intersections	Right-angle and left-turn-opposed Rear-end	−50% +25–60%	RTM* possible; spillover may have affected comparison sites; results confounded by adjustment to signal timing in middle of study period.
South et al. (*5*)	Melbourne, Australia	Installed at 46 intersections	50 signalized intersections	No significant results. Looked at right-angle, right-angle (turn), right against through, rear-end, rear-end (turn), other, all crashes, no. of casualties No significant results		RTM* possible; no accounting for changes in traffic volumes; comparison sites possibly affected by spillover and other treatments.
Andreassen (*6*)	Victoria, Australia			No significant results		Lack of an effect may be due to the fact that the sites studied tended to have few red running–related accidents to begin with (author). Comparison sites may have been affected by spillover.
Kent et al. (*7*)	Melbourne, Australia	3 intersection approaches at different intersections	Noncamera approaches	No significant relationship between the frequency of crashes at RLC and non-RLC sites and differences in red light running behavior		Cross-sectional design is problematic and there were likely spillover effects to the noncamera approaches at the same intersections.
Mann et al. (*8*)	Adelaide, Australia	Installed at 13 intersections	14 signalized intersections	Reductions at the camera sites were not statistically different from the reductions at the comparison sites.		RTM* and spillover to comparison sites are issues not dealt with.
London Accident Analysis Unit (*9*)	London, U.K.	RLC at 12 intersections and 21 speed cameras	Citywide effects looked at	No significant results		The results are confounded by the fact that two programs are being evaluated.
Hooke et al. (*10*)	Various cities in England and Wales	Installed at 78 intersections		All injuries	−18%	A simple before-after comparison not controlling for effects of other factors, regression to the mean and traffic volume changes; therefore there is limited confidence in the results.
Ng et al. (*11*)	Singapore	Installed at 42 intersections	42 signalized intersections	All Right-angle	−7% −8%	RTM* and spillover effects at comparison sites are issues.

(continued)

TABLE 1 *(continued)* **Summary of Literature Review**

Reference	City	Camera Sites	Comparison/ Reference Group	Crash Type Studied and Estimated Effects (negative indicates reduction)		Comment
Retting and Kyrychenko (*12*)	Oxnard, California	Installed at 11 intersections	Unsignalized intersections in Oxnard and signalized intersections in 3 similarly sized cities	All All injuries Right-angle Right-angle injury Rear-end	−7% −29% −32% −69% +3% (non-significant)	Looked at citywide effects, not just at RLC sites. 29 months of before and after data used.
SafeLight Charlotte (*13*)	Charlotte, North Carolina	Installed at 17 intersections	No comparison group	Angle—all approaches Angle—camera approaches All—camera approaches Rear-end—camera approaches All	−37% −60% −19% +4% ≤ 1%	Probable RTM* in site selection.
Maryland House of Delegates (*14*)	Howard County, Maryland	Installed at 25 intersections		Rear-end Right-angle Other	−32% −42% −22%	Probable RTM* in site selection.
Fleck and Smith (*15*)	San Francisco, California	Installed at 6 intersections	Citywide effects looked at	Citywide injury collisions caused by red-light violators. It is not clear how these were defined.	−9%	Question on definition of RLC crashes. Did not examine specific effects at treated sites.
Vinzant and Tatro (*16*)	Mesa, Arizona	6 intersections with RLC only, 6 with RLC plus photo speed enforcement	6 signalized intersections	Total crash rates—crashes per million entering vehicles at each intersection Combined-treatment quadrant Photo-radar quadrant RLC quadrant Control quadrant	 −15.9% −7.5% −9.7% −10.7%	It is not clear whether the assignment of treatment/no treatment to the four quadrants was random.
Halcrow Fox (*17*)	Glasgow, Scotland	Installed at 8 intersections and 3 pelican crossings	Areawide effects on injury crashes looked at	Crossing carelessly Unsafe right turn Failure to keep distance Other All per month	−54% −29% +8% −29% −32%	RTM* effects likely. Because the decreases in non-RLR crashes are greater than the RLR decreases at times, it is difficult to say what citywide effect the cameras have.
Winn (*18*)	Glasgow, Scotland	6 locations on 1 approach	Various	Injury crashes related to RLR violations	−62%	Probable RTM* effects.

*RTM—regression to the mean, also called bias by selection.

largest safety benefits were reported by studies that did not control for regression to the mean and that small effects tend to be found where the possibility of spillover was ignored. The one study cited that measured both spillover and specific effects, while ensuring that regression to the mean was not a factor, was an evaluation of the Oxnard, California, program (*12*). That study found a significant reduction in injury crashes at signalized intersections in the jurisdiction as a whole but did not examine the specific effects at the treatment sites. (A follow-up study is doing this.)

Although it is difficult to make definitive conclusions from studies that generally fail the tests on the validity of the methodology, the results of the review did provide some level of comfort for a decision to conduct a definitive, large-scale study of U.S. installations. It was important, however, that the new study capitalize on lessons learned from the strengths and weaknesses of the previous evaluations, many conducted in an era when the knowledge of potential pitfalls in evaluation studies and methods of avoiding or correcting them was not widespread.

The lessons learned required that the number of treatment sites be sufficient to ensure statistical significance of results and that the possibility of spillover effects be considered in designating comparison sites, perhaps requiring a study design without a strong reliance on the use of comparison sites. Previous research experience also pointed to a need for the definition of "red light running crashes" to be consistent, clear, and logical and for a mechanism to be provided for aggregating the differential impact on crashes of various impact types and severities.

Other considerations important in designing and planning the study were the need for use of the proper methods for accounting for regression to the mean, exposure, and other changes between before and after periods. These other changes typically include differences in crash reporting practice and other improvements (e.g., yellow interval improvements) made at time of RLC installation.

The lessons learned and other rational considerations were then incorporated into the experimental design. This paper reports on the core study question: "What impact do RLC programs have on intersection safety, as measured by changes in crash frequency and crash cost?" This paper concentrates on crash frequencies; a companion paper examines crash costs.

STUDY METHODOLOGY

Overview of General Evaluation Methodology

The general analysis methodology used is different from those used in previous RLC studies, benefiting from significant advances made in the methodology for observational before-and-after studies, which culminated in a landmark book by Hauer (*20*) that documented the empirical Bayes (EB) procedure used in the study. This approach sought to overcome the limitations of previous evaluations of red light cameras, specifically by

- Properly accounting for regression to the mean,
- Overcoming the difficulties of using crash rates in normalizing for volume differences between the before and after periods,
- Reducing the level of uncertainty in the estimates of safety effect,
- Properly accounting for differences in crash experience and reporting practice in amalgamating data and results from diverse jurisdictions,
- Avoiding the difficulties of conventional treatment–comparison experimental designs caused by possible spillover or migration effects to natural comparison groups, and
- Providing a foundation for developing guidelines for estimating the likely safety consequences of contemplated RLC installation.

In the EB approach, the change in safety for a given crash type at an RLC intersection is given by

$$B - A \tag{1}$$

where B is the expected number of crashes that would have occurred in the after period without the cameras and A is the number of reported crashes in the after period.

In estimating B, the effects of regression to the mean and changes in traffic volume were explicitly accounted for using safety performance functions (SPFs) relating crashes of different types and severities to traffic flow and other relevant factors for each jurisdiction based on locations without RLCs. Annual SPF multipliers were calibrated to account for the temporal effects on safety of variation in weather, demography, crash reporting, and so on. Because of the possibility of spillover effects to neighboring signalized intersections, it was decided to estimate annual multipliers for the period after the first RLC installation from the trend in annual multipliers of SPFs calibrated for a comparison group consisting of unsignalized intersections in the jurisdiction.

In the EB procedure, the SPF is used first to estimate the number of crashes that would be expected in each year of the before period at locations with traffic volumes and other characteristics similar to the one being analyzed. The sum of these annual SPF estimates (P) is then combined with the count of crashes (x) in the before period at a treatment site to obtain an estimate of the expected number of crashes (m) before RLC installation. This estimate of m is

$$m = w_1(x) + w_2(P) \tag{2}$$

where the weights w_1 and w_2 are estimated from the mean and variance of the regression estimate as

$$w_1 = P/(P + 1/k) \tag{3}$$

$$w_2 = 1/k(P + 1/k) \tag{4}$$

where k is a constant for a given model and is estimated from the SPF calibration process with the use of a maximum likelihood procedure. (In that process, a negative binomial distributed error structure is assumed with k being the dispersion parameter of this distribution.)

A factor is then applied to m to account for the length of the after period and differences in traffic volumes between the before and after periods. This factor is the sum of the annual SPF predictions for the after period divided by P, the sum of these predictions for the before period. The result, after application of this factor, is an estimate of B. The procedure also produces an estimate of the variance of B, the expected number of crashes that would have occurred in the after period without RLC.

The estimate of B is then summed over all intersections in a treatment group of interest (to obtain B_{sum}) and compared to the count of crashes during the after period in that group (A_{sum}). The variance of B is also summed over all sections in the treatment group.

The index of effectiveness (θ) is estimated as

$$\theta = (A_{sum}/B_{sum})/\{1 + [\mathrm{Var}(B_{sum})/B_{sum}^2]\} \tag{5}$$

The standard deviation of θ is given by

$$\mathrm{stddev}\ (\theta) = \left(\theta^2 \left\{\left[\mathrm{Var}(A_{sum})/A_{sum}^2\right] + \left[\mathrm{Var}(B_{sum})/B_{sum}^2\right]\right\} \middle/ \left[1 + \mathrm{Var}(B_{sum})/B_{sum}^2\right]^2\right)^{0.5} \tag{6}$$

The percentage change in crashes is $100(1 - \theta)$; thus a value of $\theta = 0.7$ with a standard deviation of 0.12 indicates a 30% reduction in crashes with a standard deviation of 12%.

Data Collection

The choice of jurisdictions to include in the study was based on sample size needs and the data available in potential jurisdictions. It was vital to ensure that enough data were included such that the expected change in safety can be detected with appropriate statistical significance. For a detailed explanation of sample size considerations, as well as estimation methods, see Chapter 9 of Hauer's book (*20*). The sample size analysis was conducted to examine two questions: (*a*) how large a sample is needed to detect statistically an expected change in safety? and (*b*) what changes in safety can be detected with likely available sample sizes?

The project team collected telephone survey data on the RLC systems and on the availability and quality of crash, signalization, traffic, and other data in 13 jurisdictions known to have significant RLC programs in the mid to late 1990s. On the basis of the sample size analysis undertaken in each of these jurisdictions, Table 2 was prepared to guide the choice of study jurisdictions. This table presents the authors' best judgment on the likelihood of detecting (at the 10% significance level) safety effects expected on the basis of the literature review, which revealed that it is not unreasonable to expect effects as large as a 25% decrease in right-angle crashes and a 30% increase in rear-end crashes. The italicized cities shown in the table were deemed to be the most feasible for availability of high-priority data elements (e.g., crash, average daily traffic, signal phasing changes, RLC signage). As indicated in the last row of Table 2, this group appeared to be large enough for the study.

As noted, data also were required for a reference group of signalized intersections, similar to the RLC locations, except that these were not equipped with RLCs. These sites were to be used in the calibration of safety performance functions and to investigate possible spillover effects. To account for time trends between the period before, the first RLC installation, and the period after that, crash and traffic volume data were collected to calibrate SPFs for a comparison group of approximately 50 unsignalized intersections in each jurisdiction.

After site and jurisdiction selection, the project team collected and coded the required data. In addition to crash data, information was collected on entering traffic volumes, basic geometric elements, signalization variables, RLC signing and publicity, other treatments applied during the study period, and changes in reporting practice. Before the data analyses, preliminary efforts involving file merging and data quality checks were conducted. This effort included the crash data linkage to intersections and the defining of RLC-related crashes. In general, RLC-related crashes would include right-angle crashes in the intersection itself where one vehicle is running the light, plus intersection-related rear-end crashes that could be affected by RLC systems, including those rear-end crashes occurring in the approach queue. Clearly, neither of these two types of crash is explicitly defined in crash data. Thus, the following definitions were used.

Red light running crashes included two basic types of involvement. First, there were right-angle, broadside, or right- or left-turning crashes involving two vehicles, with the vehicles entering the intersection from perpendicular approaches. Second, there were crashes involving

TABLE 2 Possibility of Detecting Safety Effects from Available Samples

	All Right-Angle	Injury Right-Angle	All Rear-End	Injury Rear-End
New York City	✓	✓	✓	✓
Howard Co., MD	✓	✓	✓	✓
Baltimore Co., MD	✓	✓	✓	✓
Charlotte, NC	✓	✖	✓	✖
San Diego, CA	✓	✖	✓	✖
San Francisco, CA	✓	✖	✓	✖
Montgomery Co., MD	✓	✖	✓	✖
El Cajon City, CA	✓	✖	✓	✖
Sacramento, CA	✖	✖	✓	✖
Prince George's Co., MD	✖	✖	✓	✖
Arlington, VA	✖	✖	✖	✖
Chandler, AZ	✖	✖	✖	✖
Boulder, CO	✖	✖	✖	✖
Feasible group (italicized)	✓	✓	✓	✓

✓ Significant results may be obtained.
✖ Significant results may not be obtained.

vehicles from opposing directions on the same road where one was turning left. The latter definition could not be limited to vehicles turning during a protected left-turn phase but would include red light running crashes in which a vehicle turning left at the end of a green phase (referred to as a sneaker in traffic engineering terminology) is broadsided by a vehicle from the opposing direction that is technically running a red light. Perpendicular approaches or opposing approaches were defined by using the compass directions of each involved vehicle's travel, a variable that was present in the data for all seven jurisdictions. In most jurisdictions, all crashes meeting these criteria and occurring in or within 20 ft (6.1 m) of the intersection center were captured.

Rear-end crashes used in the analyses were those defined as rear-end by the crash type and occurring on any approach within 150 ft (45.7 m) of the intersection. (Preliminary analysis of sites with vehicle direction data indicated that essentially all rear-end crashes within 150 ft of an intersection were approaching, rather than departing from, the intersection.)

In addition, consistent with previous research, injury crashes were defined as including fatal and definite injury crashes and excluding those classified as possible injury. Table 3 provides details of the number of sites in the treatment, reference, and comparison groups along with some basic statistics of the data collected at the treatment sites.

Calibration of Safety Performance Functions

As indicated earlier, the study required the development of SPFs for signalized and stop-controlled intersections. The signalized intersection SPFs were used to account for traffic volume changes and regression to the mean by using the EB procedure. The stop-controlled intersection SPFs were used to account for time trends in crash counts unrelated to the RLC installation. Therefore, it was necessary to first ensure that the comparison group used to calibrate the SPFs was suitable for this purpose, that is, that it had similar crash trends to the treatment group over the years before RLC installation. To this end a comparability test was performed (*20*). This test confirmed the suitability of the comparison group.

To build the strongest possible SPFs, reference group data were combined for sets of jurisdictions by considering proximity and similarity in crash reporting practices. The three California cities of El Cajon, San Diego, and San Francisco were combined. Not only are these three cities in close proximity, but they do not have full reporting of property-damage-only crashes, and the crash data all came from the state database maintained by the California State Highway Patrol. Howard County and Montgomery County, Maryland, were combined because of their close proximity and similarity in reporting practices. Baltimore, Maryland, and Charlotte, North Carolina, were combined because of their high reporting of noninjury crashes. In each case in which jurisdictions were combined, a jurisdiction-specific multiplier was calibrated and applied to account for any remaining differences in crash reporting.

Development of the SPFs involved determining which explanatory variables should be used, whether and how variables should be grouped, and how variables should enter into the model, that is, the best model form. Generalized linear modeling was used to estimate model coefficients by using the software package GenStat (*21*) and by assuming a negative binomial error distribution, all consistent with common research practice for developing these models. In specifying a negative binomial error structure, the dispersion parameter, k, which relates the mean and variance of the regression estimate, is iteratively estimated from the model and the data. The value of k is such that the smaller its value, the better a model is for a given set of data.

For specific crash types at signalized intersections, a factor is applied to the model that is equal to the proportion of total crashes that each crash type makes up. A value of k was calculated for each crash type by using a maximum likelihood program. Although data for groups of jurisdictions were combined for SPF calibration, separate factors were calculated for each jurisdiction. As well, jurisdiction-specific multipliers and k values were estimated in the regression calibration process.

TABLE 3 Treatment Site Data

Jurisdiction	No. of Treatment Sites (reference and comparison sites in parentheses)	Site-Months of Data	Entering AADT		Crashes		
			Min.	Max.	All	Right-Angle	Rear-End
El Cajon	6 (53, 38)	786	33,679	52,625	395	186	125
San Diego	19 (54, 44)	1,896	28,550	95,100	818	295	351
San Francisco	18 (52, 48)	2,298	26,474	100,718	1,427	867	251
Baltimore, MD	19 (86, 46)	1,985	13,748	80,330	1,345	426	389
Howard Co., MD	18 (34, 38)	3,530	13,490	62,362	1,088	306	565
Montgomery Co., MD	21 (55, 40)	2,178	23,100	107,700	1,562	628	599
Charlotte	31 (74, 42)	2,992	12,562	109,067	7,188	1,300	4,957
All	132 (408, 296)	15,665	12,562	109,067	13,823	4,008	7,237

The inclusion of variables such as number of lanes rarely significantly affected the fit. This is not surprising because, as previous research has shown, much of the variation in crash experience is explained by the volume of traffic entering an intersection. The results of the SPF calibration for the signalized reference group are presented in Table 4.

RESULTS

Results were obtained separately for the composite effects at the camera sites and at the reference sites analyzed for spillover effects.

Composite Effects at Camera Sites

Since the intent of the research was a multijurisdictional study, the aggregate effects over all RLC sites in all jurisdictions was of primary interest. Table 5 shows the combined results for the seven jurisdictions. As seen, there is a significant decrease in right-angle crashes (as defined earlier) but a significant increase in rear-end crashes. As seen in Table 6, the direction of these effects and the magnitude, to a lesser degree, were remarkably consistent across the jurisdictions.

Spillover Effects

To investigate possible spillover effects of RLC programs, a separate analysis was performed by using the untreated signalized intersection reference sites. For this, the before and after periods for these sites in each jurisdiction were demarcated by the year of the first RLC installation at the treatment sites. (Since specific treatment dates do not exist for each untreated reference site, this decision was based on the assumption that the public may have perceived that cameras were at noncamera locations from the time of the initial publicity campaign.) Table 7 gives the composite results of this analysis combining data from all of the jurisdictions. As seen, there are indications of a modest spillover effect on right-angle crashes. That this is not mirrored by the increase in rear-end crashes seen in the treatment group

TABLE 4 Safety Performance Functions for Signalized Intersections Reference Group

	El Cajon	San Diego	San Francisco	Howard County	Montgomery County	Baltimore	Charlotte
Model Form Crashes/Year	$\alpha(F1+F2)^b$			$\alpha(F1+F2)^b\exp(minllane*e)$		$\alpha(F1)^c(F2)^d\exp(majllane*f)$	
Three-legged							
Ln(α) (s.e.)	−5.240 (2.21)	−5.651 (2.22)	−5.240 (2.21)	−6.970 (1.800)	−6.970 (1.800)	−3.100 (1.240)	−3.100 (1.240)
b (s.e.)	0.580 (0.218)	0.580 (0.218)	0.580 (0.218)	0.709 (0.183)	0.709 (0.183)		
c (s.e.)						0.374 (0.119)	0.374 (0.119)
d (s.e.)						0.136 (0.080)	0.136 (0.080)
e (s.e.)				0.964 (0.297)	0.964 (0.297)		
f (s.e.)						0.264 (0.075)	0.264 (0.075)
Total α, k	1.00, 0.18	1.00, 0.28	1.00, 0.28	1.00, 0.30	1.00, 0.30	1.00, 0.56	1.00, 0.28
Injury α, k	0.28, 0.13	0.31, 0.26	0.26, 0.26	0.12, 0.30	0.24, 0.21	0.15, 0.91	0.07, 0.24
Right-angle α, k	0.40, 0.67	0.35, 0.91	0.55, 0.91	0.35, 0.37	0.28, 0.14	0.44, 1.0	0.25, 0.45
Rear-end α, k	0.41, 0.18	0.43, 0.25	0.22, 0.25	0.39, 0.63	0.44, 0.03	0.18, 1.1	0.61, 0.45
Model Form Crashes/Year	$\alpha(F1+F2)^b\exp(minrlane*e)$			$\alpha(F1+F2)^b$		$\alpha(F1)^c(F2)^d\exp(majllane*f)$	
Four-legged							
Ln(α) (s.e.)	−3.950 (2.010)	−4.624 (2.021)	−4.477 (2.021)	−8.370 (1.090)	−8.370 (1.090)	−3.100 (1.240)	−3.100 (1.240)
b (s.e.)	0.530 (0.197)	0.530 (0.197)	0.530 (0.197)				
c (s.e.)				0.703 (0.103)	0.703 (0.103)	0.374 (0.119)	0.374 (0.119)
d (s.e.)				0.335 (0.075)	0.335 (0.075)	0.136 (0.080)	0.136 (0.080)
e (s.e.)	−0.279 (0.129)	−0.279 (0.129)	−0.279 (0.129)				
f (s.e.)						0.264 (0.075)	0.264 (0.075)
Total α, k	1.00, 0.19	1.00, 0.24	1.00, 0.24	1.00, 0.20	1.00, 0.20	1.00, 0.56	1.00, 0.28
Injury α, k	0.26, 0.14	0.29, 0.10	0.26, 0.10	0.16, 0.20	0.25, 0.25	0.15, 0.91	0.07, 0.24
Right-angle α, k	0.48, 0.34	0.42, 0.38	0.55, 0.38	0.38, 0.36	0.48, 0.45	0.44, 1.0	0.25, 0.45
Rear-end α, k	0.32, 0.33	0.39, 0.48	0.22, 0.48	0.40, 0.45	0.32, 0.24	0.18, 0.9	0.61, 0.45

F1 = entering AADT on major road, F2 = entering AADT on minor road.
minllane = number of left-turn lanes on the minor road.
majllane = number of left-turn lanes on the major road; minrlane = number of right-turn lanes on the minor road.
(s.e.) = standard error of the estimate.
k is a calibrated parameter relating the mean and variance that are used in the empirical Bayes estimation procedure.

TABLE 5 Combined Results for Seven Jurisdictions

	Right-Angle		Rear-End	
	Total	Injury	Total	Injury
EB estimate of crashes expected in the after period without RLC	1542	351	2521	131
Count of crashes observed in the after period	1163	296	2896	163
Estimate of percent change (standard error)	–24.6 (2.9)	–15.7 (5.9)	14.9 (3.0)	24.0 (11.6)
Estimate of the change in crash frequency	–379	–55	375	32

A negative sign indicates a decrease in crashes.

TABLE 6 Results for Individual Jurisdictions for Total Accidents

Jurisdiction Number (in random order)[1]	Right-Angle Percent Reduction (standard error)	Rear-End Percent Reduction (standard error)
1	–40.0 (5.4)	21.3 (17.1)
2	+0.8 (9.0)	8.5 (9.8)
3	–14.3 (12.5)	15.1 (14.1)
4	–24.4 (8.7)	16.4 (11.3)
5	–34.3 (7.6)	38.1 (14.5)
6	–26.1 (4.7)	12.7 (3.4)
7	–24.4 (11.2)	7.0 (18.5)

A negative sign indicates a decrease in crashes.
[1]The identification of jurisdictions is not provided because of an agreement with the jurisdictions and the fact that it is irrelevant with respect to the findings.

detracts somewhat from the credibility of this result as evidence of a general deterrence effect.

DISCUSSION AND CONCLUSIONS

This statistically defendable study found effects that were consistent in direction with those found in many previous studies, although the benefits were somewhat lower that those reported in many sources. This indicates that regression to the mean may have been at play in many of those studies and emphasizes the need for controlling for these effects in an evaluation of red light camera programs and of road safety countermeasures in general.

The opposite-direction effects for rear-end and right-angle crashes deserves attention from two perspectives. First, the extent to which the increase in rear-end crashes negates the benefits for right-angle crashes is unclear at this point. An examination of the changes in crash numbers is insufficient to provide clarity on this issue because of differences in severity levels between right-angle and rear-end crashes and in the changes in these crashes following RLC installation. An examination of the economic costs of the changes based on an aggregation of rear-end and right-angle crash costs for various severity levels, which is intended to cast light on this issue, is the subject of a companion paper (*2*).

The second perspective of the opposing effects for the two crash types is the implication that RLC systems would be most beneficial at intersections where there are relatively few rear-end crashes and many right-angle ones. Better guidance on this issue requires an examination of the net effect, that is, the net economic benefit for intersections grouped by the numbers of each crash type. That examination, too, is the subject of the companion paper, which isolates the factors that would favor (or discourage) the installation of RLC systems by using the net economic benefit as the outcome variable.

The indications of a spillover effect point to a need for a more definitive study of this issue. That more confidence could not be placed in this aspect of the analysis is a reflection of the fact that this is an observational retrospective study in which RLC installations took place over many years and where other programs and treatments may have affected crash frequencies at the spillover study sites. A prospective study with an explicit purpose of addressing this issue is needed.

TABLE 7 Before–After Results for Total Crashes at Spillover Intersections

	Right-Angle	Rear-End
EB estimate of crashes expected in the after period without RLC	3430	3802
Count of crashes observed in the after period	3140	3873
Estimate of percent change (standard error)	–8.5 (2.2)	1.8 (2.3)

A negative sign indicates a decrease in crashes.

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Implementing Red Light Camera Programs

Guidance from Economic Analysis of Safety Benefits

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Red light camera (RLC) systems are believed to decrease the right-angle crashes that they are targeting but to have the undesirable side effect of increasing rear-end crashes. This belief was confirmed in a before–after study of 132 RLC installations in seven U.S. jurisdictions, as reported in a companion paper. In that research, the extent to which the increase in rear-end crashes negates the benefits for right-angle crashes was unclear, given the perceptions of severity differences in the two crash types. This paper reports on an examination of the changes in crash costs, based on a consideration of rear-end and right-angle unit crash costs for various severity levels, to establish the aggregate effects of the RLC programs evaluated. Part of the project derived the required unit costs by using information from national U.S. databases. The overall results show a modest to moderate economic benefit of between $28,000 and $50,000 per treated site year, depending on assumptions made. The ability to aggregate economic costs across crash types and severity created the opportunity to try to isolate program implementation factors and intersection characteristics that would favor the installation of RLC systems by using the aggregate economic benefit at each RLC site as the outcome variable. This investigation found, for example, that the greatest economic benefits are associated with the highest total entering annual average daily traffic, the largest ratios of right-angle to rear-end crashes, and the presence of protected left-turn phases.

A companion paper presented the elements of a multijurisdictional U.S. study, sponsored by FHWA, which estimated the effects of red light camera (RLC) systems on crashes (*1*). That study used before–after data for 132 RLC sites from seven jurisdictions (Montgomery and Howard Counties and Baltimore in Maryland; Charlotte, North Carolina; and the three California cities, El Cajon, San Diego, and San Francisco) and the empirical Bayes (EB) methodology (*2*) to estimate the effects on right-angle and rear-end crash frequencies. Consistent with what was found in other jurisdictions in previous studies, a reduction in right-angle crashes and an increase in rear-end crashes were detected overall and fairly consistently across the seven jurisdictions. Table 1, taken from the companion report, shows the results of the frequency analysis. (Injury crashes here are defined as Severity K, A, and B crashes on the KABCO scale. The frequencies shown do not contain the possible-injury crashes captured by KABCO Level C. Thus, these crashes could be labeled as definite-injury crashes.)

As Table 1 indicates, the crash frequency analysis is inconclusive on the aggregate effect of RLC systems because the extent to which the increase in rear-end crashes negates the benefits for right-angle crashes is unclear given the perceptions of severity differences in the two crash types. An examination of the aggregate economic costs of the changes based on a consideration of rear-end and right-angle unit crash costs for various severity levels was necessary to establish the overall effects of the RLC programs evaluated. This examination is the first of two analyses addressed in this paper. None of the previous studies of RLC effects reviewed attempted such a formal analysis of the aggregate effects for the various crash and severity types.

The second analysis is related to further examination of the implication of the crash-frequency results in the companion study that RLC systems would be most beneficial at intersections at which there are relatively few rear-end crashes and many right-angle ones. To provide better guidance on this issue requires an examination of the aggregate effect, that is, the aggregate economic benefit for intersections grouped by the numbers of each crash type. That examination, too, is the subject of the extended research covered in this paper, research that sought to isolate all the factors that would favor (or discourage) the installation of RLC systems, by using the net economic benefit as the outcome variable.

The project effort covered in the companion paper involved the choice of jurisdictions to study and the development of a detailed evaluation plan based on EB methodology, the collection of data from each chosen jurisdiction and the development of an analysis file, the EB analysis of before–after crash frequency data in each jurisdiction, and the combination of data from the diverse jurisdictions to develop an overall estimate of RLC effect on right-angle and rear-end crash frequency. Details of these aspects are described in the companion paper and are therefore not covered here. The following are covered:

- Development of per crash cost estimates for different crash types and police-reported crash severities,
- EB analysis of before–after economic cost of crashes in each jurisdiction by crash type and severity,
- Aggregation of the economic cost data across crash types and severity and then across jurisdictions,
- Exploratory analysis and regression modeling of cross-jurisdiction aggregate economic costs to identify the intersection and RLC program characteristics associated with the greatest economic benefits of RLC systems or that discourage their use, and
- Development of RLC implementation guidelines from these analyses of economic costs.

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TABLE 1 Combined Crash Frequency Results for Seven Jurisdictions

	Right-Angle[a] Total	Right-Angle Injury[b]	Rear-End[c] Total	Rear-End Injury
EB estimate of crashes expected in the after period without RLC	1542	351	2521	131
Count of crashes observed in the after period	1163	296	2896	163
% change in crashes (standard error) [negative indicates decrease]	−24.6 (2.9)	−15.7 (5.9)	14.9 (3.0)	24.0 (11.6)
Estimate of the change in crashes [negative indicates decrease]	−379	−55	375	32

[a]Defined as angle, broadside, or right- or left-turning crashes involving two vehicles from perpendicular approaches plus crashes involving a left-turning and a through vehicle from opposite approaches.
[b]Defined as definite injury (fatal, incapacitating, and moderate) and excludes possible injury.
[c]Defined as "rear-end" by the crash type and occurring on any approach within 150 ft of the intersection.

METHODOLOGY

Development of Unit Crash Cost Estimates

For this study, economic cost per crash was needed for the crash types of interest—costs for angle, rear-end, and other crashes at urban and rural signalized intersections. The crash cost to be used had to be keyed to police crash severity (injury = K, A, B, C; no injury = O) found in the analysis files available for use. In addition, because of limited sample sizes for fatal and severe injury (A) crashes that were found in the after periods for some intersections in the study, crash costs were needed for combined categories such as K+A severity.

Although numerous studies of injury costs have been conducted (*3*), all these were related to individual occupant injury cost rather than cost per crash, and the recent ones were all keyed to injury severity levels defined on an abbreviated injury scale (AIS) rather than on the KABCO scale found in police reports. Although FHWA had cost estimates for three levels of injury (i.e., fatal, injury, no-injury), these costs do not provide estimates for each level within the KABCO scale, and there had not been a conversion from AIS-based cost to KABCO cost for FHWA since 1994. Thus, new crash-based estimates were needed.

The Pacific Institute for Research and Evaluation (PIRE) has a long history of developing economic cost estimates, including work with both KABCO and AIS scaled injuries (*4, 5*). PIRE developed the cost estimates used in this RLC analysis as part of a larger effort of producing cost estimates for other crash types. Although the details of the estimate development are not presented in this paper, they can be found in a recent paper (*6*) and in an internal report available from FHWA (*7*). By merging previously developed costs per victim keyed on the AIS injury severity scale into U.S. traffic crash data files that scored injuries in both AIS and KABCO scales, PIRE economists were able to produce estimates for both economic (human capital) costs and comprehensive costs per crash. (The comprehensive cost estimates include both economic costs and costs associated with losses in the quality of life.) In addition, the analysis produced an estimate of the standard deviation and the 95% confidence intervals for each average cost. To meet the needs of this project and future FHWA projects, both comprehensive and human capital cost estimates were developed for six KABCO groupings within 21 selected crash types and within two speed limit categories (≤45 mph and ≥50 mph). The KABCO groupings ranged from detailed estimates for each level of crash severity within each crash type to combined levels of KABCO without regard for crash type. All estimates were stated in 2001 dollar costs.

EB Methodology for Obtaining Economic Costs

For simplicity, the theory is presented for estimating the change in crash costs over all treatment sites in a jurisdiction, for a specific crash type, aggregated over all KABCO subgroups (e.g., two subgroups: K+A+B+C and O). The crash types of interest are right-angle, rear-end, and other (i.e., other than rear-end and right-angle). The following notation is used:

Λ_{costA} = cost of crashes occurring at the treatment sites in the jurisdiction in the after period;
$Var(\Lambda_{costA})$ = variance of the cost of crashes in the after period;
Π_{costA} = expected cost of crashes in the after period over all treatment sites had there been no RLC (after correcting for regression to the mean and traffic volume and other differences between before and after periods);
$Var(\Pi_{costA})$ = variance of the expected cost of crashes over all treatment sites in the after period without RLC; and
Π_i = expected number of crashes in KABCO subgroup *i* over all treatment sites in the after period without RLC (after correcting for regression to the mean and traffic volume and other differences between before and after periods). These were derived for the crash frequency analysis presented in the companion paper by using the EB methodology.

The estimated change in crash costs is

$$\Phi_{cost} = \Pi_{costA} - \Lambda_{costA} \quad (1)$$

The variance of change in crash costs is

$$Var(\Phi_{cost}) = Var(\Pi_{costA}) + Var(\Lambda_{costA}) \quad (2)$$

The cost modification factor is

$$\theta_{cost} = (\Lambda_{costA}/\Pi_{costA})/\{1 + [\mathrm{Var}(\Pi_{costA})/\Pi_{costA}{}^2]\} \quad (3)$$

The variance of cost modification factor is given by

$$\mathrm{Var}(\theta_{cost}) = \theta_{cost}{}^2\{[\mathrm{Var}(\Lambda_{costA})/\Lambda_{costA}{}^2] + [\mathrm{Var}(\Pi_{costA})/\Pi_{costA}{}^2]\} / \{1 + [\mathrm{Var}(\Pi_{costA})/\Pi_{costA}{}^2]\}^2 \quad (4)$$

Of interest at this point is how estimates were obtained for the four terms Λ_{costA}, $\mathrm{Var}(\Lambda_{costA})$, Π_{costA}, and $\mathrm{Var}(\Pi_{costA})$.

The value of Λ_{costA} (i.e., actual after-crash cost) was estimated by summing the individual PIRE costs for each crash in the after period over all treated intersections in the jurisdiction. The value of $\mathrm{Var}(\Lambda_{costA})$ was estimated by summing the variance for each individual cost of the crashes of interest in the after period. Π_{costA} (i.e., the expected after period cost without treatment) was estimated for a KABCO subgroup by first estimating an expected cost for each site as the product of Π_i = expected number of crashes in the KABCO subgroup and the PIRE unit economic cost for the crash type, KABCO subgroup, and speed limit category. These were then summed over all treatment sites and KABCO subgroups to get Π_{costA}. $\mathrm{Var}(\Pi_{costA})$ for each site and subgroup was taken as a product of Π_i and the PIRE unit variance for the crash type, KABCO subgroup, and speed limit category. These variances were then summed over all sites and KABCO subgroups. This is an approximation that likely underestimates the variance, given that there is variance in the EB estimates of the expected number of crashes without treatment. However, the PIRE unit cost variances are also approximations in that they do not include all components (e.g., variance in medical costs by diagnosis). Fortunately, the point estimates of the economic effects, which are of primary interest in this analysis, are quite insensitive to $\mathrm{Var}(\Pi_{costA})$.

As noted, the theory so far applies for a given crash type of the three comprising all crashes. To obtain estimates of economic effect for all crash types combined, Λ_{costA}, $\mathrm{Var}(\Lambda_{costA})$, Π_{costA}, and $\mathrm{Var}(\Pi_{costA})$ are first determined for each crash type as outlined previously and then summed over the three crash types before Equations 1 through 4 are applied.

Identifying Factors Affecting RLC Effectiveness

Two types of disaggregate analysis were undertaken to identify factors associated with the greatest economic benefits or that might discourage the use of RLCs. The basic outcome measure used is the aggregate economic effects, that is, the combined economic effects on rear-end, right-angle, and other crashes of various severities. The economic effect for each crash type and severity was derived from Equation 1 as the difference between the expected cost of crashes in the after period had there been no RLC and the cost of crashes occurring at the treatment sites in the after period.

The first analysis was a univariate exploration of the results of aggregate economic effects for each intersection to identify factors that might be associated with the variation in the effects at individual sites. In this, two-dimensional plots and spreadsheets were used to sort the data and results for each site by various columns and to group by ranges of a variable to explore the relationship between factors and the measured aggregate economic effect per after period site year for a group as a whole for all crash types combined. The results of the exploratory analyses were used to guide a more formal analysis that used multivariate modeling to relate the aggregate economic effects to variables identified in the initial analysis as being of possible interest.

Some of these observations from the univariate exploratory analysis could result from correlation among the various variables that may affect the RLC impact. This could mask the effects or indicate effects that are not real. The more formal analysis described next was performed to mitigate the impact of this limitation. In this more formal disaggregate analysis, data for all jurisdictions were combined to develop a model to estimate the value of aggregate economic effect per site year for an individual site by using traffic volumes and other site characteristics (e.g., proportion of rear-end or right-angle crashes, signalization features) and RLC implementation features (e.g., publicity level) as explanatory variables. The model was linear with a normal error distribution and was of the form

$$\Phi_{cost} \text{ per after period year} = \alpha + b_1x_1 + b_2x_2 + b_3x_3 + \cdots b_nx_n \quad (5)$$

where α is the calibrated intercept and $b_1, b_2, \ldots, b_n$ are the estimated effects on Φ_{cost} per after period year of factors $x_1, x_2, x_3, \ldots, x_n$.

Stepwise linear regression was performed with the SAS statistical analysis software package by using the estimates of the Φ_{cost} per after period year as estimates of the dependent variable. The absence of a variable in the final model does not necessarily mean that the variable would not affect the safety impact of RLC, since an effect with low statistical significance could result from correlation with other variables, a lack of variation in the data, or a sample that is too small. In addition, the generally small size of the aggregate economic effect of RLC was already strongly indicative of the reality that one is unlikely to detect many factors that affect the safety effect of RLCs.

RESULTS AND DISCUSSION

Economic Cost Estimates

Because this analysis involved placing a value on fatal crashes, comprehensive cost estimates (which include quality of life losses) were used as recommended (*8*). This specific analysis was focused on angle and rear-end crashes at signalized intersections in urban areas. (Speed limits of ≤45 mph and ≥50 mph were used as surrogates for urban and rural here since it was not possible to define an urban/rural variable in the databases PIRE used; the effect of this approximation is likely small since only 10 of the 132 sites had speed limits of 50 mph or more although they were all in urban areas and therefore assigned urban unit costs.) Because the initially developed cost estimates for B- and C-level rear-end crashes indicate some anomalies in the order (e.g., C-level cost were higher, probably because on-scene police estimates of minor injury often ultimately include expensive whiplash injuries), the B- and C-level costs were combined by PIRE into one cost.

In initial economic analysis, an attempt was made to use three cost categories within each of the pertinent crash types—K+A, B+C, and no-injury. (It is not feasible to analyze fatal injuries separately in a study such as this with limited fatal crashes in any period. The cost of one fatal crash in any cell could significantly bias the results.) However, because of the low sample sizes of fatal and serious (A-level) crashes in the after period for some intersections, and because of the need to use the same cost categories across all intersections in all

TABLE 2 Original Comprehensive Crash Cost Estimates

Crash Severity Level	Angle Crash Cost	Rear-End Crash Cost
K	$4,090,042	$3,781,989
A	$120,810	$84,820
B	$103,468	$27,043
C	$34,690	$49,746
O	$8,673	$11,463
(Standard deviation)	(1,285)	(3,338)
K+A+B+C	$64,468	$53,659
(Standard deviation)	(11,919)	(9,276)

seven jurisdictions, two crash cost levels were ultimately used in all analyses—injury (K+A+B+C) and no-injury (O). The original estimate developed by PIRE and the combined cost per crash estimates used for each crash type are shown in Table 2, along with the variances for the two severity categories used in the analyses. [The cost estimate for an injury (K+A+B+C) crash for a given crash type is a weighted average of the individual costs for each KABCO level, with the weight based on the proportion of each crash injury level in the population of such crashes in the United States.]

EB Estimates of Economic Effects

Table 3 gives the results for the economic effects, including and excluding property-damage-only (PDO) crashes, estimated from Equations 1 through 4, and the associated procedures. The latter estimates are included in recognition that several jurisdictions considerably underreport PDO collisions. The columns labeled "all crashes" include nonangle, non-rear-end "other" crashes for which reliable unit costs could not be developed by PIRE because of small sample sizes. For completeness, the small changes in these other crashes needed to be accounted for in reporting effects on all crashes although the changes may be random and have nothing to do with RLC installation. It was decided that the same costs as for angle crashes would be used for the category "other."

The results show a positive aggregate economic benefit of more than $14 million over approximately 370 site years, which translates into a crash reduction benefit of approximately $38,000 per site year. The implication from this result is that the lesser severities and generally lower unit costs for rear-end injury crashes together ensure that the increase in rear-end crash frequency does not negate the decrease in the right-angle crashes targeted by red light camera systems.

Examination of the aggregate economic effect per after period year for each site indicates substantial variation, much of which could be due to randomness. However, it was reasonable to suspect that some of the differences may be due to factors that affect RLC effectiveness. The results of the examination of those factors are described next.

Since sample size considerations forced the combination of all injury crashes into one category (K+A+B+C), there was concern that the distribution of crash severity within this combined category might have changed between the before and after periods for either or both crash types. That is, injury-related angle crashes could have become more or less severe between the two periods. If so, the use of the same unit injury-crash cost for both periods would be questionable. Subsequent analysis of the before and after injury-crash distributions for both crash types indicated no discernable severity shift for rear-end crashes, but the angle crashes remaining in the after period might be more severe in the after period in two of the seven jurisdictions. An attempt was made to estimate the potential effect of this shift on the economic savings, although this could be done only by using anomalous data for individual KABCO categories whose use was argued against earlier. With these data, it appears that if the shift were real, the overall cost savings reported in the last row of Table 3 could be decreased by approximately $4 million. Note, however, that there would still be positive economic benefits, even if it is assumed that the unit cost shifts were real and correctly estimated.

Factors Affecting RLC Effectiveness

As detailed previously, this analysis involved exploratory univariate analysis and multivariate modeling aimed at identifying factors associated with the greatest and least economic benefits. The outcome measure in these models was the aggregate economic effect per after period site year. Data for all treated intersections in all seven jurisdictions were used in this analysis. However, as detailed in the companion paper (*1*), the different jurisdictions had different crash reporting thresholds, which resulted in significantly different numbers and percentages of noninjury crashes across jurisdictions. Since this analysis required that the crash costs for all intersections (and thus all jurisdictions) be calculated on a common basis, noninjury crashes were omitted from this analysis. Since the analysis is aimed at identifying factors of interest, and since these factors can be identified logically with injury crashes as with total crashes, this was believed to be proper procedure.

TABLE 3 Economic Effects

	All Severities Combined			PDOs Excluded		
	Right-Angle	Rear-End	All Crashes	Right-Angle	Rear-End	All Crashes
EB estimate of crash costs without RLC	$66,814,067	$69,347,624	$161,843,021	$61,687,367	$52,681,148	$134,407,104
Cost of crashes recorded after RLC (370 site years)	$48,319,090	$75,222,780	$147,470,550	$43,868,392	$53,944,539	$115,901,685
% decrease in crash cost (0.6) (s.e.) [negative indicates increase]	27.7 (0.7)	−8.5 (0.4)	8.9 (0.6)	28.9 (0.8)	−2.4 (0.5)	13.8
Crash cost decrease ($38,845) (per site year)			$14,372,471 ($50,015)			$18,505,419

NOTE: A combined unit cost for K+A+B+C is used.

The exploratory analysis led to the following general observations on the net economic effects:

- High publicity level (85 sites) is associated with a greater benefit than medium publicity level (47 sites). (Note that a high publicity level would be characterized by a major planned public information campaign, including such components as the FHWA public information program, combined efforts with other departments in the jurisdiction, for example, with the local health department, and television spots.)
- Fine plus demerit point penalty (90 sites) is associated with a greater benefit than a fine-only penalty (42 sites).
- Warning sign at intersections only (39 sites) is associated with a smaller benefit than warning sign at both intersections and city limits (73 sites).
- Benefits are greater at sites with one or more left-turn-protected phases (105 sites) than at those with no protected phases (27 sites). This variable may be a surrogate for the volume of left-turning traffic or opportunities for crashes involving a vehicle going straight through and one turning left at the end of a protected or permitted phase.
- There are indications that the aggregate economic benefit increases with total entering AADT, an increasing proportion of total traffic being on the major road and with an increasing ratio of right-angle to rear-end crashes.
- There are indications that the aggregate economic benefit increases with shorter cycle lengths and with shorter intergreen periods. These intuitive indications were derived despite the difficulty of defining these variables for a given intersection because of variation in them over the years and even over a single day. The maximum recorded values for these variables in the study period were used in the analysis in the absence of a more stable and pertinent measure of these factors.

Clearly, some of these variables that indicate effects are correlated and therefore may show effects that are not real. For example, left-turn protection is likely related to traffic volume levels, and high publicity levels may exist in jurisdictions with the highest traffic volumes. To mitigate this difficulty, multivariate regression analysis was undertaken to see if the effects of a given variable remain if the effects of other variables are simultaneously considered. This additional analysis confirmed the direction of all the preceding observed effects except the penalty variable and the one related to the presence of a left-turn-protected phase.

In interpreting these results, several important points should be considered:

- Factors other than the ones identified have been examined. These include traffic signal actuation, presence of turn restrictions, major road speed limit, and number of approach legs for which the inability to detect an effect may have been caused by the small samples for one level of the factor (e.g., only 27 of the 132 sites had no protected left-turn phases).
- The intent for the multivariate regression analysis was to confirm the direction of the effect, not to establish effects with statistical significance or to assess the size of the effect. To undertake analyses for these purer purposes would have required a substantially large database, much more precision in the estimate of economic effect at each site, and more accurate specification and measurement of the independent variables. For the purposes of this cursory investigation, it suffices that both the univariate and the multivariate analysis are reasonably in accord with the perceptions that are commonly held by those involved in red light camera programs.
- Some variables may be surrogates for others that more directly influence the aggregate economic effects. For example, the presence of left-turn protection, as noted, is likely associated with the volume of left-turning traffic or more directly with opportunities for crashes involving a vehicle going straight through and one turning left at the end of a protected or permitted phase.
- The results do not provide numerical guidance for trading off the effects of various factors. The intent for identifying these factors is to assist RLC implementers in the choice of sites for treatment installation and in determining the type of signing and publicity that might enhance the results of the program. For site identification, the results indicate that the implementer should give the highest priority for RLC implementation to the sites with most of or all the positive binary factors present (e.g., left-turn protection) and with the highest levels of the favorable continuous variables (e.g., higher ratios of right-angle to rear-end crashes).

On the basis of the combined univariate analyses and modeling, as well as a logical consideration of the result of the crash effects analysis that rear-end crashes increase and right-angle ones decrease following RLC implementation, it appears that the most important determinant of site choice is a high ratio of right-angle to rear-end crashes. Once site choice is made, signing at both intersection and city limits and a high-level publicity campaign would increase program benefits.

If it is of interest to quantify the aggregate benefit for a contemplated RLC site, it is possible to do so by using the SPFs and the likely estimates of safety effect presented in Table 1 in a procedure, the rudiments of which are documented in two recent publications (*9, 10*). In that procedure, crash and traffic data at the intersection of interest are used to obtain an EB estimate of the expected number of crashes by impact type and severity without RLC. The estimate of likely safety effect from Table 1 is applied to the EB estimate to derive an estimate of the expected change in crashes per year by type with RLC implemented. The cost per crash derived for this project can then be applied to the crash changes expected for each impact and severity type. The results can then be summed to obtain an estimate of the aggregate benefit per year for the contemplated installation.

CONCLUSIONS

This economic analysis represents the first attempt to combine the positive effects of right-angle crash reductions with the negative effects of rear-end crash increases and to identify factors that might further enhance the effects of RLC systems. Larger crash sample sizes would have added even more information. From these current analyses, the following primary conclusions are drawn:

- Although the positive effects on angle crashes of RLC systems are partially offset by negative effects related to increases in rear-end crashes, there is still a modest to moderate economic benefit of between $28,000 and $50,000 per treated site year, depending on whether one examines only injury crashes or includes PDO crashes and whether the shift to more severe angle crashes after treatment in two of the seven jurisdictions is indicative of a real trend.
- Even if modest, this economic benefit is important since, as operated today in many jurisdictions, the RLC systems pay for themselves through generated red light running fines. This differs from most safety treatments, in which there are installation, maintenance, and other costs that must be weighed against the treatment benefits.

- The modest benefit per site is an average over all sites. As the analysis of factors that impact effectiveness showed, this economic benefit can be increased through careful selection of the sites to be treated (e.g., sites with a high ratio of angle to rear-end crashes compared to other potential treatment sites) and program design (e.g., high publicity, signing at both intersections and at jurisdiction limits).

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Measuring Neighborhood Traffic Safety Benefits by Using Real-Time Driver Feedback Technology

Kevin N. Chang, Matthew Nolan, and Nancy L. Nihan

Local jurisdictions frequently respond to public concerns about speeding on neighborhood streets. When a speed study confirms that a significan percentage of vehicles are exceeding the posted speed limit, a traffic engineer carefully reviews the conditions to determine if additional safety measures need to be implemented. Preserving roadway safety for the motorized and the nonmotorized public alike who share the roadway is essential. Each jurisdiction is likely to use any number of solutions from its traffic safety toolbox. Additional signing, mobile radar speed display units, neighborhood speed watch programs, or targeted police enforcement may help discourage drivers from traveling at unacceptable speeds. Physical devices, such as traffic circles, speed humps, and chicanes, can also be considered but will affect emergency vehicles by increasing their response times. To balance increased driver awareness of travel speeds with vehicle accessibility, the King County Department of Transportation in Washington State installed four radar speed signs along 108th Avenue NE between NE 124th Street and Juanita-Woodinville Way NE. These radar speed signs, installed directly below the black-and-white regulatory speed limit signs, alerted each driver by indicating travel speed. To evaluate the effectiveness of these signs, speed studies were conducted before, during, and after installation. The results from these studies are presented, installation and maintenance of this device are discussed, and conclusions are drawn about whether these signs have been successful in calming neighborhood traffic.

Staff of local transportation agencies frequently address public concerns about speeding on neighborhood streets. The preservation of roadway safety for both the motorized and the nonmotorized public who share the roadway is essential.

For this research study, four radar speed signs were installed and analyzed for speed reduction effectiveness along 108th Avenue NE, a collector arterial in unincorporated King County, Washington. This paper elaborates on the results from the before and after studies, discusses installation and maintenance of this device, and draws conclusions about whether these signs have been successful in calming neighborhood traffic.

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BACKGROUND

The variability in the geography of King County, common in the Pacific Northwest, can create unique challenges for local traffic safety engineers. With a broad spectrum ranging from urban developments to more rural environments, flexibility is required during the planning and design stages of any traffic safety improvement so that the recommended solution will best meet the safety and user needs of a particular area. As a result of this wide array of landscape, countywide guidelines and protocol for traffic safety must occasionally be addressed on a case-by-case basis, and staff engineers recognize this need to adapt and adjust to each individual community (see Figure 1 and Table 1).

Because of this variability, King County engineers use a wide-ranging toolkit for traffic safety. When citizens call requesting a response to speeding, sight distance, or other traffic-related concerns, staff engineers will conduct a field investigation, collect speed and volume data, and meet with concerned citizens to identify specific issues. If a measured problem is present, engineers may install additional signage, request traffic enforcement by the King County sheriff's office, provide residents with use of a radar or readerboard vehicle, or work with neighborhoods and neighborhood associations to develop communitywide solutions identified and endorsed by the community. In the vicinity of 108th Avenue NE, an unincorporated King County neighborhood near Kirkland, Washington, radar speed signs (see Figure 2) were implemented as part of a King County Department of Transportation pilot project.

108TH AVENUE NE CORRIDOR

108th Avenue NE is a two-lane collector arterial with 10-ft-wide travel lanes, a 3- to 6-ft-wide paved shoulder, and a continuous 5- to 6-ft-wide sidewalk along both sides of the street. The roadway has a posted speed limit of 25 mph. The average daily traffic volume along the length of the corridor is approximately 2,700 vehicles on the north end to upwards of 4,900 vehicles on the south end. More than 10 local roads intersect 108th Avenue NE and provide direct access into neighborhood communities. This relatively straight roadway has long sight lines with some vertical sight distance concerns. Helen Keller Elementary School and Edith Moulton Park are directly served by 108th Avenue NE, and Juanita High School is located at the southern terminus.

Before the installation of the radar speed signs, staff from the King County Department of Transportation's road services division frequently responded to citizens concerned about traffic safety along this corridor. For traffic operations engineers, the documented com-

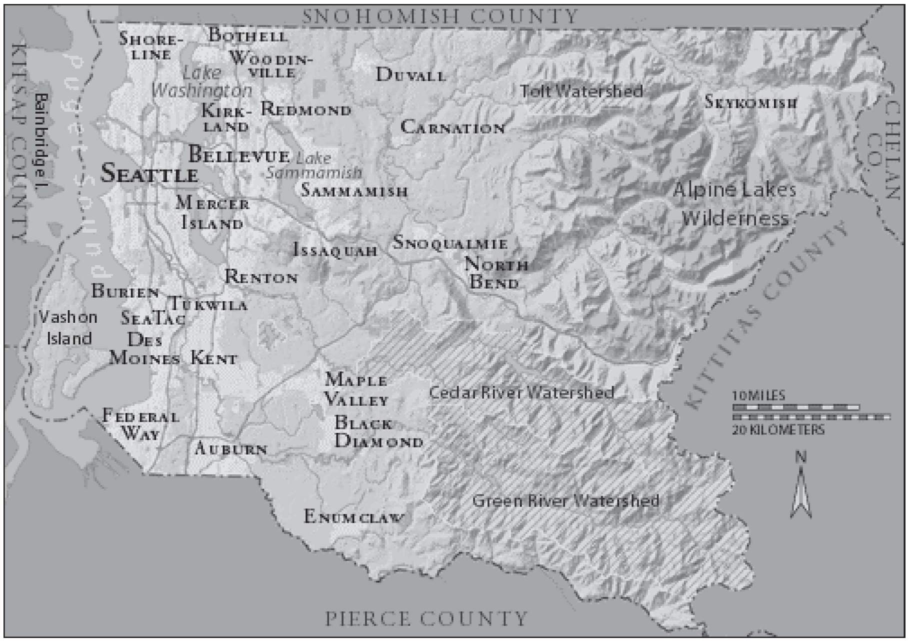

FIGURE 1 King County map.

plaints will sound quite familiar, with specific issues ranging from excessive traffic volumes and speeds, noise, and occasional drag racing to questionable driving behavior, particularly during high school dismissal times.

The installation of the radar speed signs was considered as a practical solution to balance neighborhood needs with mobility. The roadway was not a good candidate for an aggressive treatment such as physical devices since this roadway serves as an important response route for fire and other life safety vehicles. But the passive approach of adding additional signage and increasing traffic enforcement had shown limited benefit

The 25-mph posted speed limit was reviewed before the installation of the radar speed signs and was determined to be appropriate. Although the posted speed limit along collector arterials may vary from jurisdiction to jurisdiction throughout the country, the speed limit along 108th Avenue NE was based on, but not limited to, roadway classification, accident history, and roadway geometrics per existing King County guidelines.

RADAR SPEED SIGN DETAILS

Four radar speed signs were installed along 108th Avenue NE (see Figure 3). In the northbound direction, signs were installed on the east side of the roadway just north of NE 134th Street and north of NE

TABLE 1 King County Demographics

Population (2002)	1,760,000 (total) 355,000 (unincorporated)
Land	2,134 square miles (total) 1,768 square miles (unincorporated)
Managed roadways	1,794 miles of paved roadways 57 miles of unpaved roadways 760 miles of contract roadways

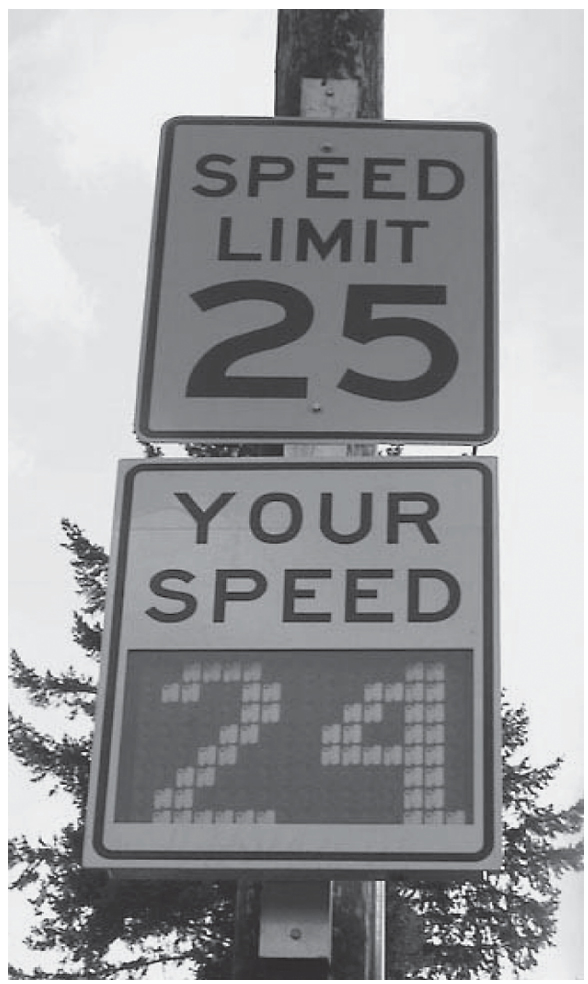

FIGURE 2 Radar speed sign.

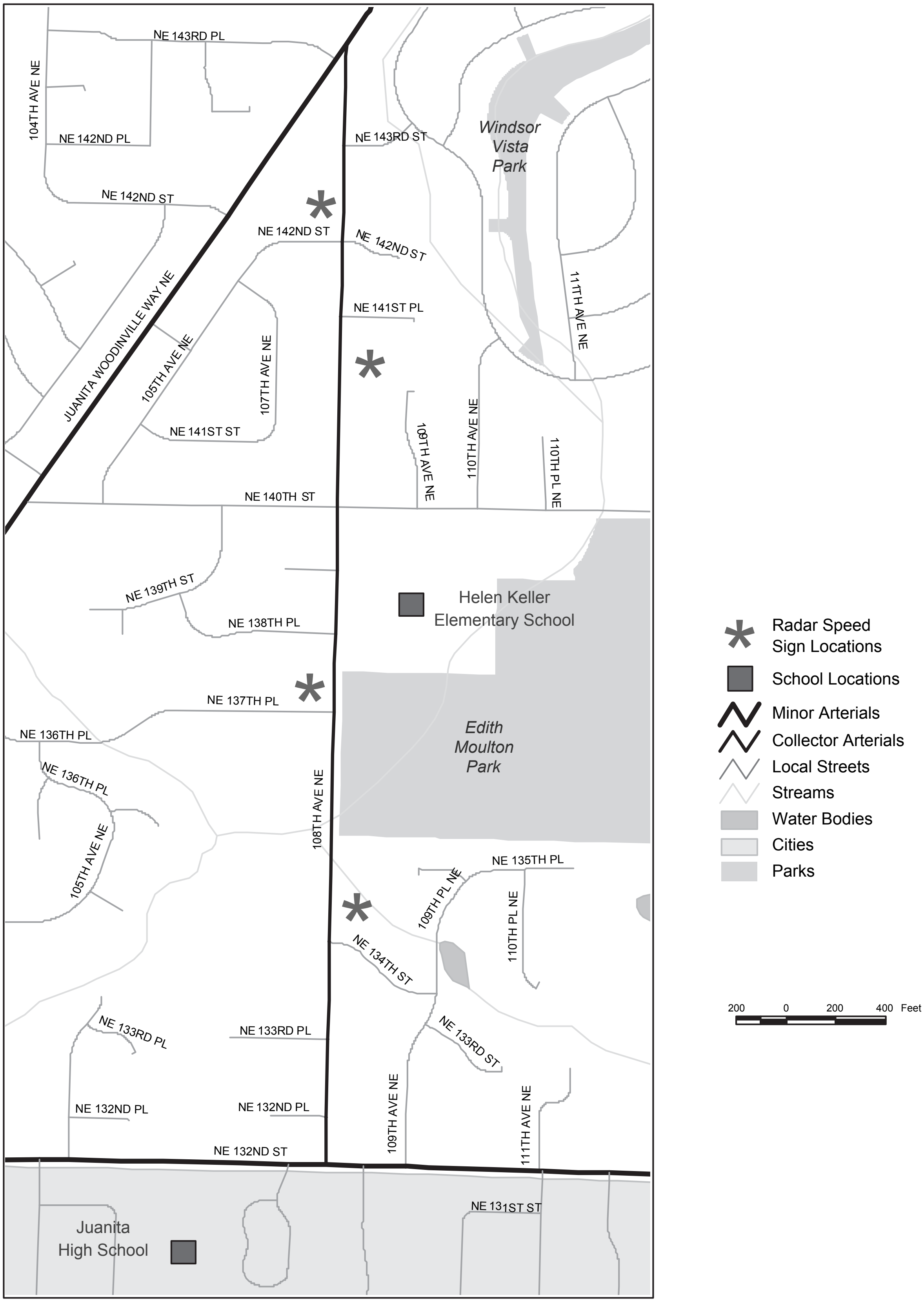

FIGURE 3 108th Avenue NE radar speed sign locations.

FIGURE 4 108th Avenue NE, north of NE 134th Street, looking north.

FIGURE 6 108th Avenue NE, north of NE 142nd Place, looking south.

140th Street. In the southbound direction, signs were installed on the west side of the roadway north of NE 142nd Street and north of NE 137th Place. Photos depicting these radar speed signs in operation at these locations are shown in Figures 4 through 7.

The total cost for each installed radar speed sign, including materials, tax, and staff time for design, coordination with local utilities, outreach, and installation, totaled approximately $8,000. Minor fluctuations in cost were attributed to varying roadside conditions and coordination time required.

The installed sign features a 12-in.-high fluorescent yellow-green readout and is the same overall size and style as the existing speed limit sign (24 × 30 in.). This size matches well with the residential character of the neighborhood. To draw additional driver attention, the display blinks when the vehicle speed exceeds the posted speed limit by 5 mph. The sign can also be programmed to blank out the screen once a high speed threshold is reached. This feature discourages drivers from speeding excessively to test the capabilities of the sign or their own driving audacity.

DATA RESULTS AND ANALYSIS

To capture the before-and-after effects of this device, average speed and volume data were collected at four locations (see Figure 8) along the roadway corridor in April 2001, February 2002, and early June 2002 before installation of the radar speed signs. The signs were activated on June 14, 2002. Average speeds and volumes were then collected at the same locations in mid-June 2002, January 2003, and April 2004. Tables 2 through 5 summarize the deviations in traffic speeds and volumes before and after installation. Although some of the newer radar speed signs can collect traffic data, all the data collected on 108th Avenue NE used traditional rubber hose technology. In some instances, the rubber hoses either were damaged or were

FIGURE 5 108th Avenue NE, north of NE 140th Street, looking north.

FIGURE 7 108th Avenue NE, north of NE 137th Place, looking south.

FIGURE 8 108th Avenue NE volume and speed count locations.

TABLE 2 108th Avenue NE Directional Volume Data, Before

Location	Data Type	April 2001	Feb. 2002	June 2002
Northbound				
North of NE 134th Street	Volume	2,935	2,484	2,712
North of NE 140th Street	Volume	1,568	1,363	1,486
Southbound				
North of NE 142nd Street	Volume	1,543	1,463	1,586
North of NE 133rd Place	Volume	2,579	2,534	2,794

TABLE 3 108th Avenue NE Directional Volume Data, After

Location	Data Type	June 2002	Aug. 2002	Jan. 2003	April 2004
Northbound					
North of NE 134th Street	Volume	2,363	2,337	2,332	2,342
North of NE 140th Street	Volume	1,352	1,286	1,270	1,409
Southbound					
North of NE 142nd Street	Volume	1,347	N/A	1,438	1,509
North of NE 133rd Place	Volume	2,353	2,246	2,533	N/A

TABLE 4 108th Avenue NE Speed Data, Before

Location	Data Type	April 2001	Feb. 2002	June 2002
Northbound				
North of NE 134th Street	Average speed	30.6	29.9	30.4
	85%ile speed	35.2	35.0	35.0
North of NE 140th Street	Average speed	21.9	27.6	27.0
	85%ile speed	25.4	32.1	31.7
Southbound				
North of NE 142nd Street	Average speed	27.9	28.0	27.1
	85%ile speed	32.2	32.4	31.6
North of NE 133rd Place	Average speed	30.5	30.9	30.7
	85%ile speed	34.8	35.5	35.7

TABLE 5 108th Avenue NE Speed Data, After

Location	Data Type	June 2002	Aug. 2002	Jan. 2003	April 2004
Northbound					
North of NE 134th Street	Average speed	28.7	30.2	28.1	28.2
	85%ile speed	33.1	34.5	32.4	32.4
North of NE 140th Street	Average speed	26.3	25.9	26.2	25.4
	85%ile speed	30.4	29.8	30.1	29.5
Southbound					
North of NE 142nd Street	Average speed	26.1	N/A	27.1	26.7
	85%ile speed	30.0	N/A	30.8	30.4
North of NE 133rd Place	Average speed	30.4	31.8	31.4	N/A
	85%ile speed	34.7	36.3	36.1	N/A

TABLE 6 108th Avenue NE Volume Data Summary

Location		Before		After		Results	
		Volume	σ	Volume	σ	t	% change
NE 133rd Place	NB	2710	225.5	2344	13.6	3.351	−13.5
NE 140th Street	NB	1472	103.2	1329	63.9	2.286	−9.7
NE 142nd Street	SB	1531	62.4	1431	81.2	1.691	−6.5
NE 134th Street	SB	2636	139.0	2377	145.0	2.233	−9.8

NB = northbound, SB = southbound.

inadequately secured in the field, so traffic speeds and volumes were not successfully collected in all cases.

The comparative results of traffic volumes and speeds before and after the installation of the radar speed signs are summarized in Tables 6 and 7. To account for seasonal fluctuations, these tables represent the cumulative data collected before and after installation. For roadway volumes, the installation of the signs resulted in an average volume decrease of up to 13.5%, but this decrease was statistically significant only at the NE 133rd Place location at the 95% confi dence level. Although the original intent of the signs was not to discourage drivers from traveling along 108th Avenue NE, the results suggest the possibility that some drivers view the signs as a nuisance and elect to use an alternative route.

The data collection results did determine a statistically significan difference between the before and after traffic speeds at the 95% confidence level. Traffic speeds decreased at three of the four locations, ranging from 4.26% to 7.15%, or 1.19 mph to 2.21 mph. Although this difference may appear incremental, the results indicated a change in driver behavior, particularly noteworthy because there were no modifications to the existing geometrics of the roadway.

At the NE 140th Street location, northbound traffic showed a statistically significant increase of 1.99%, or 0.51 mph. Because of this sign's close proximity to Helen Keller Elementary School, this location had the lowest average speed of the four locations before installation. It can be speculated that drivers were already respecting the rules of the road, and this suggests that these signs may be better suited for locations with higher average speeds from the outset.

The January 2003 and April 2004 data appear to indicate that at these measurement locations, lower speeds have been sustained. This is an extremely important finding. For this device to be effective it must satisfy this condition. The intent of these signs was not simply to incorporate developing technology at a trouble spot in hopes of drawing a short-term reaction. Rather, enabling drivers to permanently change their driving behavior—for the sake of neighborhood traffic safety—should bear more profound results and make these radar speed signs a worthwhile investment in the long run.

NEIGHBORHOOD INVOLVEMENT

On the basis of the experiences on 108th Avenue NE, King County has developed a process of implementation for future neighborhoods. Transportation engineering professionals recognize that complaints by neighborhood residents can stem from years of frustration over the perceived neglect of a public agency to listen to and understand the nature of the complaint. The following discussion outlines this process for other candidate locations for radar speed signs.

Before considering radar speed signs or any other community-wide solutions, county staff will host a neighborhood meeting to discuss the existing conditions with interested parties and identify possible solutions. If radar speed signs are preferred, an on-site investigation will be conducted to determine a physical range along the roadway where the signs would best meet the needs of the traveling public. Once that area has been determined, a map identifying that area and a petition form will be distributed to either the neighborhood liaison or the community association contact, whose responsibility will be to have the homeowners who live within that range agree to the installation of the radar speed signs. Homeowners on whose property or property line the sign is installed must recognize that there are trade-offs. The visual obstruction, along with any glare from the sign at night, must be explained as potential negative factors.

After the petition is signed and returned, county staff will coordinate with the local utility company to establish underground or overhead power to the proposed sign locations. The signs are installed after power is made available. Careful monitoring during the initial weeks of operation ensures that each sign is operating as desired and that all malfunctions are addressed in a timely manner.

LESSONS LEARNED

Traffic safety issues require any traffic engineer to consider many elements, including the raw data results as well as the concerns of neighbors to identify and implement necessary treatments that balance

TABLE 7 108th Avenue NE Speed Data Summary

Location		Before			After			Results	
		Speed	σ	Sample Size	Speed	σ	Sample Size	z	% change
NE 133rd Place	NB	30.89	4.71	15,424	28.68	4.64	24,890	46.0489	−7.15
NE 140th Street	NB	25.67	5.18	8,790	26.18	4.63	18,618	−7.8657	1.99
NE 142nd Street	SB	27.93	4.69	9,143	26.74	4.45	15,558	19.6203	−4.26
NE 134th Street	SB	31.56	6.41	15,901	30.00	4.66	33,606	27.4480	−4.96

roadway functionality with neighborhood traffic and pedestrian safety needs. Along 108th Avenue NE, a pilot project using radar speed signs was implemented. Although the data suggest a general decrease in volumes and speeds, the qualitative sentiment from the community indicates that a convincing traffic safety balance was achieved. This sentiment has immeasurable value. The community recognized that the occasional speeder will remain, but the solution implemented achieved the overall goal of improving livability. The neighborhood involvement factor, including outreach, education, input, and development of a mutually acceptable solution, established the framework of a consensus process and resulted in a solution that was acceptable and endorsed by any public agency's loudest critic—its community and tax-paying citizens who demand and expect the best service possible.

King County has not experienced any significant vandalism or maintenance issues regarding the radar speed signs. As mentioned, King County also offers its citizens the use of a radar readerboard trailer. The roadside location of the trailer made this particular treatment a regular target. The height and mounting of the radar speed sign as well as its permanency appear to have deterred vandals. Follow-up maintenance of the signs has been minimal. Although there were some initial adjustments, as expected for a new device, and minor start-up programming complications, there have been no major issues or periods of downtime.

Traffic calming is defined as "the combination of mainly physical measures that reduce the negative effects of motor vehicle use, alter driver behavior and improve conditions for non-motorized street users; traffic calming measures are intended to be self-enforcing" (*1*). On the basis of this definition, radar speed signs do represent a form of traffic calming, and these signs have shown to be an effective device with sustained traffic safety benefits

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Investigating the Sensitivity of Optimal Network Safety Needs to Key Safety Management Inputs

Godfrey Lamptey, Samuel Labi, and Kumares C. Sinha

This paper uses optimization methodologies and project-level cost and effectiveness data to assess long-term safety needs for a network. The optimal solution specifies values of decision variables (locations, years, and safety improvement types) such that overall cost-effectiveness at the network level is maximized under budgetary constraints. The paper evaluates the impact of alternative levels of safety funding on systemwide crash reduction and investigates the sensitivity of optimal funding levels to key safety management inputs. To demonstrate the methodology, data from Indiana's state highway system are used. It is shown that increases in overall safety funding have an increasing effect on crash reduction, but such increasing benefit tapers off after a certain point. It was determined that over a 10-year period (2005 to 2015) the optimal annual average safety need for the network is approximately $450 per mile. Furthermore, it is shown that the overall network safety funding need is sensitive to the method for identifying hazardous locations and the criterion for economic evaluation. The results show that with currently available data it is possible for highway agencies to incorporate road safety proactively into their transportation planning processes in a comprehensive and systemwide context. Also, agencies can use the methodology to determine optimal safety funding levels on their networks for possible comparison with current levels.

Highway asset management, of which safety management is an important component, advocates the combination of engineering principles with sound business practices and economic theory and involves the provision of tools that facilitate a more organized, logical, and integrated approach to decision making (*1*). The transportation sectors of state and local governments are therefore expected and encouraged to ensure appropriate use of public resources and operational accountability (*2*). Consistent with such asset management trends is the estimation of funding needs for cost-effective long-term management of highway infrastructure. It is expected that in the near future highway agencies will seek safety need estimates often, not on the basis of past practice (historical spending levels) as done now, but on a more rational and accountable basis, such as one that involves maximizing some systemwide utility (such as average crashes saved per dollar). With knowledge of the optimal levels of funding for safety-related highway projects, highway agencies are afforded a rational means with which to incorporate road safety proactively into their short- and long-range transportation planning processes in a comprehensive and systemwide context.

Highway safety enhancement efforts can be categorized as operator related, vehicle related, enforcement related, and environment related (including physical infrastructure). State and local highway agencies typically are responsible for addressing safety problems related to the physical road infrastructure, such as narrow lanes and shoulders. Most currently available safety needs analysis tools evaluate safety implications of alternative geometric designs at the project level. To extend such analysis to the network level, the only recourse appears to be the use of current tools to repetitively carry out project-level needs assessment for each individual section on the network. This not only is laborious but also provides a solution that often is not optimal from a systemwide perspective. With current availability of more detailed inventory and crash data, cost data, and improved safety management methodologies (*3–5*), a potential exists to extend such research to address contemporary issues of network-level safety needs. Methodologies useful for assessment of optimal network-level safety needs have been made available in past research (*6–8*) but have had little practical implementation, probably because of lack of certain analytical tools and data. FHWA is developing a comprehensive highway safety improvement model and software tools, SafetyAnalyst, to be made available in 2006.

The present paper investigates the optimal network safety needs and their sensitivity to key safety management inputs by using data from Indiana's state highway network.

ESTIMATION OF EXPECTED CRASH FREQUENCY

A basic requirement of network-level safety management is to identify sections on a road network that need some safety intervention now or will at some future time. This requires prior knowledge of estimated annual safety performance (crash frequency and severity) at various sections of the road network over an analysis period. Considerable research has been done on the prediction of expected safety performance of highway segments. For the present study, the crash prediction procedure is based on the empirical Bayesian (EB) method (*4*), which provides relatively unbiased estimates of the expected crash frequency. The EB method uses both a historical crash record and predicted crash frequency by using a multivariate crash prediction model. Crash records from 1997 to 2000 were obtained from a comprehensive Indiana state highway safety database. To exclude outliers from the data set, only sections with lengths between 0.1 and 10 mi and lane widths of less than 15 ft were used. Also, sections within a distance of 200 ft from intersections were excluded. By using the

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following general negative binomial form, crash prediction models were developed for six road categories:

$$\mu = LQ^{\phi} \exp\left(\beta_0 + \sum \beta_i x_i\right) \tag{1}$$

where

μ = expected crash frequency,
L = length of section,
Q = section average daily traffic,
x_i = explanatory variable, and
β_0, β_i, ϕ = constants.

The negative binomial distribution adds to the variance a quadratic term that represents the overdispersion and allows for extra Poisson variation due to variables not included in the model (*9*). Table 1 shows the crash prediction models developed in the present study.

For all road categories, the overdispersion factors obtained were highly significant, thus justifying the use of the negative binomial model form. The overdispersion parameters for the urban roads crash models were higher than those for the rural roads. This may be because of greater randomness at urban road sections relative to their rural counterparts. The lower value of the overdispersion factor for rural crash frequencies also means that the confidence in those models is relatively high (compared to their urban counterparts) because a smaller portion of the variance is explained by the error term. The R_α^2, which represents the model goodness of fit, was computed as follows (*10*):

$$R_\alpha^2 = 1 - \frac{\alpha}{\alpha_{max}} \tag{2}$$

where α_{max} and α are the overdispersion parameters estimated in the restricted and the unrestricted model, respectively. The overdispersion values obtained (0.46 to 0.73) suggest that the models have considerable predictive power. By using these models and observed crash records, the EB estimate of the expected safety performance of a location (highway segment) was then computed as follows:

$$\epsilon_i = \omega_i \mu_i + (1 - \omega_i) y_i \tag{3}$$

$$\omega_i = 1 \Big/ \left[1 + \mu_i \cdot (\alpha \cdot L_i)^{-1}\right] \tag{4}$$

where

ϵ_i = EB estimate of crash frequency on section i for the period for which historical crash data are available,
ω_i = weight factor,
μ_i = expected annual crash frequency on road section i from crash prediction model,
α = overdispersion factor of crash prediction model, and
y_i = number of observed crashes on road section i.

TABLE 1 Crash Prediction Models

Location	Crash Prediction Models	Overdispersion Factor	R_α^2
Rural two-lane segment	$\mu_{to} = L \times Q^{0.713} \times e^{(-4.692 - 0.049\,LW - 0.017\,RSW - 0.010\,FR - 0.025\,ARAD + 0.071\,AGRAD)}$	0.289	0.573
	$\mu_{fi} = L \times Q^{0.865} \times e^{(-7.075 - 0.070\,LW - 0.034\,RSW - 0.012\,FR - 0.022\,ARAD + 0.053\,AGRAD)}$	0.271	0.615
	$\mu_{pd} = L \times Q^{0.667} \times e^{(-4.685 - 0.042\,LW - 0.011\,RSW - 0.009\,FR - 0.025\,ARAD + 0.078\,AGRAD)}$	0.317	0.557
Rural multilane segment	$\mu_{to} = L \times Q^{0.719} \times e^{(-3.436 - 0.138\,LW - 0.004\,MW - 0.133\,AC - 0.050\,LSW)}$	0.264	0.487
	$\mu_{fi} = L \times Q^{0.801} \times e^{(-5.399 - 0.147\,LW - 0.005\,MW - 0.218AC - 0.058\,LSW)}$	0.292	0.517
	$\mu_{pd} = L \times Q^{0.685} \times e^{(-3.355 - 0.146\,LW - 0.004\,MW - 0.105\,AC - 0.046\,LSW)}$	0.273	0.458
Urban two-lane segment	$\mu_{to} = L \times Q^{1.144} \times e^{(-6.287 - 0.145\,LW + 0.054\,RSW - 0.272\,ST + 0.336\,TL - 0.316\,CRB)}$	0.852	0.579
	$\mu_{fi} = L \times Q^{1.357} \times e^{(-10.391 - 0.127\,LW + 0.074\,RSW - 0.254\,ST + 0.287\,TL - 0.251\,CRB)}$	0.878	0.647
	$\mu_{pd} = L \times Q^{1.108} \times e^{(-6.307 - 0.137\,LW + 0.050\,RSW - 0.262\,ST + 0.339\,TL - 0.289\,CRB)}$	0.799	0.586
Urban multilane segment	$\mu_{to} = L \times Q^{1.252} \times e^{(-7.153 - 0.158\,LW - 0.457\,AC - 0.276\,CRB - 0.261\,TL - 0.037\,FR)}$	0.748	0.730
	$\mu_{fi} = L \times Q^{1.164} \times e^{(-7.541 - 0.157\,LW - 0.569\,AC - 0.330\,CRB - 0.278\,TL - 0.064\,FR)}$	0.782	0.680
	$\mu_{pd} = L \times Q^{1.253} \times e^{(-7.600 - 0.156\,LW - 0.429\,AC - 0.248\,CRB - 0.255\,TL - 0.030\,FR)}$	0.705	0.731
Rural Interstates	$\mu_{to} = L \times Q^{0.794} \times e^{(6.450 - 0.566\,LW - 0.010\,MW - 1.378\,LSW)}$	0.680	0.568
	$\mu_{fi} = L \times Q^{0.870} \times e^{(0.096 - 0.418\,LW - 1.0011\,LSW)}$	0.368	0.624
	$\mu_{pd} = L \times Q^{0.778} \times e^{(6.328 - 0.571\,LW - 0.011\,MW - 1.3341\,LSW)}$	0.688	0.551
Urban Interstates	$\mu_{to} = L \times Q^{2.122} \times e^{(-8.999 - 0.7979\,LW - 0.0156\,MW - 0.254\,LSW)}$	1.370	0.510
	$\mu_{fi} = L \times Q^{2.092} \times e^{(-9.279 - 0.917LW - 0.178\,LSW)}$	1.661	0.486
	$\mu_{pd} = L \times Q^{2.069} \times e^{(-10.319 - 0.651\,LW - 0.018\,MW - 0.244\,LSW)}$	1.256	0.526

Where
μ_{fi} = expected annual fatal and injury crash frequency
μ_{pd} = expected annual PDO crash frequency
μ_{to} = expected annual total crash frequency
Q = annual average daily traffic for roadway section, in vehicles per day
L = roadway section length, in miles
LW = lane width (ft)
RSW = right shoulder width (ft)
MW = median width (ft)
LSW = left shoulder width (ft)
AC = access control (1 = none, 2 = partial, 3 = full)
FR = pavement friction (0 = worst, 1 = best)
TL = presence of turning lanes on segment (1 = turning lanes present, 0 = otherwise)
CRB = presence of curbs (1 = curbs present, 0 = otherwise)
ST = shoulder type (1 = unpaved, 2 = paved)
ARAD = average horizontal curve radius on road section
AGRAD = average vertical curve grade on road section

The crash estimates obtained from Equation 3 represent the expected crashes for the period for which historical crash data are available. To obtain future crash estimates, annual average daily traffic growth factors obtained from the Indiana Department of Transportation's traffic statistics unit were used to convert the expected crash frequency for the preceding period to expected crash frequencies for each year of the analysis period as follows:

$$F_{it} = \epsilon_i \times Q_i \times (1+g)^{n\beta_1(t-1)} \qquad t = 1, 2, \ldots, p \tag{5}$$

where

F_{it} = EB estimate of expected crash frequency at section i in analysis year t,
Q_i = traffic volume for section i for the period for which historical crash data are available,
g = traffic growth factor at section i,
n = number of years between the start of the analysis period and the period for which historical crash data are available,
β_1 = coefficient of the traffic volume variable in the crash prediction model at section i,
t = analysis year = 1, 2, . . . , p,
p = length of analysis period (years).

SELECTION OF CANDIDATE LOCATIONS

Highway safety programming involves the selection of road sections that require some safety attention while maximizing benefits from available safety funding for a network over an analysis period. Most highway agencies use some form of statistical analysis to select candidate locations for safety improvements (*11*). The most commonly used are the quality control and the crash severity methods, which involve actual or observed crash frequency. To avoid the regression-to-the-mean effect and to facilitate identification of candidate locations at future years, the present study modified these methods by replacing the actual crash frequency with the EB estimates of the crash frequency for each year within the analysis period. Thus, the best features of all available methods of candidate location identification were combined to establish the integrated quality control method (IQCM). IQCM involves the selection of a section as a candidate location for safety improvement if that section's expected crash frequency, crash rate, and equivalent property-damage-only (EPDO) rate exceed their respective critical values. Critical values were established by using the methodology suggested by Zegeer (*12*). Of the 2,372 sections on Indiana's state highway network, candidate locations were determined and ranked on the basis of their IQCM safety index value, IV, given by

$$\mathrm{IV}_{it} = \left(\frac{F_{it}}{F_{\mathrm{cr}}} + \frac{R_{it}}{R_{\mathrm{cr}}} + \frac{\mathrm{EPDO}_{it}}{\mathrm{EPDO}_{\mathrm{cr}}}\right)$$

and

$$\frac{F_{it}}{F_{\mathrm{cr}}} \geq 1 \qquad \frac{R_{it}}{R_{\mathrm{cr}}} \geq 1 \qquad \frac{\mathrm{EPDO}_{it}}{\mathrm{EPDO}_{\mathrm{cr}}} \geq 1 \tag{6}$$

where

IV_{it} = integrated quality control severity index value for section i in year t,
F_{cr} = critical crash frequency for section i,
R_{it} = EB estimate of crash rate at section i in year t,
R_{cr} = critical crash rate for section i,
EPDO_{it} = EB estimate of the equivalent property-damage-only (PDO) crash rate at section i in year t, and
$\mathrm{EPDO}_{\mathrm{cr}}$ = critical equivalent PDO crash rate for section i.

The IQCM may be considered satisfactory for both system and user because compared with other methods, it considers a relatively wide range of the important attributes of crash experience (crash frequency, severity, and traffic exposure) at a section. Use of IQCM generates a subset of candidate locations that are most deserving of safety intervention. A drawback of the IQCM method is that similar to other candidate locations methods, it does not consider potentially hazardous sites that have safety-related geometric deficiencies but have zero-crash histories. However, it may be argued that such limitation may not be debilitating to the analysis because very few sites have zero-crash histories.

IDENTIFICATION OF APPROPRIATE SAFETY IMPROVEMENT PROJECTS

After the safety improvement candidate locations were identified, the next step was to define the set of alternative safety improvement projects for each candidate location. These improvements vary from site to site, are based on the identified contributory crash factors, and can be placed into three categories: vehicle, driver, and road environment (including engineering or infrastructure-related factors). Of these three, it is the engineering category that can be most readily controlled by state highway agencies through the funding of physical safety projects. Second, there is a school of thought that addressing the engineering factors likely will lead to reduced influence of the other crash categories. In the present study, the methodology focuses only on the road environment safety improvement, particularly involving engineering factors that are related to the physical highway infrastructure. For each candidate location, the factors considered in selecting an appropriate safety project are discussed.

Deficient Roadway Geometric Features

The geometric features considered include right and left shoulder width, lane width, median width, access control, pavement friction, horizontal alignment, and vertical alignment. A roadway geometric feature at a given candidate location is considered deficient if its value at the location does not meet the recommended design value specified by the Indiana Department of Transportation's road design manual (*13*).

Expected Predominant Crash Pattern

The crash patterns considered in this paper are rear-end, head-on, opposite-direction sideswipe, same-direction sideswipe, off-road, and night crashes. A crash pattern is identified as predominant at a road section if its expected crash frequency at a given location significantly exceeds its critical crash frequency. The methodology assumes that the historical proportions of the crash patterns remain unchanged throughout the analysis period. Thus the expected frequency for the various crash patterns is obtained by distributing the expected crash frequency by using default estimates of the historical proportions among the various crash patterns. The critical frequency for each crash pattern is given as (*12*)

$$P_{c(iz)} = P_{az} + \delta_z \quad (7)$$

where

$P_{c(iz)}$ = threshold or critical crash frequency for crash pattern z at candidate location i,
P_{az} = average crash frequency for crash pattern z for similar road sections, and
δ_z = standard deviation for expected average crash frequency of crash pattern z on similar road sections.

Identification of Countermeasures

After the roadway deficiencies and predominant crash patterns were identified at each candidate location, the next step was to identify the set of appropriate safety countermeasures that would effectively mitigate or eliminate such deficiencies. A default set of alternative safety improvement projects was established for each roadway deficiency and predominant crash pattern (Table 2) on the basis of recommendations by Zegeer et al. (*5*). The default set of safety improvement projects may not always represent the full range of implementable safety projects at a site because of unavailability of detailed site data (such as weather and road surface conditions during crash, inadequate sight distance, and obstructions). By default, the do-nothing alternative was added as an alternative safety improvement project for each candidate location.

ECONOMIC ANALYSIS OF SAFETY PROJECTS

For a given candidate location, the best safety improvement project from a set of viable alternatives was chosen after the costs and benefits (over the analysis period) of all alternatives were estimated. A road section may fail to qualify as a candidate location in the base year but may qualify as such at a subsequent future year because of increasing traffic. A safety improvement project may be implemented at a given location in any year within the analysis period only when its implementation year equals or exceeds the critical year of that location. The critical year for a given location is defined as the year in which traffic at the section increases to such a level as to qualify it as a candidate location.

Estimation of Project Costs

Unit safety improvement project costs were obtained from the Indiana contracts database and several other sources (*8, 14–16*). For equitable comparison of alternative safety improvement projects with different service lives, the project costs were converted to their equivalent uniform annual amounts (EUAC) over the analysis period as follows:

$$\mathrm{EUAC}_{ijt} = C_{ijt} \cdot \left[\frac{1}{(1+r)^t} \cdot \frac{r(1+r)^n}{(1+r)^n - 1}\right] + M_{ijt} \cdot \left[\frac{1}{(1+r)^t}\right] - S_{ijt} \cdot \left[\frac{1}{(1+r)^{n+1}} \cdot \frac{r(1+r)^n}{(1+r)^n - 1}\right] \quad (8)$$

where

EUAC_{ijt} = EUAC for safety improvement project j at location i in year t,
C_{ijt} = initial construction cost for safety improvement project j at location i in year t,
M_{ijt} = annual maintenance cost for safety improvement project j at location i in year t,
S_{ijt} = residual value for safety improvement project j at location i in analysis year t,
r = minimum attractive rate of return, and
n = life span (service life) of the safety improvement project j.

TABLE 2 Default Safety Improvement Projects Considered

Road Environment Factor	Recommended Safety Improvement Project
Roadway deficiency	
Left shoulder width	• Widen left shoulder if less than design standard (2 ft or 4 ft)
Right shoulder width	• Install 6 ft right shoulder if not existent • Widen right shoulder if less than design standard (2 ft or 4 ft)
Lane width	• Widen roadway lanes if less than design standard (1 ft or 2 ft)
Median width	• Widen roadway median width if less than design standard
Access control	• Change access control from none to partial control
Horizontal alignment	• Realignment of horizontal curves
Vertical alignment	• Realignment of vertical grades
Predominant crash pattern	
Off-road	• Install 6 ft outside shoulder if not existent • Widen right shoulder if less than design standard (2 ft or 4 ft) • Install guard rail • Install rumble strips on outside shoulder
Head-on or opposite-direction sideswipe	• Widen roadway lanes if less than design standard (1 ft or 2 ft) • Install nonmountable median for two-lane road • Install rumble strips on inside shoulder if present
Same-direction sideswipe	• Install 6 ft right shoulder if not existent • Widen right shoulder if less than design standard (2 ft or 4 ft) • Widen roadway lanes if less than design standard (1 ft or 2 ft)
Rear-end	• Improve pavement friction if less than design standard • Install rumble strips in roadway pavement
Night crash	• Install or improve pavement markings • Install or improve roadway lightening

Estimation of Project Benefits

The benefits associated with each safety improvement project were measured for expected crash reduction. Crash reduction factors (CRFs) were obtained from the Indiana road design manual (*13*) and other sources (*17, 18, 8, 13*). The benefits were estimated in two alternative ways: in nonmonetary terms, as the annual crash reduction, or in monetary terms, such as the equivalent uniform annual benefit (EUAB) that incorporates crash costs, as follows:

$$\mathrm{CR}_{ijt} = \sum_{s=1}^{2} \left(F_{sit} \cdot \mathrm{CRF}_{sij} \right) \tag{9}$$

$$\mathrm{EUAB}_{ijt} = \sum_{s=1}^{2} \left[\frac{F_{sit} \cdot \mathrm{CRF}_{sij} \cdot \mathrm{CC}_{sit}}{(1+r)^{t-1}} \right] \cdot \left[\frac{r(1+r)^p}{(1+r)^p - 1} \right] \tag{10}$$

where

CR_{ijt} = crash reduction for safety improvement project j at location i in year t,
EUAB_{ijt} = EUAB for safety improvement project j at location i in year t,
F_{sit} = expected crash frequency of severity s at location i in year t,
CRF_{sij} = CRF for severity s associated with safety improvement project j at location i,
CC_{sit} = crash cost for severity s at location i in year t,
s = crash severity (1 = fatal or injury crash, 2 = PDO crash), and
r = minimum attractive rate of return.

Crash cost rates consistent with the economic approach for crash costing were used to estimate the present worth of benefits (*19*). These values measure only the direct and indirect costs of crashes (i.e., property damage, medical treatment, lost productivity, insurance administration and legal costs, and travel delay).

Economic Evaluation

To combine the benefits and costs into a commensurate unit for evaluation, the economic value (EV_{ijt}) of a safety improvement project j at location i at analysis year t was evaluated by using each of the following alternative economic evaluation criteria:

$$\text{cost-effectiveness} = \frac{\mathrm{CR}_{ijt}}{\mathrm{PWC}_{ijt}} \tag{11}$$

$$\text{net present value} = \mathrm{PWB}_{ijt} - \mathrm{PWC}_{ijt} \tag{12}$$

$$\text{benefit-cost ratio} = \frac{\mathrm{PWB}_{ijt}}{\mathrm{PWC}_{ijt}} \tag{13}$$

where

$$\mathrm{PWC}_{ijt} = \mathrm{EUAC}_{ijt} \cdot \left[\frac{(1+r)^p - 1}{r(1+r)^p} \right]$$

= present worth of costs for safety improvement project j at location i in analysis year t, and

$$\mathrm{PWB}_{ijt} = \mathrm{EUAB}_{ijt} \cdot \left[\frac{(1+r)^p - 1}{r(1+r)^p} \right]$$

= present worth of benefits for safety improvement project j at location i in analysis year t.

PROGRAMMING OF SAFETY INVESTMENTS

State and local highway agencies actively seek to proactively incorporate road safety in their short- and long-range transportation planning processes in a comprehensive and systemwide context. Thus, agencies seek the amounts of safety investments needed for their networks for a given time. Such programming can be addressed in two ways that both may be of interest to an agency: (*a*) assessment of all needs, assuming no budgetary restriction (i.e., unconstrained needs), and (*b*) assessment of needs given annual budgets in each year.

Safety Needs Assessment Study—Unconstrained Needs

The study methodology was applied to the state highway network in Indiana to determine the unconstrained physical and monetary safety needs for a 10-year analysis period (2005 to 2015). The expected crash frequency for each roadway section was estimated for each year of the analysis period by using the EB method and crash prediction models discussed earlier. The road sections deserving safety improvements were then identified and prioritized by using each of the following alternative methods for candidate location identification:

- Frequency quality control method (FQCM),
- Rate quality control method (RQCM),
- EPDO rate quality control method (EQCM), and
- Integrated quality control method (IQCM).

For each candidate location, a set of alternative safety improvement projects was identified by using the procedure described earlier. Cost computations for the safety improvement projects were carried out as described earlier. A discount rate of 4% [as suggested in the Indiana design manual (*13*)] was used to estimate the present worth of the project costs and benefits at the beginning of the analysis period. By using each of the three alternative economic analysis criteria, unconstrained optimization was carried out to select the best safety project to be implemented at each candidate location, assuming an infinite amount of funds were available. Table 3 shows the results of the physical and monetary needs for the Indiana state highway network for the period 2005–2015.

The results of the needs assessment study showed that the estimates of the physical and monetary safety needs are influenced by the method used for identifying safety candidate locations and the economic evaluation criteria used. It was found that different methods for identifying candidate locations yielded somewhat different sets of candidate locations. It was also observed from the results that 79% to 93% of all candidate locations were identified within the first year of the analysis period. This represents the cumulative backlog of safety improvement on the state highway network. Another, rather curious, result was that the set of candidate locations identified by using the IQCM method is a subset of all those identified by using the other

TABLE 3 Safety Needs Assessment for Indiana State Highway Network

	Candidate Location Method			
Safety Needs Assessment	FQCM	RQCM	EQCM	IQCM
Physical needs				
# of sections	641	237	332	158
Mileage	3438.88	1165.22	1725.63	775.78
Monetary needs				
NPV	$321,533,571	$120,619,737	$183,938,549	$102,676,971
Benefit–cost	$223,574,937	$69,649,075	$118,106,017	$54,281,985
Cost-effective	$235,493,229	$80,913,860	$122,180,418	$54,729,336

NOTE: Costs are in Year 2005 constant dollars.

three methods. This appears to suggest that the IQCM candidate locations largely represent those sites that most deserve some safety intervention. The results also show that the IQCM method yielded the highest monetary benefits and crash reduction for every dollar spent on safety improvements, as shown in Figure 1, making it the method that provides the greatest justification for safety investments.

The choice of economic evaluation criterion for analysis typically depends on agency policy. The study results showed that when either net present value (NPV) or benefit–cost ratio is used, the viability of alternative improvements (and hence safety project choice) is influenced by which costing method (comprehensive or economic crash cost) is used in the estimation of project benefits. As cost-effectiveness is a nonmonetary economic evaluation criterion, the resulting safety project selection is independent of the method used or monetary value of the crash cost estimate.

To demonstrate the remaining aspects of the study methodology by using the case study, candidate locations identified on the basis of the IQCM were used. The NPV economic evaluation criterion was used for the project evaluation and selection because it resulted in the highest benefits and total crash reduction, although the monetary benefit per dollar spent was the lowest. The use of the NPV economic evaluation criterion also satisfies the most important objective for the

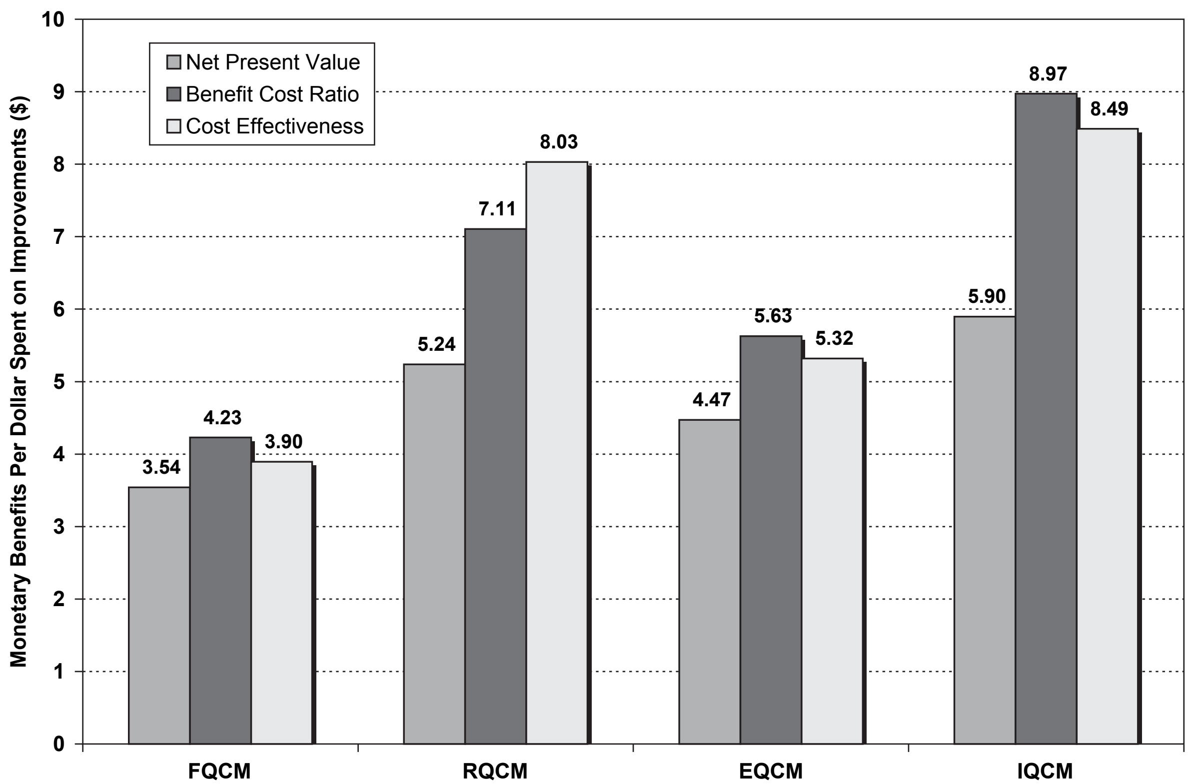

FIGURE 1 Monetary benefits per dollar spent on safety improvements.

implemented safety projects, that is, to minimize crash occurrence on the network by maximizing crash reduction.

Constrained Needs Assessments

In cases in which there is no budgetary constraint, the unconstrained needs assessment scenario is most suited for safety programming. However, most highway agencies typically have an annual budgetary limit for safety improvement projects. To establish realistic optimal safety programs at the network level, therefore, it is appropriate to consider such limitations. The establishment of optimal safety programs involves identifying the most appropriate safety improvement project and optimal time for implementation at each candidate location within the available budget for the analysis period. Sinha et al. (*6*) and Pal and Sinha (*7*) used integer programming techniques to develop resource allocation methodologies for highway safety improvements by maximizing cost-effectiveness. Harwood et al. reviewed various methods of resource allocation such as incremental benefit–cost ratio, integer programming, and dynamic programming and concluded that when formulated properly, these methods produce similar results (*8*). For the present study, a dynamic integer programming model was used that considered the time-dependent nature of resource allocation. The objective for the optimization is to maximize the total economic value for all the safety improvement projects selected during the analysis period.

In the unconstrained needs assessment, it was observed that the initial capital required in the first year of the analysis period to clear the backlog of candidate locations represents approximately 89% of the total cost of improvements. In practice, an agency may not have enough funds to clear such a large backlog. Agencies may prefer to phase such backlog clearing over time. As such, a multiyear investment strategy with annual budgetary constraints is more realistic. In this scenario, the agency seeks to determine which road sections to improve, what type of improvements, and in which year they should be carried out. It was assumed that the annual funding is uniform throughout the analysis period and that any unspent funding leftover from the previous year is not carried over to the following year, a scenario that is consistent with current practice at many agencies. The optimal allocation of the funding can be obtained by solving the following integer programming problem:

$$\text{maximize} \sum_{i=1}^{h} \sum_{j=1}^{d_i} \sum_{t=1}^{p} (x_{ijt}\text{EV}_{ijt}) \tag{14}$$

subject to

$$\sum_{i=1}^{h} \sum_{j=1}^{d_i} (x_{ijt}C_{ijt}) \leq B_{c(t)} \qquad \text{for all } t \tag{15}$$

$$\sum_{i=1}^{h} \sum_{j=1}^{d_i} \left(\sum_{k=1}^{t-1} x_{ijk}M_{ijk} \right) \leq B_{m(t)} \qquad \text{for all } t \tag{16}$$

$$\sum_{j=1}^{d_i} \sum_{t=1}^{p} x_{ijt} = 1 \qquad \text{for all } i \tag{17}$$

$$\sum_{i=1}^{h} \sum_{j=1}^{d_i} x_{ijt} \geq 1 \qquad \text{for all } t \tag{18}$$

$$x_{ijt} = 0 \qquad \text{if } t < \text{CR}_i \tag{19}$$

$$x_{ijt} = 0, 1 \tag{20}$$

where

- h = number of candidate locations within selected network;
- d_i = number of alternative safety improvement projects for candidate location *i;*
- $B_{c(t)}$ = annual capital budget for year *t;*
- $B_{m(t)}$ = annual maintenance budget for year *t;*
- CR_i = year when location *i* becomes hazardous (critical year);
- EV_{ijt} = economic value of safety improvement project *j* at location *i* in year *t;* and
- x_{ijt} = 1, if safety improvement project *j* is implemented at location *i* in year *t;* 0, otherwise.

Equation 14 represents the objective function of this integer program model, and the constraints are represented by Equations 15 through 20. Equation 15 constrains the annual capital expenditure to a level that is at most the available annual capital budget, whereas Equation 16 constrains the annual maintenance expenditure to a level that is at most that of the available annual maintenance budget and may be excluded from the analysis. Equation 17 ensures that in each year, only one safety improvement project is selected from the alternative safety improvement projects at each candidate location. Equation 19 ensures that a safety improvement project is not implemented at a location before the location becomes hazardous (in other words, it ensures that the implementation year of the project exceeds the critical year of the candidate location). Equation 18 requires that at least one safety improvement project be implemented in each year of the analysis period. The do-nothing "project" is also considered as an alternative. The solution of the integer programming model was accomplished by using the CPLEX solver. The optimal solution represents the implementation schedule, which specifies what safety improvement project should be implemented at each candidate location and in what year of the analysis period.

SENSITIVITY OF OPTIMAL NETWORK FUNDING LEVELS AND EFFECTIVENESS TO KEY SAFETY MANAGEMENT INPUTS

The optimal budget for the multiyear safety investment strategy for the identified candidate locations within the analysis period was carried out by using marginal effects analysis of the annual budget on crash reduction. Integer programming models (Equations 14 through 20) were used to develop separate multiyear safety investment strategies for nine budgeting scenarios involving annual amounts ranging from \$4 million to \$12 million. The results (Figure 2) demonstrate the sensitivity of the average annual expected crash reduction to annual safety budget for casualty (fatal and injury), PDO, and total crashes.

The results show a nonlinear relationship between the annual crash reduction and annual safety budget. The percentage increase in annual crash reduction for every \$1 million increase in annual safety expenditure depends on the current level of funding. On average, 107 crashes (36 fatal or injury and 71 PDO) are expected to be saved for every \$1 million increase in annual safety expenditure. Equation 21 shows the average annual crash reduction functions obtained from the sensitivity analysis results:

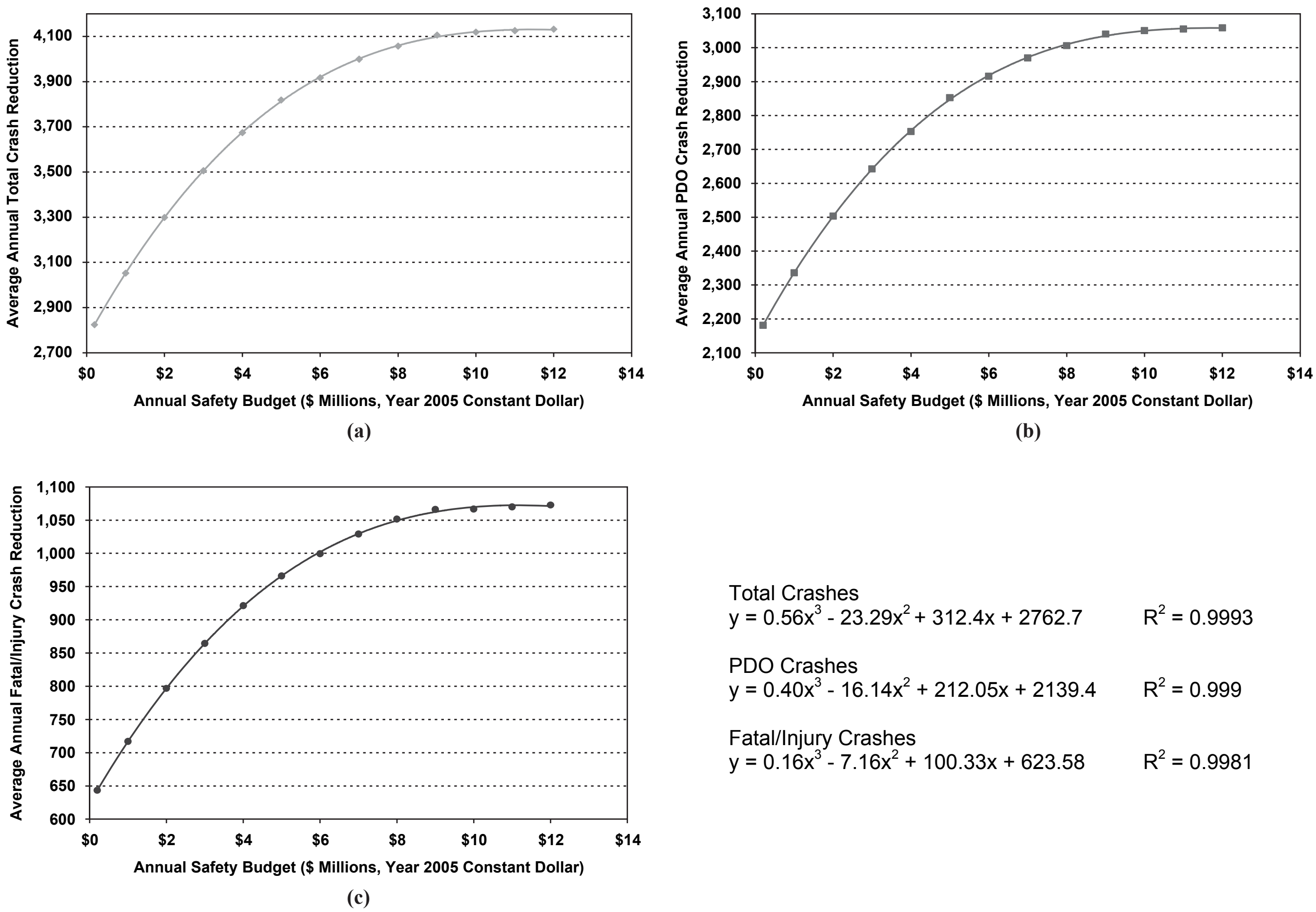

FIGURE 2 Sensitivity of annual safety budget levels on annual crash reduction.

total crashes:

$$y = 0.56x^3 - 23.29x^2 + 312.4x + 2762.7 \qquad R^2 = 0.9993$$

PDO crashes:

$$y = 0.40x^3 - 16.14x^2 + 212.05x + 2139.4 \qquad R^2 = 0.999$$

fatal or injury crashes:

$$y = 0.16x^3 - 7.16x^2 + 100.33x + 623.58 \qquad R^2 = 0.9981 \tag{21}$$

where y is the expected annual crash reduction and x is the annual safety budget.

By using these functions, the marginal effect of the annual safety budget on crash reduction was obtained, as shown in Figure 3. The marginal effect is the unit percentage change in annual crash reduction for every unit percentage change in annual safety expenditure and is given by the relation

$$\text{marginal effect} = \frac{dy}{y} \Big/ \frac{dx}{x} \tag{22}$$

The results show that the marginal effect of the annual safety budget on crash reduction is significantly different for the various crash severities. It was found that for a given percentage increase in safety expenditure, the expected percentage increase in annual fatal and injury crashes is higher than that of PDO crashes. This is because the selected safety improvements projects for implementation have relatively higher potential for fatal and injury crash reduction. The optimal annual budget (which is the turning point on the marginal effects curve) for multiyear safety investment strategy was found to be $4.7 million.

Optimal practice as identified in the preceding may not always be practical, because certain agencies may not be able to afford the funding levels associated with optimal practice. For such agencies, the optimization methodology could proceed with their budgetary constraint, as explained earlier. However, the benefits and cost-effectiveness of budget-constrained optimal practice are generally expected to be inferior to that of true optimal practice.

CONCLUSIONS

This paper addressed the issue of optimal funding amounts to address safety needs for physical highway infrastructure for a state highway network. The paper used methodologies developed in past research work but included new crash prediction functions. The paper also used a newly developed IQCM for identifying hazardous locations on the basis of both the supply side (roadway deficiencies) and the demand

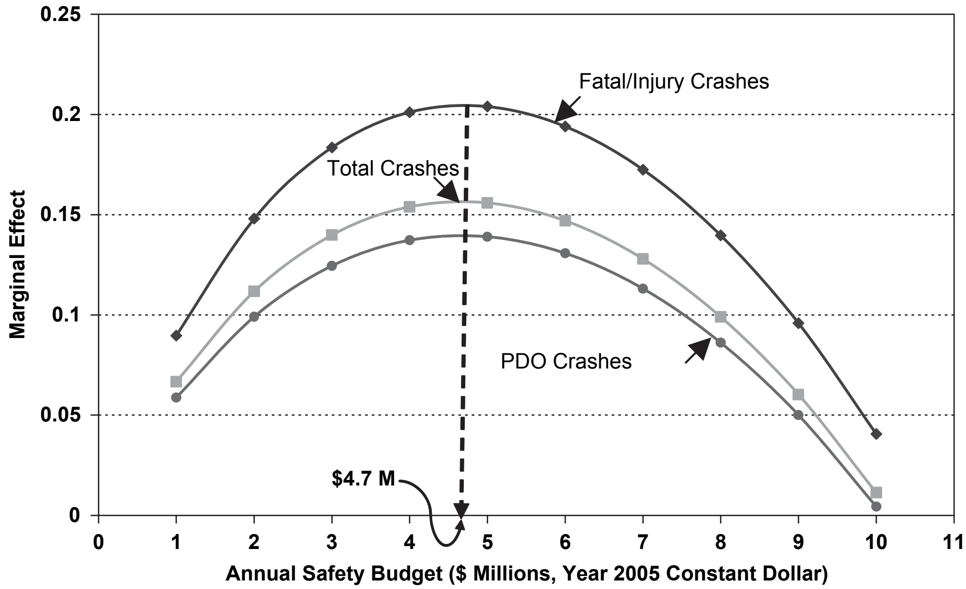

FIGURE 3 Marginal effect of annual safety budget level on annual crash reduction: (*a*) total crashes, (*b*) PDO crashes, and (*c*) fatal and injury crashes.

side (traffic volume and crash frequency). The paper identified candidate locations over a specified analysis period and selected safety improvement projects on the basis of identified roadway deficiencies and predominant crash patterns at each location. Life-cycle costing and integer programming techniques, together with newly developed crash-prediction functions, were used to determine which sections need improvement, what improvement is needed, and in what year.

The results showed that the estimated level of safety needs is influenced by the method used for identifying safety candidate locations and the criterion used for economic evaluation. Compared to the other methods of hazardous location identification, the IQCM yielded the highest monetary benefits and crash reduction for every dollar spent on safety improvements. This suggests that the IQCM could probably be the upper bound method that could be used by agencies to justify safety funding investments at a network level.

The study results showed that it is generally beneficial to increase safety funding and that higher safety investments will always yield nondecreasing effectiveness (crash reductions). However, it may not be cost-effective to increase safety funding beyond a certain point. As such, it is possible to determine an optimal point for safety investment. By using the Indiana state highway network as a case study, the paper estimated that the optimal level of funding for addressing the state's physical infrastructure safety needs for the 2005–2015 period is approximately $4.7 million (constant dollars). This corresponds to an average annual amount of $450 per mile. Furthermore, the results showed that the network-level safety needs are sensitive to key safety management inputs, such as the methods used for identifying hazardous locations and economic evaluation. A sensitivity analysis for crash types indicated that the marginal effect of budgetary levels on crash reduction was higher for fatal and injury crashes compared to PDO crashes.

With the methodology developed and demonstrated in this paper, agencies may determine the optimal levels of safety investments and therefore determine how existing practice compares to optimal practice for spending levels. It would also enable agencies to proactively incorporate road safety funding requirements into their short- and long-range transportation planning processes in a comprehensive and systemwide context. Besides facilitating integration of safety management into their overall transportation planning in such a manner, agencies can investigate the sensitivities of safety management inputs on effectiveness and cost-effectiveness at a network level. The methodology presented in the present paper as well as its findings (particularly, the sensitivity of investment decisions to key safety management inputs) may also be useful for consideration during the development, implementation, and validation phase of SafetyAnalyst (a set of analytical and software tools for decision making in development by FHWA) as well as other future research efforts.

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Safety Reviews of Existing Roads

Quantitative Safety Assessment Methodology

Alfonso Montella

Safety reviews of existing roads are becoming an accepted practice in many agencies around the world. These reviews can be highly cost-effective, but the subjective nature of the process can give rise to inconsistencies that limit their effectiveness. To address this issue, a technique to support safety reviews to quantify the safety gains that could be achieved by addressing the problems identified in the review process is presented. The approach is based on known accident relationships. A systematic process to determine which road features should be investigated and how each feature should be evaluated during the review is described. The procedure addresses rural two-lane highways at nonintersections. From the process, a potential for a safety improvement index (PFI) was calculated. Validation of the procedure was carried out by a comparison of the PFI values with the expected collision frequency. PFI was assessed in 406 km of rural two-lane rolling highways in Italy. Collision frequency was determined by application of a collision prediction model, calibrated in the study network, and was refined by application of the empirical Bayes (EB) technique. Correlation between EB safety estimates and PFI values is highly significant, with 93% of the variation in the estimated number of accidents explained by the PFI value. Because of the validation and quantitative nature of the PFI, the procedure can be used to support safety reviews and decision making.

In-service safety reviews aim to identify potential hazards, which are assessed by measuring risk in relation to road features that may lead to future crashes, so that remedial treatments may be implemented before crashes happen. From the review, safety issues and recommendations for improvement are derived.

Safety reviews are complementary and not alternative to accident investigation studies. Accident investigation is a reactive program; it examines past accidents and aims to remove or change the features that contributed to those past crashes. Safety review is a proactive program, aimed at reducing road accidents before they occur. Accident investigations tend to concentrate on single locations, whereas safety reviews are more akin to mass action studies. Moreover, the accident records are far from complete, not only in coverage but also in detail. In countries with poor accident statistics, the role of safety reviews as complement to accident investigation studies becomes more important. Indeed, the fewer the accident data, the less the information accidents can give about accidents to be prevented.

Safety reviews may be highly cost-effective. An Austroads research study reports that the analysis of a range of existing roads reviews indicated benefit–cost ratios (BCRs) between 2.4:1 and 84:1 when one considers the value of completing the proposed actions identifie in response to the review findings (*1*). More than 78% of all proposed actions had BCR > 1.0.

Even if safety reviews may be cost-effective, the subjective nature of the process may give rise to inconsistencies that limit their effectiveness. That the results of the review are a matter of judgment does not downgrade the value of the procedure. However, caution must be exercised if the results of one safety review are compared to another. There is no guarantee that two different review teams reviewing the same network will come up with exactly the same results. To address this issue, a quantitative method of safety impact assessment that complements in-service safety reviews is presented.

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RISK ASSESSMENT IN SAFETY REVIEWS

When review recommendations are considered, capital expenditure may be needed to address the safety issues identified to reduce the collision risk, and the owner would need to prioritize the remedial actions. Risk assessment helps determine the priority of safety issues identified by the safety reviews. Main existing road safety impact assessment procedures are presented, and advantages and drawbacks in their application are emphasized. Existing studies show that risk assessment is a key point in the development of the review process, but further research is needed.

Road Risk Index

In British Columbia, a criterion for a driver-based evaluation of road safety risk was developed (*2*). The process is based on well-define and quantifiable characteristics of road features that are studied and scored during a drive-through review. These scores are combined to produce a safety index, formulated by combining three components of risk: the exposure of road users to road hazards, the probability of becoming involved in a collision, and the resulting consequences should a collision occur. Specific and combined risk indices are assessed. The specific index defines the risk associated with each road feature, while the combined risk defines overall risk.

The methodology can effectively support safety review results. Nevertheless, it requires input data that in many instances are not available to the review team.

Road Protection Score

In 2002, the AA Foundation for Road Safety Research launched the Euro Road Assessment Programme. Part of the program is the development of a procedure for a drive-through inspection of routes and the assessment of the road protection score. The road protection score has been tested by scoring a sample of roads in seven countries, and further development of the scoring system has been proposed (*3*). A direct visual inspection of the road quality was used, and the roads were assessed by using the road protection score to measure the extent to which roads offer protection from accidents and from injury when collisions do occur. Risk tables have been developed on the basis of speed limit and road design features for the injury protection that the road provided in relation to three key accident types: head-on collisions, single vehicles leaving the road, and side impacts at intersections.

The road protection score differs from normal road safety reviews because its aim is to assess the general standard of a route rather than to identify individual sites of concern, but the methodology looks promising.

New Zealand Road Infrastructure Safety Assessment

In New Zealand, safety reviews of existing roads have been extensively carried out in the last decade. A Transfund manual of safety audits of existing roads defines a risk assessment procedure that involves the prediction of the frequency and severity of potential accidents associated with each problem identified in the audit report (*4*). A matrix is provided on which one axis is the exposure to risk and the other axis is the severity of the expected crash. The cells of the matrix are fille with words such as "low," "medium," and "high" level of importance. To assess the repeatability of the procedure, Transfund commissioned two independent safety audits of the same road network. The lack of common findings and the variation in assessing risk level ratings raised concerns about a lack of repeatability (*5*). Transfund also commissioned a study into the relationship between the issues raised by auditors and actual traffic crashes. This work produced widely varying results and showed that some of the assigned risk ratings were not accurate.

On the basis of these considerations, Transfund is developing a rating methodology to improve the systematic quantification of the safety impact associated with the items identified during safety reviews (*6*). The method, although not definitive, is very well suited as a support to the review process.

POTENTIAL FOR SAFETY IMPROVEMENT INDEX

General Aspects of Procedure

The main objective for developing a potential for a safety improvement index (PFI) was to produce a technique to support road safety reviews to quantify the safety gains that could be achieved by addressing the problems identified in the review process. The procedure looks at rural two-lane highways and does not take into account junctions. Key elements in developing the PFI procedure were as follows:

- Ensure that the PFI can be assessed as part of the safety review process without relevant supplementary work;
- Construct the process such that the results can be used to prioritize locations that hold promise for accident reduction; and
- Ensure that the PFI is valid by comparing the results with collision history.

The PFI assessment is based on evaluation of safety items that have a known impact on road safety. For each safety item, the relative increase in accident number and severity has been estimated. Safety reviewers, after a site investigation, by examination of videos recorded during the inspection, identify the presence of individual features and measure the approximate exposure length of each feature, dividing the road into homogeneous segments. By combining the different safety issues, exposure length, and relative increase in accident frequency and severity, the relative risk increase for injury and fatal accidents is computed. Potential for improvement is assessed for both injury and fatal accidents; it is equal to the product of the relative risk and traffic volume (raised to a power coefficient that depends on the accident predictive model calibrated in the study network).

Formulation of PFI

Ten general safety issues have been identified: alignment, cross section, markings, longitudinal rumble strips, pedestrian crosswalks, delineation, signs, pavement, roadside, and accesses. General issues are divided into detailed issues (see Table 1). The safety issues have been selected by considering that they are common issues and that effective remedial measures do exist and have already proved their effectiveness. On the basis of existing literature (*6–23*), the safety effect of each detailed issue has been estimated. The safety effect is expressed by two indices (see Table 1): ΔA, which represents the estimated relative increase in injury accidents risk caused by the safety issue, and ΔS, which is the estimated relative increase in accident severity. Accident severity is the ratio between fatal accidents and all-injuries accidents. Estimated relative increase in accident severity is different from 0 only for roadside issues. Since some safety features do not affect all accident types, related accidents have been defined for each detailed issue (see Table 1). Length of road affected by each item is expressed by the parameter related effect (see Table 1).

In each section of the road (it is suggested to assume that any one section is 200 m), the review team scores the detailed issues: 0 if the issue is not present, 1 if the issue is present (point items, such as not breakaway barrier terminals, are scored by their number). Scores are multiplied for the related effect and summed over all the sections; the ratio between the length of road affected by the safety item and the total length of the road (twice the length of the road for roadside items) represents the exposure of the safety item.

Relative risk of the detailed issue *j*, which represents the global estimated increase in injury accidents risk due to the issue *j*, is computed by the formula

$$\mathrm{RR}_j = \mathrm{expo}_j \times \Delta A_j \times P_j \tag{1}$$

where

RR_j = relative risk of the detailed issue *j*;
expo_j = exposure of the issue *j*, that is, the proportion of road affected by the issue *j*;
ΔA_j = estimated relative increase in injury accidents risk due to the issue *j*; and
P_j = proportion of accidents affected by the issue *j*.

Fatal accident RR_j is computed by the formula

$$\mathrm{RR}_{\mathrm{fa}j} = \mathrm{RR}_j \times \left(1 + \Delta S_j\right) \tag{2}$$

where $\mathrm{RR}_{\mathrm{fa}j}$ is the fatal accidents relative risk of the detailed issue *j* and ΔS_j is the estimated relative increase in accident severity (fatal and injury accidents) due to the issue *j*.

Relative risk of the general issue *i* is computed by the formula (equal to the formula for fatal accidents)

$$\mathrm{RR}_i = \sum_{j=1}^{n} \mathrm{RR}_j \tag{3}$$

TABLE 1 Safety Items

General Issues	Detailed Issues	ΔA (%)	ΔS (%)	Related Accidents	Related Effect
Alignment					
	Very severe curve realignment needed	100	0	All	200 m
	Inadequate sight distance on horizontal curves caused by removable obstacles (stopping sight distance, <0.75)	5	0	All	200 m
	Inadequate sight distance on crest curves (stopping sight distance, <0.5)	50	0	All	200 m
Cross section					
	Lane width				
	Very narrow <2.75 m	$5\text{–}50_{f(AADT)}$	0	Run off the road	Segment
	Narrow <3.25 m	$2\text{–}30_{f(AADT)}$	0	Head-on Sideswipe	Segment
	Shoulder width				
	Very narrow <0.3 m	$9\text{–}40_{f(AADT)}$	0	Run off the road	Segment
	Narrow <1.0 m	$6\text{–}20_{f(AADT)}$	0	Head-on Sideswipe	Segment
	Missing passing lane in section where there are not passing opportunities	33	0	All	Segment
	Missing climbing lane where high speed difference between cars and trucks do exist in mountainous terrain	33	0	All	Segment
Markings					
	Edgelines missing or inadequate	8	0	All	Segment
	Centerline missing or inadequate	13	0	All	Segment
	No-overtaking line missing	50	0	Head-on	Segment
Longitudinal rumble strips					
	Audible edgelines missing	40	0	Run off the road	Segment
	Audible centerline missing	11	0	Head-on	Segment
Pedestrian crosswalks					
	Missing or ineffective crosswalks in areas with pedestrian activity	60	0	Hit pedestrian	Segment
Delineation					
	Chevron missing or ineffective on severe curve	20	0	All	200 m
	Guideposts (or barrier reflectors) damaged or missing	8	0	All	Segment
Signs					
	Curve warning missing or not visible on severe curve	10	0	All	200 m
Pavement					
	Inadequate skid resistance	30	0	Wet	Segment

(continued)

TABLE 1 *(continued)* **Safety Items**

General Issues	Detailed Issues	ΔA (%)	ΔS (%)	Related Accidents	Related Effect
Roadside					
	Unshielded embankment (3<h<6m and i>0.5)	80	800	Run off the road	Segment
	Unshielded embankment (h>6m and i>0.5)	100	1,400	Run off the road	Segment
	Embankment shielded with very low containment (or ineffective) safety barrier (3<h<6m and i>0.5)	10	70	Run off the road	Segment
	Embankment shielded with very low containment (or ineffective) safety barrier (h>6m and i>0.5)	11	100	Run off the road	Segment
	Ditch	50	150	Run off the road	Segment
	Trees	90	1,000	Run off the road	50 m
	Rigid utility poles	90	1,000	Run off the road	50 m
	Rigid obstacles	90	1,000	Run off the road	25 m
	Not breakaway barrier terminals	60	300	Run off the road	25 m
	Missing transition between barriers (or between barrier and wall)	60	300	Run off the road	25 m
	Inadequate bridge rails	6	2,000	Run off the road	25 m
Accesses					
	Excessive density of uncontrolled accesses (>10/km)	75	0	All	Segment

h = height; i = longitudinal grade.

where

RR_i = relative risk of the general issue i,
RR_j = relative risk of the detailed issue j associated with the general issue i, and
n = number of detailed issues associated with the general issue i.

Relative risk of the segment, which represents the global estimated increase in injury accidents risk due to the identified issues, is computed by the formula (equal to the formula for fatal accidents)

$$RR = RR_1 + RR_2 \times (1 + RR_1) + RR_3 \times (1 + RR_2) \times (1 + RR_1) + \ldots \quad (4)$$

where RR is the relative risk of the segment and $RR_{1, 2, 3, \ldots, n}$ is the relative risk of the general issues.

PFI represents a measure of the accident increase due to the identified safety items. That is, PFI is a measure of the safety gains that can be obtained by eliminating the safety issues. It depends both on the relative risk and the traffic volume and is equal to

$$PFI = RR \times (AADT)^b \quad (5)$$

where AADT is the average annual daily traffic [(vehicles per day)/1,000] and b is the exponent of AADT in the pertinent accident predictive model.

Formula 5 is used also for calculating PFI_i of each safety item, by inserting in the formula the relative risk of the item. PFI_{fa} of fatal accidents is calculated by inserting into Formula 5 fatal accidents relative risk.

An example real-world application of the procedure is presented in Table 2.

Safety Issues

Many road features affect traffic safety, but not all factors can be considered in determining the PFI. It is important to point out that the safety effect of each item depends also on other road, traffic, and environmental features that all together play a key role. However, to make the assessment more objective, it has been decided to assign a relative increase in accident risk for each factor independent from the interaction of the different road features. The review team will decide if one item applies in relation to the road contest (e.g., chevron missing has to be evaluated in relation to the road alignment and perception).

Road alignment is the road factor with the greatest safety impact, even if its upgrading is generally quite expensive. Circumstances in which severe curve realignment is needed (e.g., horizontal radius less than 150 m following long tangents) give rise to an increase in the risk accident up to 100% applying accident modification factors reported by Harwood et al. (*7*). In the literature, severe curves are defined as curves where operating speed difference with preceding tangent is greater than 20 km/h (*8*); in the PFI procedure, curves with estimated operating speed differential greater than 30 km/h are classified as severe. Inadequate sight distance on horizontal and vertical curves

TABLE 2 Example Real-World Application of Procedure for Road Ex SS 400 dir

General Issue	Detailed Issue	$Expo_j$ (%)	ΔA_{ij} (%)	P_j(%)	RR_i (%) (see Eq.1)	ΔS_{ij} (%)	RR_{fa} (%) (see Eq.2)
Alignment					2.19		2.19
	Very severe curve	0.0	100.0	100.0	0.0	0.0	0.00
	Inadequate sight distance on horizontal curves caused by removable obstacles (<0.75 SSD)	43.75	5.0	100.0	2.19	0.0	2.19
	Inadequate sight distance on crest curves (<0.5 SSD)	0.0	50.0	100.0	0.0	0.0	0.0
Cross section					35.57		35.57
	Lane width				17.92		17.92
	Very narrow <2.75	81.25	50.0	44.12	17.92	0.0	17.92
	Narrow <3.25	0.0	30.0	44.12	0.0	0.0	0.00
	Shoulder width				17.65		17.65
	Very narrow <0.3	100.0	40.0	44.12	17.65	0.0	17.65
	Narrow <1.0	0.0	20.0	44.12	0.0	0.0	0.0
	Missing passing lane and passing opportunities	0.0	33.0	100.0	0.0	0.0	0.0
	Missing climbing lane where high speed differentials between cars and trucks do exist because of longitudinal grade	0.0	33.0	100.0	0.0	0.0	0.0
Markings					17.06		17.06
	Edgelines missing or poor	81.25	8.0	100.0	6.5	0.0	6.5
	Centerline missing or poor	81.25	13.0	100.0	10.56	0.0	10.56
	No-overtaking line missing	0.0	50.0	18.14	0.0	0.0	0.0
Longitudinal rumble strips					7.20		7.20
	Audible edgeline missing	81.25	40.0	17.16	5.58	0.0	5.58
	Audible centerline missing	81.25	11.0	18.14	1.62	0.0	1.62
Pedestrian crosswalks					0.96		0.96
	Missing or ineffective crosswalks in areas with pedestrian activity	27.27	60.0	5.88	0.96	0.0	0.96
Delineation					12.50		12.50
	Chevron missing or ineffective on severe curve	50.0	20.0	100.0	10.0	0.0	10.0
	Guideposts (or barrier reflectors) damaged or missing	31.25	8.00	100.0	2.5	0.0	2.5
Signs					1.25		1.25
	Curve warning missing or not visible on severe curve	12.50	10.0	100.0	1.25	0.0	1.25
Pavement					8.53		8.53
	Smoothing surface pavement	100.0	30.0	28.43	8.53	0.0	8.53
Roadside					3.95		41.45
	Unshielded embankment (3<h<6m and *i*>0.5)	0.0	80.0	17.16	0.0	800	0.0
	Unshielded embankment (h>6m and *i*>0.5)	12.5	100.0	17.16	2.15	1,400	32.18
	Embankment shielded with very low containment (or ineffective) safety barrier (3<h<6m and *i*>0.5)	0.0	10.0	17.16	0.0	70	0.0
	Embankment shielded with very low containment (or ineffective) safety barrier (h>6m and *i*>0.5)	12.5	11.0	17.16	0.24	100	0.47
	Ditch	0.0	50.0	17.16	0.0	150	0.0
	Trees	0.0	90.0	17.16	0.0	1,000	0.0
	Rigid utility poles	0.0	90.0	17.16	0.0	1,000	0.0
	Rigid obstacles	2.34	90.0	17.16	0.36	1,000	3.98
	Not breakaway barrier terminals	11.72	60.0	17.16	1.21	300	4.83
	Missing transition between barriers (or between barrier and wall)	0.0	60.0	17.16	0.0	300	0.0
	Inadequate bridge rails	0.0	6.4	17.16	0.0	2,000	0.0
Accesses					0.0		0.0
	Excessive density of uncontrolled accesses (>10/km)	0.0	75.0	100.0	0.0	0.0	0.0

$RR = RR_1 + RR_2 \times (1+RR_1) + RR_3 \times (1+RR_2) \times (1+RR_1) + \ldots$	125.55%
AADT [(veh/day)/1000]	12.425
b (exponent of AADT in the accident predictive model)	0.9722
$PFI = RR \times AADT^b$	14.54
$RR_{fa} = RR_{1fa} + RR_{2fa} \times (1+RR_{1fa}) + RR_{3fa} \times (1+RR_{2fa}) \times (1+RR_{1fa}) + \ldots$	206.93%
$PFI_{fa} = RR_{fa} \times AADT^b$	23.97

is a common accident contributory factor. Relative increase in accident risk due to inadequate sight distance (<75% stopping sight distance) on horizontal curves caused by removable obstacles has been assumed equal to 5% (*9*); relative increase in accident risk due to inadequate sight distance (<50% stopping sight distance) on crest curves has been assumed equal to 50% (*10*).

Lane and shoulder widths affect single-vehicle run-off-the-road and multiple-vehicle head-on, opposite-direction sideswipe and same-direction sideswipe accidents (*7*). The greater the lane and shoulder widths, the fewer the accidents. The effect of lane and shoulder widths depends on traffic volumes. Considering the task of the review team, which does not measure in continuum the pavement width, two classes of lanes and shoulders have been selected. Lanes are classified as very narrow if the width is less than 2.75 m and are narrow if the width is between 2.75 and 3.25 m. Shoulders are classified as very narrow if the width is less than 0.30 m and as narrow if the width is between 0.30 and 1.00 m. If AADT is more than 2,000, the relative increase in accident risk is 50% for very narrow lanes, 30% for narrow lanes, 40% for very narrow shoulders, and 20% for narrow shoulders. If AADT is less than 400, the coefficients are 5% for very narrow lanes, 2% for narrow lanes, 9% for very narrow shoulders, and 6% for narrow shoulders. For intermediate values of AADT, the coefficients vary linearly (*7*). Missing passing lane, in sections where there are not passing opportunities, and missing climbing lane, where high speed differences between cars and trucks exist in mountainous terrain, give rise to an increase in accident risk, which has been quantified equal to 33% (*7*).

Much research has investigated the effect of road marking on accidents, showing that road marking improvements are likely to be cost-effective. Detailed items considered are edge lines missing or inadequate, centerline missing or inadequate, and no-overtaking line missing in sections where passing sight distance is not provided. Relative increase in injury accidents risk has been assumed equal to 8% for edge lines missing and equal to 13% for centerline missing (*6*). Relative increase for no-overtaking line missing has been assumed equal to 50%; this factor applies only to head-on accidents (*6*).

An effective safety measure, which has been applied recently by many road authorities, is the installation of shoulder rumble strips (or audible edge lines), which are warning devices intended to alert drivers that they are leaving the traveled way and that a steering correction is required, and centerline rumble strips (or audible center line), which are intended to alert drivers that they have crossed the center of the road and are traveling in the opposing traffic lanes. The former have a positive effect on run-off-the-road accidents, the latter on head-on accidents. On the basis of Transportation Association of Canada (*11*) and NCHRP (*12–14*) suggestions, relative increase in accident risk due to rumble strips missing has been assumed equal to 40% for shoulders and equal to 11% for centerline, although other literature sources suggest even greater values (*15, 16*).

Missing or ineffective crosswalks in areas with pedestrian activity are one of the main contributory factors in pedestrian accidents. Relative increase in accident risk due to this safety issue has been assumed equal to 60% (*17, 18*).

Delineation is an important safety factor in any condition. On severe curves, missing or ineffective chevrons can lead to an accident risk increase equal to 20% (*6*). It has been assumed that this factor applies to a segment 200 m long. Damaged or missing guideposts or barrier reflectors on nonsevere curves and on tangents are also a safety deficiency; relative risk factor has been assumed equal to 8% (*6*). Some studies report positive effects associated with the installation of permanent raised pavement markers (PRPMs); however, recent comprehensive research tasks state that PRPMs have a positive effect only under certain particular conditions (*19*), and it has been decided not to include PRPMs in the safety issues.

Road signs that have the greatest effect on traffic safety are warning signs. They call attention to unexpected conditions and to situations that might not be readily apparent to road users, giving suggestions for safe behavior. For missing or ineffective curve warning signs on severe curves, the relative risk factor has been assumed equal to 10% (*6*).

The pavement factor that has more effect on road safety is friction. Relative risk increase when skid resistance is inadequate has been assessed equal to 30% (*6*); this applies to wet road accidents. Experimental results show even greater wet accident increase in poor friction conditions (*20*).

Roadside improvement measures may reduce either accident frequency or accident severity. Accident frequency can be reduced by removing or relocating roadside hazards to provide a clear zone along the roadside that provides errant vehicles an opportunity to recover and return to the travel way or to come to a controlled and safe stop. Accident severity can be reduced by making the hazards forgiving or shielding the hazards with road restraint systems. Injury accidents and fatal accidents risk increase, for different road features, has been calculated with the AASHTO severity indices (*21*). In relation to design speed, severity indices for each roadside feature define the probability of injuries and fatalities, given a collision. By comparing the injuries and fatalities probability of roadside obstacles to those of safety barriers, or of breakaway terminals, the risk increase factors reported in Table 1 were obtained. Length of road affected by risk increase was calculated by using the impact angle distribution reported by Mak et al. (*22*). Risk increase for safety barriers with low containment level and inadequate bridge rails was calculated by taking into account analytical relationships between a barrier's containment capacity and impact conditions that allow evaluating the number of vehicles successfully redirected in relation to the safety barriers containment level (*23*).

Direct access to roads can significantly increase accidents. Accident modification factors (AMFs) that take into account driveway density have been developed (*7*). AMFs show that a roadway segment with 10 driveways per kilometer can experience 75% more accidents than a segment with four driveways per kilometer.

VALIDATION OF PROCEDURE

A pilot study was done to evaluate the validity of the procedure. Values of the PFI index and expected collision frequency were compared.

Pilot Study

A pilot study was carried out as part of a safety review of a rural road network in Italy. The network is composed by 406 km of rural two-lane rolling highways with at-grade junctions and direct access from properties, located in the province of Avellino (Region Campania) and divided into 24 segments (see Table 3). Safety reviews were carried out by two experienced reviewers according to the procedures defined in the Italian road safety audit guidelines (*24*), and the PFI index was evaluated as a research task. Traffic data are based on traffic simulations (*25*) and ANAS (Italian National Roads Institute) traffic counts (for year 2000).

The accident data analysis was carried out by elaborating ISTAT (Italian National Institute of Statistics) electronic data of Region Campania for the period 1995–2002. Intersection accidents were excluded.

TABLE 3 Relative Risk and Potential for Improvement

Segment	Observed Injury Accidents	Segment Length (km)	Segment AADT (veh/day)	RR (%)	PFI
Ex SS 7 dir/c "Appia"$_{\text{(from km 12.6 to km 24.2)}}$	4	11.6	6,023	55.51	3.18
Ex SS 88$_a$ "Dei due Principati"$_{\text{(from km 15.6 to km 32.0)}}$	7	16.4	9,561	60.34	5.42
Ex SS 88$_b$ "Dei due Principati"$_{\text{(from km 36.0 to km 56.4)}}$	34	20.4	11,958	116.54	13.01
Ex SS 91$_a$ "Della Valle del Sele"$_{\text{(from km 0 to km 31.2)}}$	13	31.2	3,539	81.51	2.78
Ex SS 91$_b$ "Della Valle del Sele"$_{\text{(from km 31.2 to km 44.4)}}$	0	13.2	2,545	44.00	1.09
Ex SS 91$_c$ "Della Valle del Sele"$_{\text{(from km 44.4 to km 58)}}$	1	13.6	1,270	65.92	0.83
Ex SS 91 bis "Irpinia"	3	8.2	1,985	129.34	2.52
Ex SS 164$_a$ "Delle Croci di Acerno"$_{\text{(from km 34.2 to km 53.4)}}$	2	19.2	2,314	97.50	2.20
Ex SS 164$_b$ "Delle Croci di Acerno"$_{\text{(from km 53.4 to km 76.2)}}$	8	22.8	1,800	104.12	1.84
Ex SS 165 "Di Materdomini"	1	14.8	576	51.48	0.30
Ex SS 303$_a$ "Del Formicoso"$_{\text{(from km 20.2 to km 41.0)}}$	12	20.8	5,600	87.77	4.69
Ex SS 303$_b$ "Del Formicoso"$_{\text{(from km 41.0 to km 59.0)}}$	7	18.0	1,560	87.09	1.34
Ex SS 368 "Del Lago Laceno"	1	19.2	3,565	82.07	2.82
Ex SS 371 "Della Valle del Sabato"	7	10.8	4,532	95.18	4.14
Ex SS 374 "Di Summonte"$_{\text{(from km 0 to km 20.0)}}$	18	20.0	4,020	92.67	3.58
Ex SS 374 dir "Di Montevergine"	0	11.0	650	117.02	0.77
Ex SS 399 "Di Calitri"	8	19.8	5,204	68.46	3.40
Ex SS 400 "Di Catelvetere"	31	29.4	7,000	107.72	7.14
Ex SS 400 dir "Di Catelvetere"	6	3.4	12,425	125.55	14.54
Ex SS 403 "Della Valle di Lauro"$_{\text{(from km 3.0 to km 9.8)}}$	9	6.8	7,492	99.11	7.02
Ex SS 414 "Di Montecalvo Irpino"	15	18.6	3,191	130.67	4.04
Ex SS 428 "Di Villa Maina"	7	15.0	2,100	103.07	2.12
Ex SS 574 "Del Monte Terminio"	8	38.4	2,430	77.35	1.83
Ex SS 574 dir "Del Monte Terminio"	0	3.6	1,200	79.86	0.95
Total	202	406.2			

Ex SS indicates a regional road that was a national road.

The database includes injury and fatal accidents only. Most common accident types (see Table 4) are right-angle/turning (32.2%), head-on (18.3%), and run-off-the-road (17.3%). Right-angle accidents occur in proximity to accesses and are classified by ISTAT as non-intersection accidents. Accidents on wet pavement account for 28.7% of the total.

For each segment, relative risk and PFI were assessed (see Table 3). Relative risk ranges from 44% to 131%; that is, significant accident reductions may be obtained if road safety improvements are carried out. Ranking of safety issues in each segment shows that cross section, markings, and delineation generally are the safety issues with greater relative risk.

Accident History

The number of accidents expected to occur on the study segments was estimated by using the EB technique, which corrects for regression-to-mean bias (*26*). The estimate of the expected accidents depends on the accident count and the number of accidents predicted by a model.

A model that predicts the nonintersection collision frequency, as based on the segment length and the AADT volume, was developed with data reported in Table 3. Generalized linear modeling techniques (GLM) were used to fit the model, and a negative binomial distribution error structure was assumed. Several researchers have demonstrated the inappropriateness of conventional linear regression for modeling discrete, nonnegative, and rare events such as traffic collisions. GLM has the advantage of overcoming these shortcomings associated with conventional linear regression (*2*). The regression analyses were performed by using the GENMOD procedure in SAS.

The model form is as follows:

$$\hat{E}(Y) = e^{a_0} \times L^{a_1} \times \text{AADT}^{a_2} \tag{6}$$

where

$\hat{E}(Y)$ = predicted accident frequency (1995–2002),
L = segment length (km), and
a_0, a_1, a_2 = model parameters.

The model parameters and the indicators for the model significanc are given in Table 5. The reported indicators are the *t*-ratio for the model parameters, the κ-value (the negative binomial parameter), the scaled deviance, and the Pearson χ^2 statistic. The formulations of the scaled deviance (for a negative binomial distribution) and of the Pearson χ^2 statistic are shown in Equations 7 and 8. For a well-fitte model, both the scaled deviance and the Pearson χ^2 should be significant compared with the value obtained from the χ^2 table for the given degrees of freedom. These measures indicate that the prediction model has a relatively good fit and the values that are calculated for the *t*-ratios for all independent variables are significant

$$\text{SD} = 2\sum_{i=1}^{n}\left\{y_i \ln\left[\frac{y_i}{\hat{E}(y_i)}\right] - (y_i + \kappa)\ln\left[\frac{y_i + \kappa}{\hat{E}(y_i) + \kappa}\right]\right\} \tag{7}$$

TABLE 4 Aggregate Accident Data

	Injury Accidents		Fatalities		Injuries		Fatalities/ Injury Accidents
	N	%	*N*	%	*N*	%	
Head-on	37	18.32	3	23.08	85	22.79	8.11%
Right-angle/turning	65	32.18	2	15.38	124	33.24	3.08%
Sideswipe	17	8.42	0	0.00	33	8.85	0.00%
Rear-end	21	10.40	0	0.00	41	10.99	0.00%
Hit pedestrian	12	5.94	2	15.38	14	3.75	16.67%
Hit stopped vehicle	5	2.48	1	7.69	6	1.61	20.00%
Hit parked vehicle	1	0.50	0	0.00	3	0.80	0.00%
Hit obstacle in carriageway	6	2.97	2	15.38	8	2.14	33.33%
Run-off-the-road	35	17.33	3	23.08	51	13.67	8.57%
Sudden braking	1	0.50	0	0.00	5	1.34	0.00%
Falling from a vehicle	2	0.99	0	0.00	3	0.80	0.00%
Total	202	100.00	13	100.00	373	100.00	6.44%
Wet	58	28.71	1	7.69	125	33.51	1.72%
Other	144	71.29	12	92.31	248	66.49	8.33%
Total	202	100.00	13	100.00	373	100.00	6.44%

where

SD = scaled deviance,
y_i = observed number of accidents in the segment *i,*
$\hat{E}(y_i)$ = predicted number of accidents in the segment *i,* and
κ = negative binomial parameter.

$$\text{Pearson } \chi^2 = \sum_{i=1}^{n} \frac{\left[y_i - \hat{E}(y_i)\right]^2}{\text{Var}(y_i)} \tag{8}$$

where $\text{Var}(y_i)$ is the variance of the observed accidents.

The collision estimates were then subjected to an EB refinemen technique to obtain a better estimate of the existing safety performance (see Table 6), produced as follows:

$$\text{EB} = \left[\frac{\hat{E}(Y)}{\kappa + \hat{E}(Y)}\right] \times (\kappa + \text{count}) \tag{9}$$

where count is the observed collision frequency.

Comparison of PFI and Accident History

To test the procedure, comparisons of the PFI scores and the EB safety estimates were carried out (see Table 7 and Figure 1). Since PFI is assessed per unit of length, it can be compared to the number of accidents per year and per kilometer. EB estimates have been divided for the road segment lengths and the number of years.

The correlation between EB safety estimates and PFI values is highly significant ($t = 17.39$, p-value < 0.001), with 93% of the variation in the estimated number of accidents explained by the PFI value. This means that the relationship between EB estimates and PFI scores had less than 0.1% chance of occurring by accident.

To determine the level of agreement between the sorting of segments based on EB estimates and PFI values, each of the 24 segments was ranked in descending order according the two criteria, and the Spearman's rank-correlation coefficient was calculated by the formula (see Table 7)

$$\rho_s = 1 - \frac{6 \times \sum_{i=1}^{n} d_i^2}{n \times (n^2 - 1)} \tag{10}$$

where

ρ_s = Spearman's rank-correlation coefficient,
d_i = differences between ranks, and
n = number of paired sets.

Under a null hypothesis of no correlation, the ordered data pairs are randomly matched, and thus the sampling distribution of ρ_s has a mean of zero. Since this sampling distribution can be approximated with a normal distribution even for relatively small values of *n,* it is possible to test the null hypothesis on the statistic given by

$$z = \rho_s \times \sqrt{(n-1)} \tag{11}$$

TABLE 5 Model Parameters

df	Parameter	Estimate	*t*-Ratio	$t_{0.05,\,21}$	κ	SD	Pearson χ^2	$\chi^2_{0.05,\,21}$
	a_0	−8.694	−4.77					
21	a_1	0.9648	3.93	2.08	4.06	28.01	20.45	32.67
	a_2	0.9722	5.09					

TABLE 6 EB Safety Estimates

Segment	Model Predicted Accidents	Observed Injury Accidents	EB Estimate
Ex SS 7 dir/c "Appia"$_{\text{(from km 12.6 to km 24.2)}}$	8.43	4	5.44
Ex SS 88$_a$ "Dei due Principati"$_{\text{(from km 15.6 to km 32.0)}}$	18.46	7	9.07
Ex SS 88$_b$ "Dei due Principati"$_{\text{(from km 36.0 to km 56.4)}}$	28.32	34	33.29
Ex SS 91$_a$ "Della Valle del Sele"$_{\text{(from km 0 to km 31.2)}}$	13.06	13	13.01
Ex SS 91$_b$ "Della Valle del Sele"$_{\text{(from km 31.2 to km 44.4)}}$	4.13	0	2.05
Ex SS 91$_c$ "Della Valle del Sele"$_{\text{(from km 44.4 to km 58)}}$	2.16	1	1.76
Ex SS 91 bis "Irpinia"	2.05	3	2.37
Ex SS 164$_a$ "Delle Croci di Acerno"$_{\text{(from km 34.2 to km 53.4)}}$	5.41	2	3.46
Ex SS 164$_b$ "Delle Croci di Acerno"$_{\text{(from km 53.4 to km 76.2)}}$	5.00	8	6.66
Ex SS 165 "Di Materdomini"	1.09	1	1.07
Ex SS 303$_a$ "Del Formicoso"$_{\text{(from km 20.2 to km 41.0)}}$	13.80	12	12.41
Ex SS 303$_b$ "Del Formicoso"$_{\text{(from km 41.0 to km 59.0)}}$	3.46	7	5.09
Ex SS 368 "Del Lago Laceno"	8.24	1	3.39
Ex SS 371 "Della Valle del Sabato"	5.97	7	6.58
Ex SS 374 "Di Summonte"$_{\text{(from km 0 to km 20.0)}}$	9.63	18	15.52
Ex SS 374 dir "Di Montevergine"	0.92	0	0.75
Ex SS 399 "Di Calitri"	12.25	8	9.06
Ex SS 400 "Di Catelvetere"	23.94	31	29.98
Ex SS 400 dir "Di Catelvetere"	5.22	6	5.66
Ex SS 403 "Della Valle di Lauro"$_{\text{(from km 3.0 to km 9.8)}}$	6.23	9	7.91
Ex SS 414 "Di Montecalvo Irpino"	7.17	15	12.17
Ex SS 428 "Di Villa Maina"	3.88	7	5.40
Ex SS 574 "Del Monte Terminio"	11.07	8	8.82
Ex SS 574 dir "Del Monte Terminio"	0.57	0	0.50

Ex SS indicates a regional road that was a national road.

The results from the correlation analysis ($\rho_s = 0.94$, $z = 4.52$) indicate that the ranking from the subjective PFI and the objective EB estimate do agree at the 99.9% level of significance. These results provide a valuable validation for the PFI. Indeed, studies on methodologies aimed at identifying the sites with promise (*27*), which are the sites where the greatest cost-effectiveness of the safety measures is expected, found that ranking criteria based on accident frequency gives the best results.

The ranking from the subjective PFI was compared with the ranking from the analytical PFI, which is calculated as the difference between the accident EB estimate and the model prediction (*28*). The results from the correlation analysis ($\rho_s = 0.30$, $z = 1.46$) indicate that the ranking from the subjective PFI and the analytical PFI agree at the 92.7% level of significance. The correlation, albeit significant, is not as strong as the correlation between the subjective PFI and EB the estimate. This has two main reasons:

- The accident prediction model used for estimates does not take into account road feature explanatory variables other than segment length.
- Segments with low traffic volume have analytical PFI greater than segments with high traffic volume that experienced fewer accidents than predicted. However, these high-traffic-volume segments may have high potential for improvement because of factors not included in the accident predictive model.

Both PFI and accident frequency are dependent on traffic volume, but this is not the main explanation of their correlation. The relative risk, which depends only on the identified safety issues, is robustly correlated with the accident rate. The hypothesis of correlation was tested by assessing Spearman's rank-correlation coefficient for two criteria: descending order of accident rate (EB estimate of accident frequency/10^8 veh × km) and descending order of relative risk. The rankings from the two criteria agree at the 99.9% level of signifi cance ($\rho_s = 0.63$, $z = 3.02$).

CONCLUSIONS

A systematic process to determine which road features should be investigated and how each feature should be evaluated during the safety review was proposed. The approach is based on known accident relationships, and as a final result a PFI was computed. PFI quantifie the safety gains that could be achieved by addressing the problems identified in the review process.

Validation of the procedure was carried out by comparing the PFI values with the expected collision frequency. PFI was assessed in 406 km of rural two-lane rolling highways in Italy. Collision frequency was estimated by applying a collision prediction model, calibrated in the study network, and was refined by applying the EB technique. Correlation between EB safety estimates and PFI values is highly significant, with 93% of the variation in the estimated number of accidents explained by the PFI value. The level of agreement between the results of the EB estimates and the PFI was evaluated also by the Spearman's rank-correlation coefficient. Sites were ranked according to both the EB estimate and the PFI, with the results of

TABLE 7 Comparison of PFI and EB Ranks

Segment	PFI	PFI Rank	EB Estimate [acc./(km×year)]	EB Rank	Rank Difference
Ex SS 7 dir/c "Appia" $_{\text{(from km 12.6 to km 24.2)}}$	3.18	11	0.59	10	1
Ex SS 88_a "Dei due Principati" $_{\text{(from km 15.6 to km 32.0)}}$	5.42	5	0.69	9	−4
Ex SS 88_b "Dei due Principati" $_{\text{(from km 36.0 to km 56.4)}}$	13.01	2	2.04	2	0
Ex SS 91_a "Della Valle del Sele" $_{\text{(from km 0 to km 31.2)}}$	2.78	13	0.52	12	1
Ex SS 91_b "Della Valle del Sele" $_{\text{(from km 31.2 to km 44.4)}}$	1.09	20	0.19	20	0
Ex SS 91_c "Della Valle del Sele" $_{\text{(from km 44.4 to km 58)}}$	0.83	22	0.16	22	0
Ex SS 91 bis "Irpinia"	2.52	14	0.36	15	−1
Ex SS 164_a "Delle Croci di Acerno" $_{\text{(from km 34.2 to km 53.4)}}$	2.20	15	0.23	18	−3
Ex SS 164_b "Delle Croci di Acerno" $_{\text{(from km 53.4 to km 76.2)}}$	1.84	17	0.36	14	3
Ex SS 165 "Di Materdomini"	0.30	24	0.09	23	1
Ex SS 303_a "Del Formicoso" $_{\text{(from km 20.2 to km 41.0)}}$	4.69	6	0.75	8	−2
Ex SS 303_b "Del Formicoso" $_{\text{(from km 41.0 to km 59.0)}}$	1.34	19	0.35	16	3
Ex SS 368 "Del Lago Laceno"	2.82	12	0.22	19	−7
Ex SS 371 "Della Valle del Sabato"	4.14	7	0.76	7	0
Ex SS 374 "Di Summonte" $_{\text{(from km 0 to km 20.0)}}$	3.58	9	0.97	5	4
Ex SS 374 dir "Di Montevergine"	0.77	23	0.09	24	−1
Ex SS 399 "Di Calitri"	3.40	10	0.57	11	−1
Ex SS 400 "Di Catelvetere"	7.14	3	1.27	4	−1
Ex SS 400 dir "Di Catelvetere"	14.54	1	2.08	1	0
Ex SS 403 "Della Valle di Lauro" $_{\text{(from km 3.0 to km 9.8)}}$	7.02	4	1.45	3	1
Ex SS 414 "Di Montecalvo Irpino"	4.04	8	0.82	6	2
Ex SS 428 "Di Villa Maina"	2.12	16	0.45	13	3
Ex SS 574 "Del Monte Terminio"	1.83	18	0.29	17	1
Ex SS 574 dir "Del Monte Terminio"	0.95	21	0.17	21	0

Ex SS indicates a regional road that was a national road.
$\rho_s = 0.94$; $z = 4.52$.

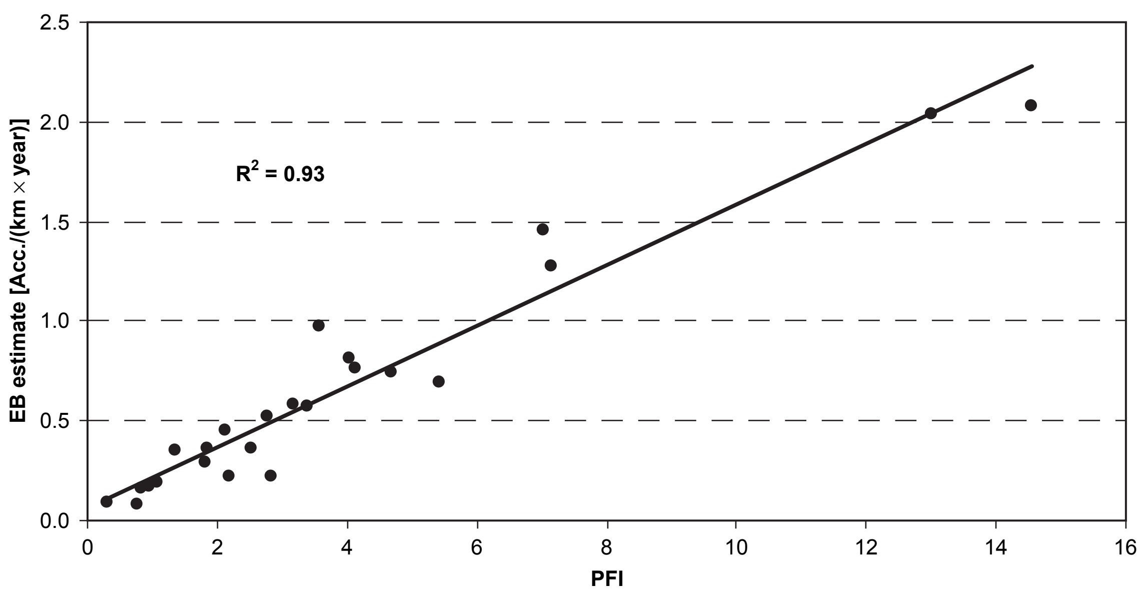

FIGURE 1 Correlation between EB accidents estimate and PFI.

the Spearman correlation indicating agreement at a 99.9% significanc level. This means that ranking of segments that hold promise for accident reduction gives comparable results in terms of PFI or accident history.

PFI can be assessed whether accident data are available or not. If accident data are available and their quality is good, PFI can be effectively used in conjunction with accident frequency as ranking criteria, improving the ranking made by using accident frequencies alone. Indeed, segments with similar accident frequency may give rise to different potential benefits of the safety measures. The PFI index quantitatively assesses these potential benefits. If accident data are not available or are poor, PFI can be used as a proxy of accident data and becomes the only ranking criteria.

The PFI has two main practical applications. High-risk segments, where safety measures that can reduce accident frequency and/or severity do exist, can be identified and ranked by the global PFI. Specific safety issues that contribute the most to safety problems are pointed out to give guidance about more appropriate mass action programs. Relative risk ranks different types of safety measures in each segment, whereas PFI of single safety issues ranks the segments in relation to a specific safety improvement program.

PFI can be assessed as part of the safety review process without relevant supplementary work. Safety reviews represent a low-cost process for the periodic evaluation of the network safety performance, and the PFI assessment is an effective tool for the development of safety strategies incorporating the reviews in a more comprehensive road safety program. The low cost and applicability in road networks where geometric and accident data are lacking make the procedure very attractive for low-volume roads.

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Programming Safety Improvements on Pavement Resurfacing, Restoration, and Rehabilitation Projects

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As part of the project planning process, highway agencies must allocate limited funding to a substantial list of projects that exceeds available resources. For preservation projects, a key component of this decision is to determine which projects receive safety improvements and which are "pave only." Traditionally, this decision has been made project by project, with the possible result of a selection that does not maximize safety benefits. This paper takes a case study approach and applies a new tool developed in *NCHRP Report 486,* the Resurfacing Safety Resource Allocation Program (RSRAP), to a subset of the Oregon Department of Transportation's (DOT's) highway network. The RSRAP tool maximizes safety improvements for a given set of projects and budget. Thirty-three projects scheduled to receive a new road surface were selected and analyzed with RSRAP. These projects were subdivided into smaller sites to meet the assumptions of RSRAP. Road geometry, traffic volumes, and crash history for each site were collected and input into the program. The type and cost of the safety improvements output by RSRAP were compared with those selected by Oregon DOT. This research determined that RSRAP, which selected more projects for safety improvements than did Oregon DOT, is a tool that could be used by the department to select various safety improvements on pavement preservation projects. It was also determined that the budget used by Oregon DOT was large enough that all cost-effective improvements could be made.

Highways across the country are a vital part of the transportation network and people's daily lives. There is a continual need to perform maintenance, rehabilitation, and, in some cases, replacement of existing roads. State agencies have thousands of miles of roadway that need to be maintained or repaired, and only a portion of these miles can be resurfaced each year. In spending limited public dollars, agencies want funds to be spent effectively and to return the most benefit to the public. Many state agencies use a pavement management system to select roads for resurfacing. The federal 3R (resurfacing, restoration, and rehabilitation) program was developed to help provide funding for preservation projects selected by state agencies. 3R projects include pavement resurfacing, lane and shoulder widening, changes to curve alignment, and removal of roadside obstructions. 3R projects must upgrade certain design features to minimum standards but typically do not include significant safety improvements (*1*).

The Oregon Department of Transportation (DOT) and many of its counterparts around the country have separate funding for different types of projects. 3R projects are usually funded by preservation funds, whereas safety, modernization, and bridge projects have their own funding categories. Further, other institutional incentives, for miles resurfaced each year, in effect discourage addressing safety features if they significantly detract from available preservation dollars. Oregon DOT has developed a policy, however, that would allow some funding from the safety category to be spent on safety features on 3R work. For most pavement preservation projects on rural highways, safety performance generally is not considered in the project selection process; rather, it is considered after the projects have been selected to maintain pavement quality. This method may not select the optimal blend of pavement and safety improvements that yield the most benefit. An alternative approach, considered in this research, is to select safety improvements to maximize the benefits of crash reductions across all preservation projects in the program, rather than consider each project individually.

Until recently, no software had been available that optimally distributes funds to safety improvements on various urban and rural highways. The Resurfacing Safety Resource Allocation Program (RSRAP), which was developed in 2003 in *NCHRP Report 486,* can now estimate the benefits of various combinations of improvement alternatives for multiple sites and allocate the funding in a way that maximizes the total benef t while keeping the costs under a specified budget (*2*). The main objective for this research is to compare the current methods used by Oregon DOT to select safety improvements on 3R projects versus an optimization tool developed. A secondary objective was to determine the feasibility of RSRAP software for selecting safety improvements in the Oregon DOT pavement preservation project selection.

METHODOLOGY

A case study of projects programmed for construction in the 2004 to 2007 Statewide Transportation Improvement Program (STIP) was used to investigate the two objectives. To limit data collection efforts and analysis, a subset of the statewide projects was used. The Oregon DOT divides the state into five regions; projects within Region 2, which consists of Clatsop, Tillamook, Yamhill, Polk, Marion, Linn, Lincoln, Benton, and Lane counties, were used. The regions are shown

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in Figure 1. The primary tool used in this study was RSRAP, developed by NCHRP to select sites for improvements by evaluating both safety and operational effects. In its full application, the RSRAP software determines which sites should be resurfaced and improved and which sites should simply be improved. This research limited the use of the RSRAP software to determine which sites should be improved since the projects to be resurfaced were already selected (as part of the STIP process). The RSRAP software requires that a significant amount of data be collected for each project. At a minimum, the following information must be collected:

- Route number,
- Site description,
- Site length,
- Number of lanes,
- Lane width,
- Roadway type,
- Area type,
- Average daily traffic,
- Crash data,
- Average travel speed,
- Shoulder type, and
- Shoulder width.

If roadside, curve, or turn lane improvements are considered, additional data must be collected. However, the required data should be available to most state agencies. Historical crash data are an important input used in the calculation of safety benefits. Costs for each improvement were also a required input. The analysis, described in the following, includes nearly 250 mi of roadway. The data collection and analysis process took approximately 2 months to perform.

As used in this study, the RSRAP software selects a set of safety improvements for the identified projects that maximize the safety benefits of the improvements given the available budget. RSRAP can also include a benefit for time travel improvements and factor in an increased crash frequency for a short time following resurfacing. These options were not used in this study. The RSRAP software has seven defined alternatives: pavement resurfacing, lane widening, shoulder widening, shoulder paving, horizontal curve improvements, roadside improvements, and intersection turn lane improvements. Users can define their own alternatives, if needed. The average number of crashes was calculated for each section by using the five most recent years of available crash data and was used in the RSRAP analysis. Construction costs are calculated by using user-defined costs or default values in RSRAP. The default values are based on average cost data from various highway agencies. These values can be changed to values that are more representative of costs experienced by the particular highway agency. The safety benefits are calculated by applying an accident modification factor to the average annual number of crashes. The AMF default values were used in this study, but site-specific factors can be used. The cost savings per crash values are taken from the estimates published by FHWA in 1994 and updated in 2002 (*3*).

The net benefits are calculated by summing all the present values of the benefits and subtracting the costs of construction. A single alternative is selected for each site by using integer programming. A single alternative can be a combination of several different alternatives. During the optimization process, a list of possible alternatives is developed. This list is then reduced by eliminating alternatives that are "dominated" by other alternatives. An alternative is dominated by another if it has a larger cost and smaller benefits. The remaining alternatives are input into the solver application in Microsoft Excel for optimization. The optimum solution is the group of alternatives that provides the maximum total benefit given the constraints. The con-

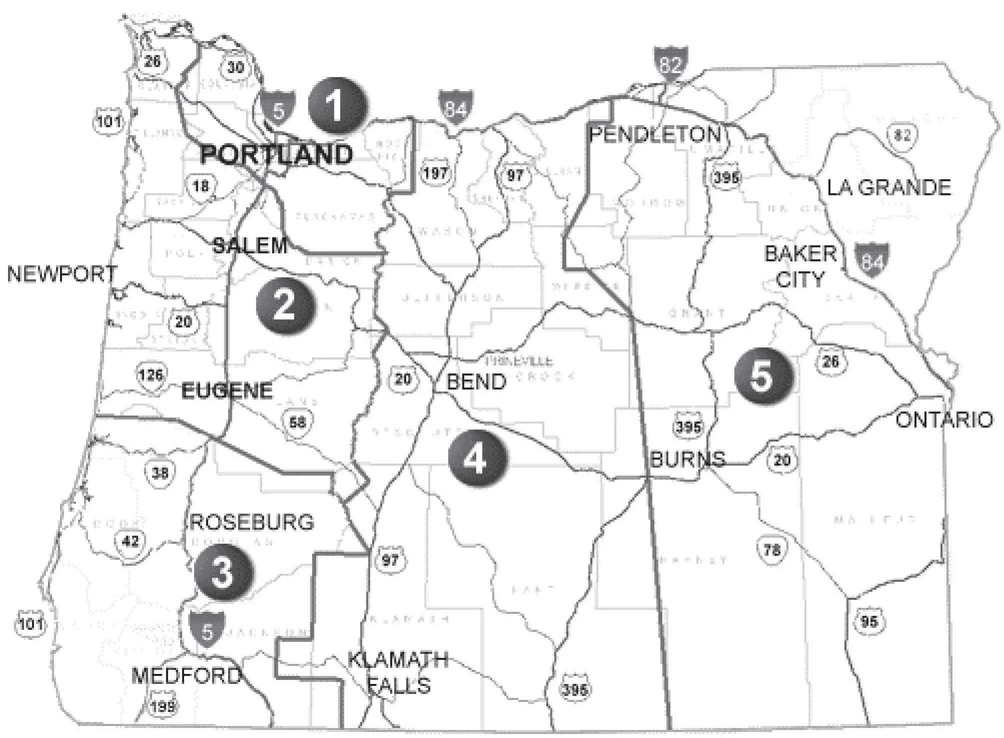

FIGURE 1 Oregon DOT regions.

straints are that only one alternative per site can be selected and that the costs must not exceed the budget.

Division of Projects into Similar Sections

A total of 33 projects from the 2004 to 2007 STIP from Region 2 were selected for this case study. RSRAP requires that the cross sections be consistent throughout the site. Roadway sections should be broken into smaller subsections if the characteristics vary within a site. For this reason, the original 33 projects in Region 2 were broken into 101 smaller sites. Projects were subdivided into homogenous sections primarily by the number and width of lanes, shoulder characteristics, and, to a lesser extent, average daily traffic, average travel speed, area type, and roadway type. For each site, the RSRAP user must identify all feasible safety improvements. Feasibility of the six default improvements to be considered in the optimization was determined by using the following considerations:

- Lane widening was considered at increments of 1 ft up to a total width of 12 ft. Of the 101 sites studied, all but three already had lane widths of 12 ft or greater. These three sites are considered for lane widening.
- Shoulder widening was considered at increments of 2 ft up to a total width of 8 ft. Sites that already had a shoulder width of 8 ft or greater were not considered for shoulder widening.
- All 101 sites have paved shoulders, so this option was not considered.

Each of the next three alternatives used a more detailed process that would be specific to Oregon DOT. The process described was used to determine which sites should be considered for roadside, curve, or turn lane improvements.

Roadside Improvements

Roadside improvements remove obstructions or flatten fore or back slopes outside the traveled way. The RSRAP software classifie roadside improvements on the basis of the roadside hazard rating (RHR). Oregon DOT does not compile or collect roadside inventory data, so RHRs were determined from visual observations of the state's digital video log. To limit data collection efforts, roadside improvements were considered only for roadways that had a greater than expected number of crashes. Oregon DOT maintains a high-crash-location list and statewide crash rate tables but not average crash frequencies for each road type or category. The Interactive Highway Safety Design Model (IHSDM) crash prediction model for rural highways was used to determine an expected number of crashes per year per mile by using the default model (*4*). The model was not calibrated to Oregon data. This expected number of crashes was then used as a reference and compared to the actual number of crashes per year per mile for each of the rural segments. For roadside improvements, if the number of crashes with a fixed object or the number of crashes occurring off the roadway for a segment was greater than the number of crashes expected, the roadway would be considered for roadside improvements. For more detailed use, the model should be calibrated, but in this case it was used only to estimate whether the section should be considered for roadside improvements. Further, the Washington State base model should compare favorably to Oregon roadways. The average number of crashes per year was calculated by using data from 1998 to 2002 and was used to calculate the safety benefits for each section of roadway. The road was split into both travel directions, and each side of the road was classified. An RHR of 1 is representative of an open shoulder with a wide clear zone, whereas a high RHR 7 is representative of a steep shoulder with obstructions close to the edge line of the roadway. Sections that were determined to have an RHR of 3 to 4 were considered for improvement to RHR 2.5, and sections with an RHR of 5 to 6 were considered for improvement to RHR 3 to 4 by adding guardrail. RHRs of 7 were not considered for improvement because of cost. A flowchart of this process is shown in Figure 2.

Horizontal Curve Improvements

Horizontal curve improvements are seldom performed on preservation projects because they are costly. The process for determining if a rural site should be considered for horizontal curve improvements again used the IHSDM in the initial steps. Figure 3 shows the process that was followed in selecting horizontal curve improvements. Urban sites were not considered for curve improvements. The IHSDM model was evaluated for basic conditions, which means that the crashes per year per mile were calculated for straight sections. The number of curve crashes for each site was then compared to the IHSDM expected crashes, and if the curve crashes were larger, the site could be considered for horizontal curve improvements. Once a site was flagge for horizontal curve improvements, the curve crashes were examined

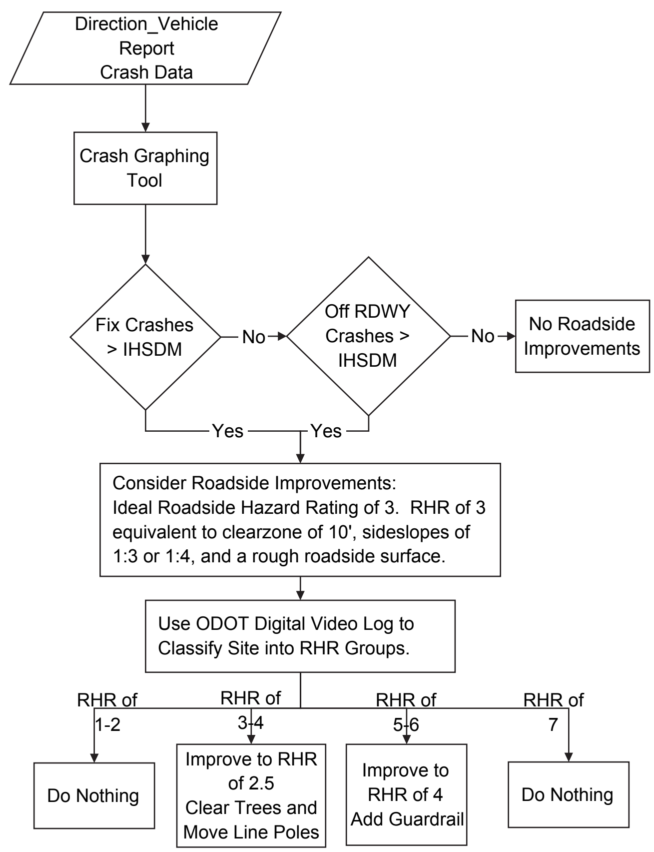

FIGURE 2 Roadside improvement selection (RDWY = roadway).

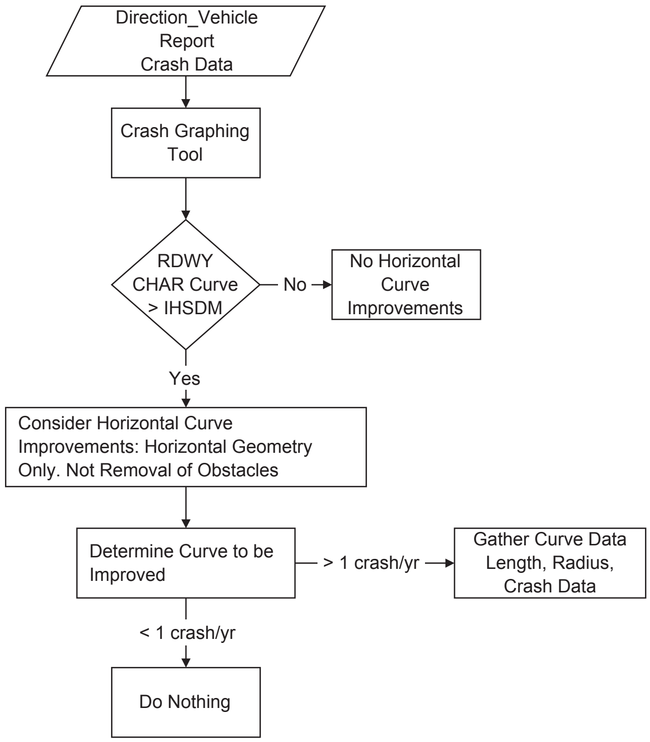

FIGURE 3 Curve improvement selection process (RDWY CHAR = roadway character).

by milepost and broken down by curve. If a curve had four or fewer crashes in a 5-year period, it was not considered for curve improvements.

Intersection Turn Lane Improvements

Turn lane improvements were considered on rural sections only. Figure 4 shows the process that was followed for selecting turn lane improvements on rural sections. The IHSDM intersection crash prediction models were used to identify rural sites with high numbers of intersection crashes. The average number of crashes per year occurring within the intersection was compared to the expected number of crashes from the IHSDM. Each of the sites eligible for improvements contained several intersections and was examined by using the digital video log. On further examination, most of the urban intersections already had existing turn lanes, and additional turn lanes were not added.

Cost Data

Cost data are a crucial input because RSRAP is designed to determine if an alternative or set of alternatives is cost-effective. RSRAP provides default cost data for five different improvements: pavement resurfacing, lane widening, shoulder widening, and the installation of left and right turn lanes. These default values are based on average cost data from various highway agencies across the country. These values include the cost of building the subgrade and the cost of the road surface and do not include the cost of obtaining additional right-of-way. If additional right-of-way is required, then the costs for widening the

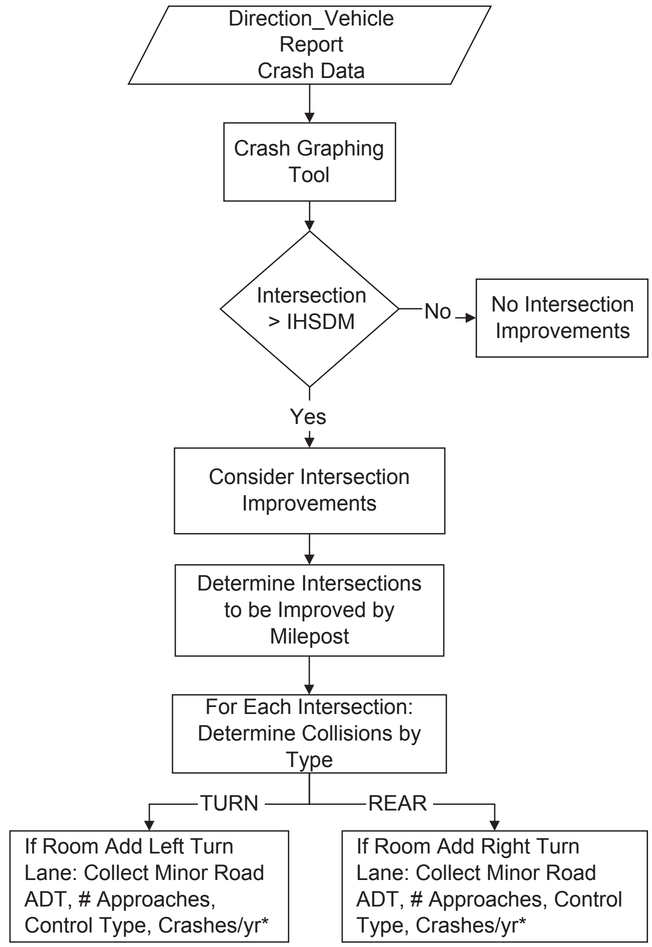

FIGURE 4 Turn lane improvement selection. (*If a right or left turn lane already exists, consider user-defined alternatives such as alternative signal timing plans or adding a signal. Two-way left turn lanes are treated as left turn lanes; ADT = annual daily traffic.)

shoulder are increased. The default values were used for cost values except on shoulder widening improvements. There were a larger number of sites with shoulder widening improvements, and, therefore, a low and a high cost were determined.

Roadside, horizontal curve, and user-defined alternatives do not have default values in the program because of the great variation in costs from site to site. No user-defined alternatives were considered for this project. Roadside costs were calculated on the basis of the improvements being considered. For improvements made to an RHR of 3 to 4, it was assumed that this would include removing any trees and relocating line poles that were too close to the roadway. Cost calculations are based on *Heavy Construction Cost Data* (*5*). Several sites were used to determine the estimated number of trees and line poles per mile. These estimates were then multiplied by the per-tree or per-pole costs to arrive at a cost of $21,000 per mile for improvements to RHR 3 to 4 groups. Areas with an RHR of 5 to 6 would have guardrails installed for the improvement. The cost of adding guardrail was determined to be approximately $55 per foot. The total cost of roadside improvements was then determined by adding the costs of the RHR 3 to 4 improvements and the RHR 5 to 6 improvements. Horizontal curve improvements often have a very high cost. This

TABLE 1 Improvement Costs Used in Case Study

Improvement	Cost
Lane resurfacing*	
Rural	$1.07 per ft^2
Urban	$1.80 per ft^2
Shoulder resurfacing*	$0.47 per ft^2
Lane widening*	$3.93 per ft^2
Shoulder widening**	
High	$5.15 per ft^2
Low	$4.30 per ft^2
Turn lanes (left/right)*	
Rural	$60,000 per intersection
Urban	$112,000 per intersection
Roadside**	
3–4	$21,335 per mile
5–6	$55 per foot
Horizontal curve**	$3,000,000 per curve

*RSRAP default costs
**User-generated costs

can be because of the large amounts of earthwork, and in some cases the blasting of rock, that may be required to straighten the curve. It was estimated that a horizontal curve improvement would cost approximately $3 million per curve. Table 1 summarizes the costs that were used in the case study.

Apply RSRAP to Projects

After the site data have been entered for all projects and the improvement alternatives have been selected, the optimization process is performed. This study was concerned only with the allocation of funds to safety alternatives and therefore the "optimize safety improvements only" option was selected. Under this condition, traffic operational benefits and the penalty for resurfacing without geometric improvements were not included. The analysis was run for six budgets. As a matter of policy, region engineers usually attempt to program approximately 25% of the total available safety funding for safety improvements on preservation projects. This policy has not been followed consistently among the Oregon DOT regions. The total available budget for all safety improvement projects in Region 2 was $19.8 million (a total for the 4 years). This funding level was the maximum budget tested. Five additional budget levels were tested ($10 million, $5 million, $3 million, $ 1.5 million, and $0.75 million, respectively).

RESULTS

The RSRAP selects cost-effective improvements that maximize safety benefits without exceeding the available budget. As discussed in the previous section, the optimization was done on safety improvements only, by considering only safety benefits without a penalty for resurfacing without geometric improvements. Budgets of $10 million, $5 million, and $3 million all produced the same site selection with a total estimated cost of $2.4 million. Twelve sites were selected to receive improvements. Lower budgets, obviously, selected a different improvements list. The $1.5 million budget selected eight sites to receive safety improvements. The improvements selected for the remaining eight sites were unchanged from the first four budgets, but one project reduced the amount of shoulder widening from 6 ft to 4 ft. The total benefits, however, do not change as much as the total costs. For the four largest budgets, the total benefits are just over three times the dollar amount of the total costs. For the two smallest budgets, the benefits become five to seven times greater than the total costs. Further analysis revealed that turn lane additions in rural areas were particularly cost-effective. If the turn lane costs were more expensive, then the program might select other combinations of improvements that provide more benefit than the turn lanes. The results showed that the benefits for adding turn lanes when compared to the cost are much greater than other improvements that were selected. In addition, the estimated costs that were included in the analysis greatly influenc the analysis that is performed.

Comparison of the locations recommended for improvement by RSRAP to those selected by Oregon DOT's project selection process revealed that Oregon DOT selected fewer locations for improvement than the RSRAP program selected. However, the three projects that received funding by using the Oregon DOT method include 28 RSRAP sites. Not all these RSRAP sites received work from Oregon DOT funding, and 10 of those 28 sites are part of a single Oregon DOT project. It is reasonable that the project selection should be different since the Oregon DOT method does not consider all possible alternatives. Sites considered by Oregon DOT for safety funds are subject to a benefit-to-cost analysis and are often on the state's high-crash-location list.

The amount of money being spent by Oregon DOT was different from that allocated by the RSRAP program. Three projects in the 2004 to 2007 STIP include safety money in their costs. Some of the remaining 28 projects receive some safety improvements, such as those recommended by RSRAP, but do not receive safety funds. Typically, the Oregon DOT tries to spend 25% of its safety budget on preservation projects. This means that $4.96 million would be spent on safety improvements on preservation projects between 2004 and 2007. The $1.4 million scheduled to be spent by Oregon DOT is below the $2.4 million recommended by RSRAP for the Region 2 budget of $19.84 million. Figure 5 shows the money allocated by RSRAP and by Oregon DOT. Neither RSRAP nor Oregon DOT approached the goal of 25% of the overall safety budget. This suggests that, at least in the projects studied in this case study, spending 25% of the overall budget on additional safety features may not be feasible.

CONCLUSIONS

RSRAP could help state departments of transportation improve how they distribute safety funding to projects that receive a new road surface. The program uses road characteristics, crash data, and cost information to determine the optimal distribution of funds. This project examined how RSRAP could be applied within the state of Oregon and how those results compared to the selections made by Oregon DOT. The primary objective for this research was to examine how the allocation of safety funds by Oregon DOT compares to those of RSRAP. This research suggests that Oregon DOT spends less than the typical 25% of its safety funds in Region 2 on preservation projects by using current allocation methods. RSRAP allocated greater amounts of safety funding to these preservation projects than Oregon DOT but was also below the 25% value. As discussed, this result could be sensitive to the cost estimates. The RSRAP software could be a valuable tool that Oregon DOT could implement in the future. The

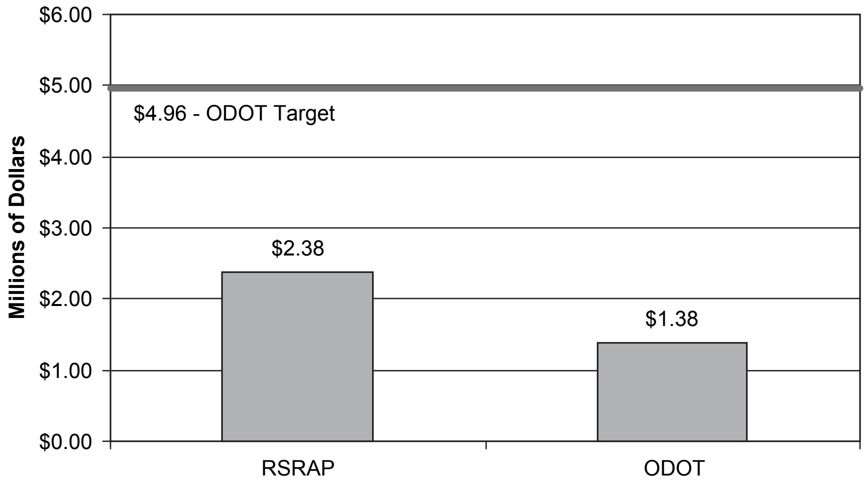

FIGURE 5 Comparison of cost of safety improvements selected.

current process of allocating safety funds on preservation projects is suboptimum because it is not a systemwide analysis. RSRAP would allocate a budget by using a process that could result in a near optimum allocation of funding by considering all projects within a region or state. Further research would help determine whether some of the problems encountered in this research could be overcome.

ACKNOWLEDGMENTS

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Integrating Safety into the Transportation Planning Process

Case Study in Hampton Roads, Virginia

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The Transportation Equity Act for the 21st Century requires metropolitan planning organizations (MPOs) to incorporate safety and security into the transportation planning process as one of the seven planning factors. The Hampton Roads Planning District Commission (HRPDC) is the designated MPO for Southeastern Virginia. In 2001, as part of its congestion management system (CMS) program, the HRPDC staff initiated a comprehensive regional safety study, which included collecting comprehensive crash data and creating a regional database for 151 Interstate segments and 13,000 intersections. The crash severity method was used to analyze, rank, and determine the top high-crash locations for Interstate segments as well as the CMS intersections. This regional safety study was designed to help local communities understand safety-related problems and issues. Congestion, failure to yield the right-of-way, following too closely, driver inattention, and disregarding traffic signals were found to be the main causes of traffic crashes in Hampton Roads between 1998 and 2000. Rear-ends and right angles were the predominant crash types during the period. The study analyzed and recommended a series of safety-related countermeasures and solutions for the top-10 high-crash locations throughout the region. Some common countermeasures that were recommended were adding roadway capacity, adding turn lanes at intersections, improving signal timing, improving signage, increasing enforcement, and providing additional driver education.

The Transportation Equity Act for the 21st Century (TEA-21) requires states and metropolitan planning organizations (MPOs) to incorporate safety and security into the transportation planning process as one of the seven planning factors. The Hampton Roads Planning District Commission (HRPDC) is the designated MPO for Southeastern Virginia, which comprises 13 jurisdictions, including the cities of Norfolk, Virginia Beach, and Newport News. In 2001, the HRPDC staff as part of its congestion management system (CMS) program initiated a comprehensive regional safety study, the first of its kind for the region. This paper is based on a project that was prepared in three parts by HRPDC (*1–3*).

Traffic crashes claim more than 40,000 lives in the nation every year (*1*). These losses include more than 900 people within the state of Virginia. On average, someone is killed in a crash in Hampton Roads every 2.7 days. A comprehensive roadway safety program is needed to reduce both the number and the impact of traffic crashes on Hampton Roads residents. A crucial element of this program is the collection and effective use of crash data to identify and correct safety deficiencies in the highway system. The regional safety study represents the first step in achieving such a program in Hampton Roads. This study is designed to help local communities understand roadway-related traffic safety issues and problems. In addition, results would provide local transportation engineers and decision makers with a useful tool for setting funding priorities for safety projects in Hampton Roads.

The HRPDC regional safety study is the first step of integrating safety into the transportation planning and programming process in Hampton Roads. The objectives for the study were to

- Collect and organize crash data for the CMS system network in the region,
- Develop and create a regional database with geographic information system (GIS) capabilities,
- Analyze and identify high-crash locations for Interstates and intersections,
- Develop countermeasures and examine strategies to address safety problems, and
- Recommend safety projects and secure funding for incorporation into the region's transportation improvement program (TIP).

LITERATURE REVIEW

In 2001, the HRPDC staff conducted a thorough search of safety-related reports, websites, and other safety planning studies performed by various MPOs to gather ideas for a regional study. It was found that few MPOs in the United States had initiated safety study efforts. In 1997, the Southeast Michigan Council of Governments (SEMCOG) completed the second edition of its traffic safety manual, which includes a comprehensive crash data analysis and safety tools and countermeasures to be used by local governments and decision makers for correcting safety deficiencies in the roadway system (*4*). In addition, SEMCOG's numerous safety studies and results are widely used to increase sensitivity of the public and media to safety issues. In 2001, the Maricopa Association of Governments (MAG) began identifying key regional transportation safety issues and needs and possible steps for addressing them through initiatives at state, regional, and local levels. MAG formed the Transportation Safety Stakeholders Group in November 2001 to guide this effort (*5*). The New Jersey Transportation Planning Authority recently initiated development of a regional safety priorities project to identify transportation safety needs and solutions in northern and central New Jersey (*6*).

Hampton Roads Planning District Commission, 723 Woodlake Drive, Chesapeake, VA 23320.

Transportation Research Record: Journal of the Transportation Research Board, No. 1922, Transportation Research Board of the National Academies, Washington, D.C., 2005, pp. 79–89.

DATA COLLECTION AND DATABASE DEVELOPMENT

The first step of the data collection process was to determine what motor vehicle crash information was available and how to obtain it. It was discovered that the Virginia Department of Motor Vehicles (DMV) annually releases the *Virginia Traffic Crash Facts* report, which provides a comprehensive view of traffic crashes statewide. The Virginia Department of Transportation (VDOT) and the Virginia DMV maintain a database for all crashes statewide that occur on public roadways and involve at least one injury or fatality or property damage of at least $1,000. The database includes detailed crash information; however, specific crash locations are provided only for state-maintained roadways (which include Interstates and all roadways outside of city boundaries). Each city within Hampton Roads was contacted individually to obtain its crash data. Departments that maintain crash databases vary among cities from local police departments to traffic engineering. Crash data were obtained from the following departments:

- VDOT, Richmond central office (mobility management division);
- City of Chesapeake Department of Public Works;
- City of Hampton, traffic engineering department;
- City of Poquoson, traffic engineering department;
- City of Portsmouth, traffic engineering department;
- City of Suffolk police department (crime analysis);
- City of Newport News, engineering department, transportation services division;
- City of Norfolk, traffic engineering;
- City of Virginia Beach, public works engineering (traffic); and
- City of Williamsburg police department.

Creation of Regional Crash Databases

The main objective was to collect and organize crash data from each jurisdiction in Hampton Roads for January 1998 to December 2000 to be included in a comprehensive regional crash database, the firs of its kind for the region. This was a complex task of assembling data that varied greatly in quality and quantity for each jurisdiction. Many jurisdictions have unique software and database programs for their crash data, which are usually in different formats. Other databases are hand keyed and often are incomplete and have misspelled crash entries. The goal was to obtain all available crash information for each jurisdiction to make regionwide comparisons. Two separate regional crash databases were created for this study: Interstates and at-grade intersections. The Interstate crash database was created entirely from data obtained from VDOT. The intersection crash database was created by using a combination of data from VDOT for counties in Hampton Roads and individual crash databases from cities.

Regional Interstate Crash Database

The regional Interstate crash database was simple to create because all the necessary data were complete (from VDOT) and in the same format. All Interstates in Hampton Roads (approximately 130 centerline miles) were included. The Interstate crash data included locations for each crash by direction and mile marker (e.g., I-64 East at mile marker 289.45). From these data, crashes were assigned to Interstate segments by direction. These segments were broken at each full interchange, regardless of the distance between each interchange. The database contains number of crashes, injuries, and fatalities; major factor; time of day; driver action before crash; information for GIS coding; segment lengths; average daily traffic (ADT); daily vehicle miles of travel (VMT); crash rates; injury rates; equivalent property-damage-only (EPDO) crash rates; and other related calculations.

Regional Intersection Crash Database

For the purpose of this study, a crash must have occurred within 250 ft of an intersection to be included in the regional intersection database. All remaining crash data were excluded. Crash data were collected for approximately 13,000 intersections in Hampton Roads. Approximately 1,300 intersections were selected from the 13,000 to be included in the regional intersection crash database, on the basis that at least two roadway legs at the intersection are included in Hampton Roads CMS and at least one other intersecting roadway leg was a collector or higher roadway class. Furthermore, any intersection included in the regional database for which traffic count data were available and at least three or more legs were CMS roadways was identified as a CMS intersection; approximately 500 met this criterion. The Hampton Roads CMS includes a comprehensive regional roadway network consisting of all Interstates, expressways, principal and minor arterials, and selected collectors. CMS intersections include only at-grade intersections, where two or more CMS roadways intersect at the same elevation. Interchanges were not included since the intersecting roadways do not have conflict points at the same elevation. Ramps at interchanges also create unique conflict points and have traffic volumes that are quite different from at-grade intersections and therefore are not easily comparable. The CMS intersections were used to further analyze the crash data by incorporating the number of vehicles entering the intersection to compute crash rates. Only CMS intersections were analyzed with rates per amount of travel because of the availability of traffic count data. The database contains number of crashes, injuries, and fatalities; collision type; time of day; driver action; information for GIS coding; vehicles entering the intersection (CMS intersections only); and crash rates, EPDO crash rates, and other calculations.

Compiling crash data for the counties was a relatively simple task as the data for each crash (from VDOT) included all the information found in a Virginia DMV police crash report. A six-digit intersection node number was provided for each crash location, which was matched to intersection names by using VDOT's roadway information system. Once intersections were identified and summarized, the data were transferred to the regional intersection crash database.

Assembling crash data from each city in Hampton Roads proved to be a more difficult task. Most crash databases from each city did not include all the items found in the police crash reports. In particular, crash databases for the cities of Williamsburg and Poquoson did not contain a distance from the cross street for each crash. Without this information, crashes that occurred beyond 250 ft of the intersection could not be excluded. Thus, the intersection data for these two cities may contain more crashes than occurred at each intersection.

One of the challenging aspects of this study was formatting each individual city's crash database. Much of the data contained inaccuracies, such as misspelled roadway names or varying abbreviations (i.e., parkway, pkwy, pky, pwy), which had to be corrected so that crashes that occurred at the same location could be summarized. Also, many intersections contain legs with more than two roadway names. A typical intersection contains four legs with two roadway names. Some intersections had roadway legs with different names on opposite sides of the intersection. In this case, the roadway names had to be changed throughout the entire database so that intersection names were uniform and contained only two roadway names per intersection. Crash data for intersections on city lines also had to be combined. For example, crashes that occurred at the intersection of Newtown Road and Virginia Beach Boulevard may have been in either Norfolk or Virginia Beach's crash database. In some cases, the same crash was recorded in both databases, which required adjustment before summarizing of the intersection. An intersection node number assignment, similar to the county data, would help ease this problem.

VDOT's database contained crash data within each city, with the exception of the crash location. For the city databases that did not contain all vital information, each individual crash had to be searched for in the VDOT crash database by using characteristics found in both databases, matched by date and other similar data, and incorporated.

Another problem occurs when crash database software is changed or crash databases are not regularly updated. Crash databases for each city contained 3 years of data (1998–2000), except for the cities of Chesapeake, Hampton, Portsmouth, and Suffolk. For these cities, only 2 years of data were available. Crash data for the city of Hampton were available for all 3 years; however, the crash database software for the city changed during the 3-year period, which produced some questionable results.

One important observation during this process was a lack of standard or uniform format among the jurisdictions' crash databases. A centralized crash database containing both city and county data would have eliminated many of the problems that were encountered. To perform a true regional analysis and comparison of traffic crashes, crash databases for the entire region must be uniform and updated on a regular basis.

DATA ANALYSIS AND RESULTS

The creation of a regional database was instrumental in analyzing traffic-related crashes and depicting high-crash locations in the region. Crash data for the most recent 1- to 3-year period are usually used, and the analysis was based on crash data collected for the 1998–2000 period.

Analysis Methodology

Two separate data analyses were conducted for this study: Interstates and at-grade intersections. Several methodologies were examined for this analysis, including crash frequency, crash rate, and crash severity methods. For the purpose of this study, the crash severity method was selected for identifying high-crash locations for all Interstate segments and at-grade CMS intersections. The crash severity method was the preferred method for determining high-crash locations in this study since it compares the number of crashes to the amount of traffic along a segment of the interstate or at an intersection. This method uses EPDO crash rates to address high-crash locations with more serious hazards. For a given location, the number of crashes at each severity level is multiplied by an arbitrary weighting factor to transform crash frequency into an equivalent frequency of EPDO crashes. The weighting factors used in this study were based on numbers used in different localities in this and other regions. EPDO crash rates were then calculated by incorporating the amount of traffic at a specific segment or intersection.

All regional Interstates were analyzed by the crash severity method (EPDO crash rate) for each segment (interchange to interchange) by direction. Approximately 500 CMS intersections from the regional intersection crash database were analyzed with the crash severity method (EPDO crash rate) since traffic count data were not available for all intersections. An additional crash frequency analysis (crashes per year) was performed for all intersections in Hampton Roads (approximately 13,000 regionwide) for informational purposes. Crash severity method:

$$\begin{aligned}\text{yearly EPDO} = {} & 12 \times (\text{fatality crashes per year}) \\ & + 3 \times (\text{injury crashes per year}) \\ & + \text{PDO crashes per year}\end{aligned}$$

Interstate segment:

$$\text{EPDO crash rate} = \frac{1,000,000 \times \text{yearly EPDO}}{365 \times \text{ADT} \times \text{segment length}}$$

Intersection:

$$\text{EPDO crash rate} = \frac{1,000,000 \times \text{yearly EPDO}}{365 \times \text{daily vehicles entering intersection}}$$

Daily vehicles entering intersection were estimated by summing the ADT volumes for all legs of the intersection and dividing by two.

Primary Analysis Factors

Several crash data characteristics were selected and included in the regional analysis:

- Number of crashes,
- Number of injuries,
- Number of fatalities,
- Driving under influence
- Driver vision,
- Time of day,
- Day of the week,
- Weather,
- Number of vehicles per crash,
- Crash type,
- Major factors, and
- Most prevalent driver action.

Analysis Results

Similar to the creation of the regional database, the crash data analysis was conducted in two separate parts: Interstate segments and intersections.

Interstate Segments

There were an average of approximately 3,900 crashes per year on the Interstate system in Hampton Roads, with 1,286 injuries and 16 fatalities per year resulting from these crashes. This accounts for nearly 13% of all traffic crashes, 10% of all injuries, and 13% of all fatalities in Hampton Roads. By comparison, the Interstate system in Hampton Roads carries more than 27% of all regional VMT.

The study further analyzed the top-10 high-crash Interstate segment locations to determine the causes of crashes. Appropriate remedies and countermeasures to improve safety-related traffic problems were recommended for each of the top-10 high-crash locations. Figure 1 shows the top-10 high-crash locations for the Interstate system in Hampton Roads. To facilitate the analysis, the study included Interstate segment roadway geometry diagrams, collision diagrams, and crash data summaries. These details and summaries allowed observations to be made about existing crash problems, and samples are shown in Figures 2 through 5.

A total of 151 Interstate segments were ranked by descending EPDO crash rates, in which crash frequencies are weighted by crash

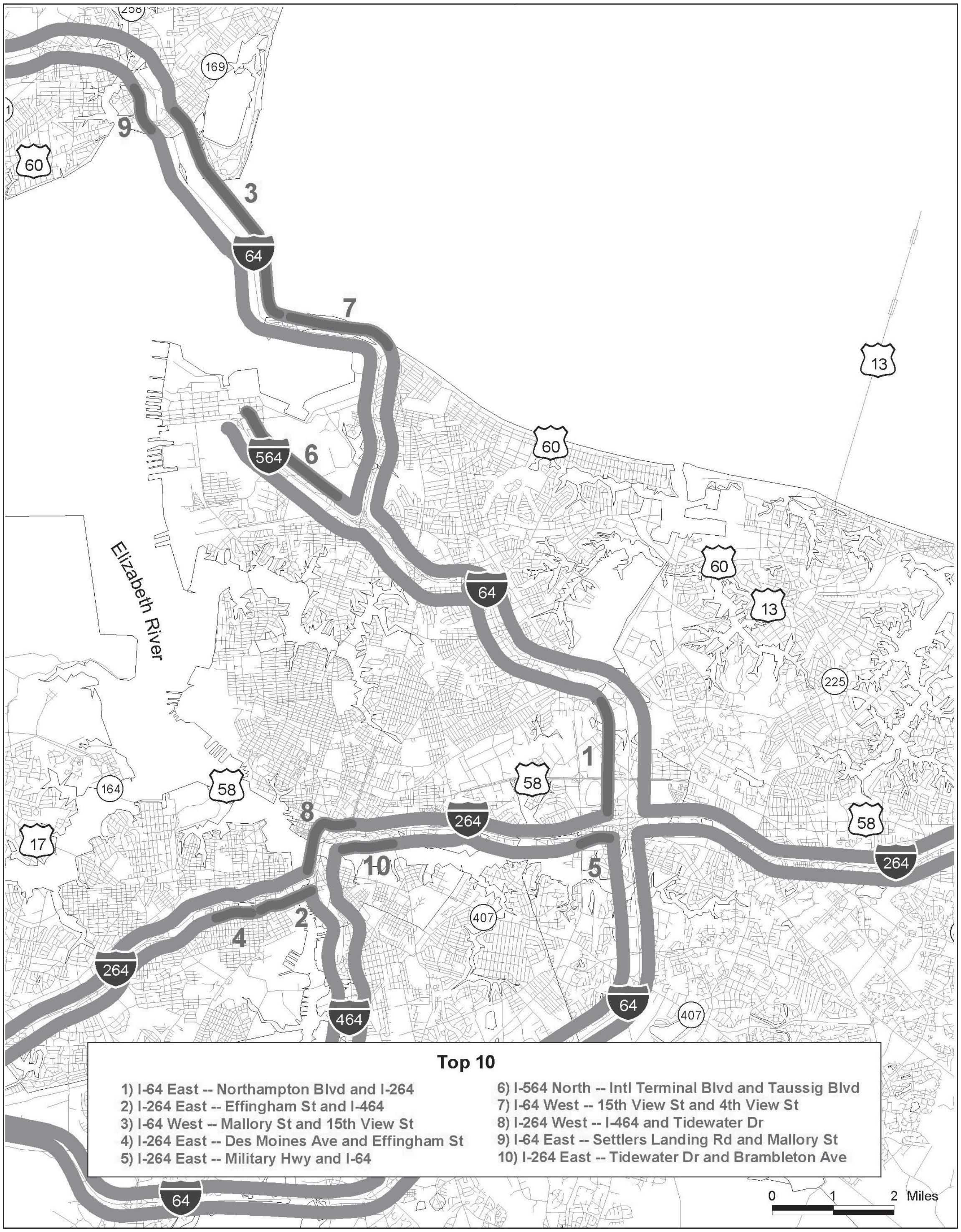

FIGURE 1 Top-10 high-crash Interstate segments by EPDO crash rate, 1998–2000. (Source: HRPDC.)

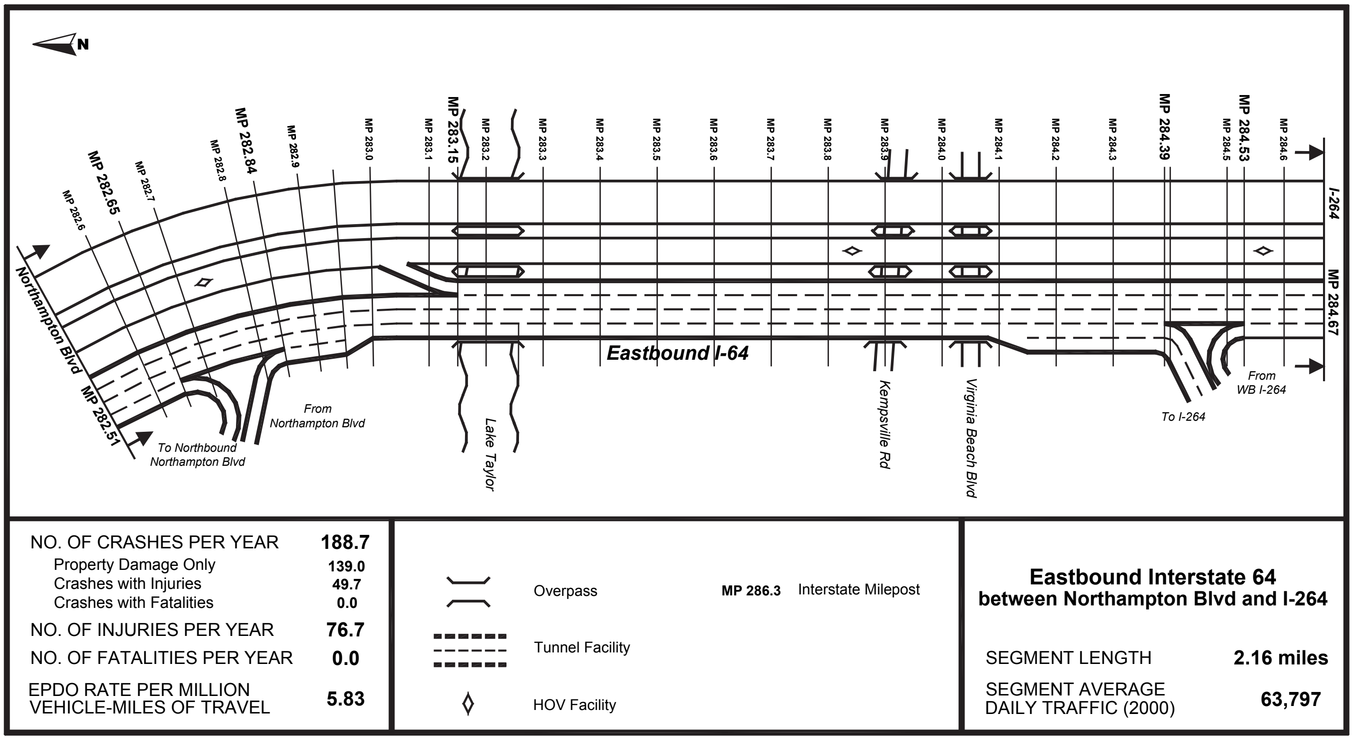

FIGURE 2 Interstate example, roadway geometry (not to scale).

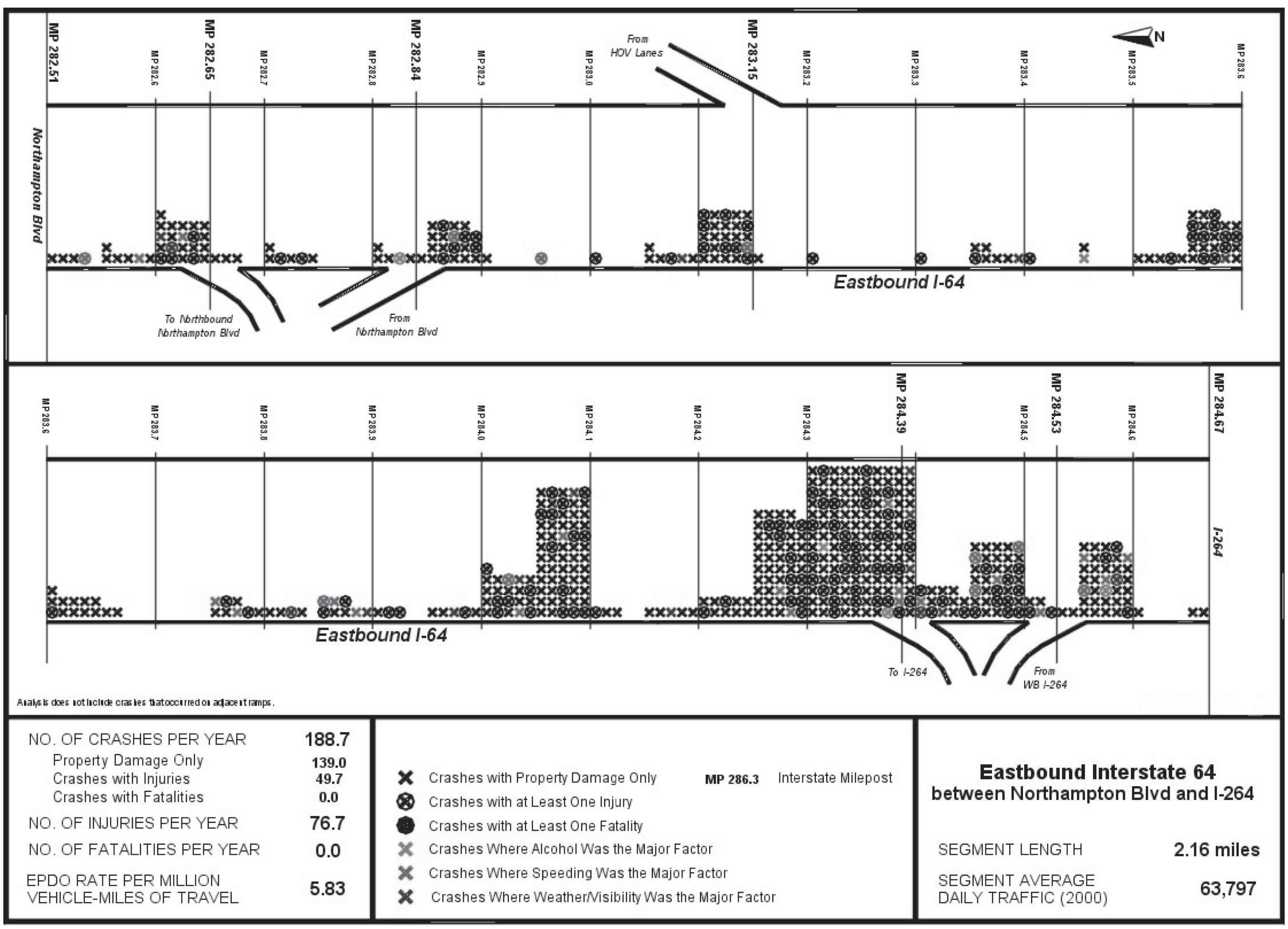

FIGURE 3 Interstate example, collision diagram (not to scale).

ACCIDENT SUMMARY

	1998	1999	2000	Total
Total Crashes	181	166	219	566
PDO Crashes	134	119	164	417
Injury Crashes	47	47	55	149
Fatality Crashes	0	0	0	0
Injuries	73	58	99	230
Fatalities	0	0	0	0

Segment Length (Miles)	2.16	
Average No. of Crashes Per Year	189	
Average EPDO	288	
Average EPDO Rate	5.83	
Average Daily Traffic (Eastbound)	63,797	
% of crashes involving heavy vehicles	- Segment	4.9%
	- Regionwide	7.8%

DRIVING UNDER INFLUENCE

Crashes with…	Segment	Regionwide
Nobody Drinking	97.5%	92.3%
Drinking Without Impaired Ability	0.7%	4.9%
Drinking - Not Known if Impaired	0.9%	1.4%
Drinking with Impaired Ability	0.9%	1.4%

DRIVER VISION

Crashes with…	Segment	Regionwide
No vision obscured	98.4%	96.4%
Obscured by rain/snow	0.5%	1.1%
Obscured by trees/hills	0.2%	0.2%
Obscured by other vehicles	0.4%	0.6%
Obscured by sun/glare	0.0%	0.3%
Other	0.0%	0.3%
Not stated	0.5%	1.1%

TIME OF DAY

Time of Day	Segment	Regionwide
6:00am - 8:59am	12.9%	17.3%
9:00am - 2:59pm	35.5%	27.9%
3:00pm - 6:59pm	**43.3%**	32.0%
7:00pm - 5:59am	8.3%	22.8%

DAY OF THE WEEK

Day of Week	Segment	Regionwide
Monday	17.8%	14.6%
Tuesday	13.4%	13.9%
Wednesday	15.4%	14.3%
Thursday	15.0%	15.0%
Friday	**24.0%**	18.7%
Saturday	5.1%	12.6%
Sunday	9.2%	10.9%

WEATHER

Weather	Segment	Regionwide
Clear/Cloudy	82.2%	79.2%
Fog	0.2%	0.3%
Rain/Mist	16.3%	18.8%
Snow/Sleet	1.4%	1.4%
Other/Not Stated	0.0%	0.3%

NUMBER OF VEHICLES

Vehicles Per Crash	Segment	Regionwide
1	7.6%	26.4%
2	**56.0%**	50.4%
3	**26.1%**	16.5%
4	7.2%	4.8%
5+	3.0%	1.9%

CRASH TYPE

Crash Type	Segment	Regionwide
Rear End	**77.6%**	53.9%
Head On	0.0%	0.3%
Sideswipe	11.7%	13.3%
Fixed Object	9.4%	28.2%
Pedestrian	0.0%	0.1%
Animal	0.0%	0.4%
Other	1.4%	3.8%

MAJOR FACTORS

Major Factors	Segment	Regionwide
Miscellaneous	3.2%	5.3%
Driver Asleep/Handicap	0.4%	3.4%
Driver Under the Influence	1.1%	4.8%
Driver Speeding	5.3%	10.7%
Driver Error	**84.1%**	67.0%
Vehicle Defective	1.4%	2.3%
Weather/Visibility	0.5%	1.1%
Road Defective	0.0%	0.0%
Road Slick	2.5%	3.8%
Not Stated	1.6%	1.6%

MOST PREVALENT DRIVER ACTION

Driver Action	Segment	Regionwide
Following too closely	**47.2%**	27.8%
Driver Inattention	**14.7%**	22.9%
Exceeded safe speed but not speed limit	9.2%	11.5%
Exceeded speed limit	4.1%	9.5%
Cutting in	3.9%	4.0%
Avoiding other vehicle	2.1%	3.0%

FIGURE 4 Interstate example, crash analysis data and statistics: I-64 eastbound between Northampton Boulevard and I-264.

Interstate Segment: I-64 EB between Northampton Blvd & I-264
Jurisdiction: Norfolk

EPDO Rate per Million Vehicle Miles of Travel: 5.83
Regional Rank by EPDO Rate: 1 of 151

Number of Crashes Per Year 188.7
Number of Injuries Per Year 76.7
Number of Fatalities Per Year 0

Segment Average Daily Traffic 63,797

OBSERVATIONS:

- 43% of crashes occurred during the PM peak period (3 – 7 pm), which was higher than the regional average of 32%.
- 24% of crashes occurred on Friday, while the regional average was 18.7%.
- Mostly 2 (56%) and 3 (26.1%) vehicle crashes occurred at this interstate location, which was higher than the regional averages of 50.4% and 16.5%, respectively.
- 77.6% of crashes were the result of rear ends. Regional average was 53.9%. The other significant crash types were sideswipe and fixed object.
- 84% of crashes resulted in drivers making errors and bad judgment. Regional average was 67%.
- It appears that over half of all crashes were the result of following too closely and driver inattention.
- A close examination of the collision diagram for this segment of I-64 indicates that 40% of all crashes occurred in approaches to the heavily congested interchange with I-264. Crashes were also frequent at the ramp junctions.

REMEDIES AND COUNTERMEASURES:

- There are currently no capacity improvements planned and programmed for this segment of I-64. One solution would be to extend the Northampton entrance ramp through to the high-occupancy vehicle (HOV) exit ramp at milepost 283.15 to eliminate the bottleneck that currently exists; there will, of course, be high costs associated with this improvement.
- Encourage carpools and vanpools on I-64 HOV lanes.
- Increase employees' use of HOV Express buses traveling between communities in VA Beach and the Norfolk Naval Base.
- Educate the public of the dangers involved in tailgating and not paying attention when traveling on the interstate, particularly during congested time periods.
- Police enforcement of tailgating.
- Improve the incident response and clearance times during peak periods.
- Expedite the Regional Telecommute program that was approved in 2003 as part of the Governor's Congestion Relief Program for Hampton Roads.
- Eliminate fixed objects and debris from the interstate.

FIGURE 5 Interstate example, crash analysis summary, 1998–2000.

severity and compared to the VMT along a segment for a given period. Results showed that on average there were 24.7 crashes per year per Interstate segment. Of the 151 roadway segments, 13 had EPDO crash rates per million VMT (MVMT) of greater than or equal to 3.0. The most hazardous location of these segments had an EPDO crash rate per MVMT of 5.83. The average EPDO crash rate was 1.62 per MVMT.

Among the leading reasons that so many crashes have occurred on the Interstate system in Hampton Roads are tailgating and not paying attention when driving. Study results indicated that 28% of the crashes were caused by following too closely and 23% by driver inattention on the Interstate system. The study also found that increasing congestion on roads is a major contributing factor to crashes during peak periods. Major crash types were found to be rear-end (54%), fixed object (28%), and sideswipe (13%). Sixty-seven percent of crashes were caused by driver error, and the overwhelming majority, 80%, took place during good weather. As expected, the most accident-prone time of the week was found to be Fridays between 3:00 and 7:00 p.m. Finally, the study revealed that on average, 92% of the crashes involved sober drivers, and in 96% of the crashes the driver's vision was not blocked.

Since many crashes throughout the region are the result of driver error, the study recommendations focused largely on enhanced driver education. In many instances, the study called on increased police enforcement or redesigned roadways to enhance driver safety, although little money is available for major rebuilding of roadways. More carpooling and transportation demand management measures were encouraged as a way to reduce the number of vehicles, particularly during peak periods. Recommendations were made to improve real-time information delivered to motorists via variable message signs in advance of congested Interstate segments. A regional freeway traffic management system was implemented as part of the region's intelligent transportation systems deployment, which should help with some of these issues.

Furthermore, although 92% of the crashes were not related to alcohol consumption, some Interstate segments had a high number of alcohol-related crashes. Recommendations were made to provide regional safety forums and coordinate efforts with other public agencies to educate the public on the hazards of drinking and driving.

TABLE 1 Top High-Crash Intersections in Hampton Roads, 1998–2000

City of Chesapeake	
▪ Top Crash	– Battlefield Boulevard at Volvo Parkway
▪ Top EPDO	– Dominion Boulevard at Cedar Road
Gloucester County	
▪ Top Crash	– Guinea Road at George Washington Highway
▪ Top EPDO	– Same
City of Hampton	
▪ Top Crash	– Coliseum Drive at Mercury Boulevard
▪ Top EPDO	– LaSalle Avenue at Settlers Landing Road
Isle of Wight County	
▪ Top Crash	– Brewers Neck at Carrollton Boulevard
▪ Top EPDO	– Same
James City County	
▪ Top Crash	– Richmond Road at Lightfoot Road
▪ Top EPDO	– John Tyler Highway at Centerville Road
City of Newport News	
▪ Top Crash	– Jefferson Avenue at Oyster Point Road
▪ Top EPDO	– Briarfield Road at Chestnut Avenue
City of Norfolk	
▪ Top Crash	– Newtown Road at Virginia Beach Boulevard
▪ Top EPDO	– Ocean View Avenue at 4th View Street
City of Poquoson	
▪ Top Crash	– Victory Boulevard at Wythe Creek Road
▪ Top EPDO	– Same
City of Portsmouth	
▪ Top Crash	– George Washington Highway at Victory Boulevard
▪ Top EPDO	– Same
City of Suffolk	
▪ Top Crash	– Constance Road at Main Street
▪ Top EPDO	– Godwin Boulevard at Kings Highway
City of Virginia Beach	
▪ Top Crash	– Lynnhaven Parkway at Princess Anne Road
▪ Top EPDO	– Pacific Avenue at 22nd St
City of Williamsburg	
▪ Top Crash	– Jamestown Road at Route 199
▪ Top EPDO	– Capitol Landing Road at Bypass Road
York County	
▪ Top Crash	– Rochambeau Drive at Route 143
▪ Top EPDO	– Route 132 at Route 143

Intersections

The intersection crash analysis was slightly different from that of the Interstate segments. Each city in the region used a different methodology to collect crash data, and in some cases the data were available for only two of the three years of the study period. This lack of standardization made regional comparisons difficult and resulted in treating the top high-crash locations in a slightly different manner. The intersections that were further analyzed were determined by using the top high-crash location by crash frequency and the top high-crash location by crash severity (EPDO crash rate) for each jurisdiction (Table 1). For some jurisdictions, the same intersection had both the top number of crashes and the highest EPDO crash rate; in these cases, only one intersection was analyzed for remedies and countermeasures to improve safety problems. Similar to the Interstate analysis, the intersection analysis included roadway geometry diagrams, collision diagrams, and crash data summaries for each of the 22 analyzed intersections. (Examples are shown in Figures 6 through 9.) These analyses and summaries helped to identify probable causes and deficiencies for the existing high-crash intersection locations.

On average, approximately 8,200 crashes per year occurred at CMS at-grade intersections in Hampton Roads, and 3,568 injuries and 22 fatalities per year resulted from these crashes. This accounts for nearly 19% of all traffic crashes, 19% of all injuries, and 15% of all fatalities in Hampton Roads. The top high-crash intersection location resulted in an EPDO rate of 3.73 per million entering vehicles (MEV), and the average EPDO rate was 1.62 per MEV.

Among the major reasons that crashes occurred at intersections throughout the region were driver inattention (18%), driver not having the right-of-way (18.5%), and following too closely (16%). Combined, disregarding the traffic signal and hit-and-run conditions resulted in 10% of all crashes. The off-peak period between 9:00 a.m. and 3:00 p.m. had the highest percentage of crashes, and Friday was the day of the week with the most crashes. Twenty percent of all crashes occurred during rainy days and bad weather. Major crash types at these intersections were rear-end (41%), right-angle (40%), sideswipe (7%), and fixed-object (7%). Similar to the Interstate system, 7%

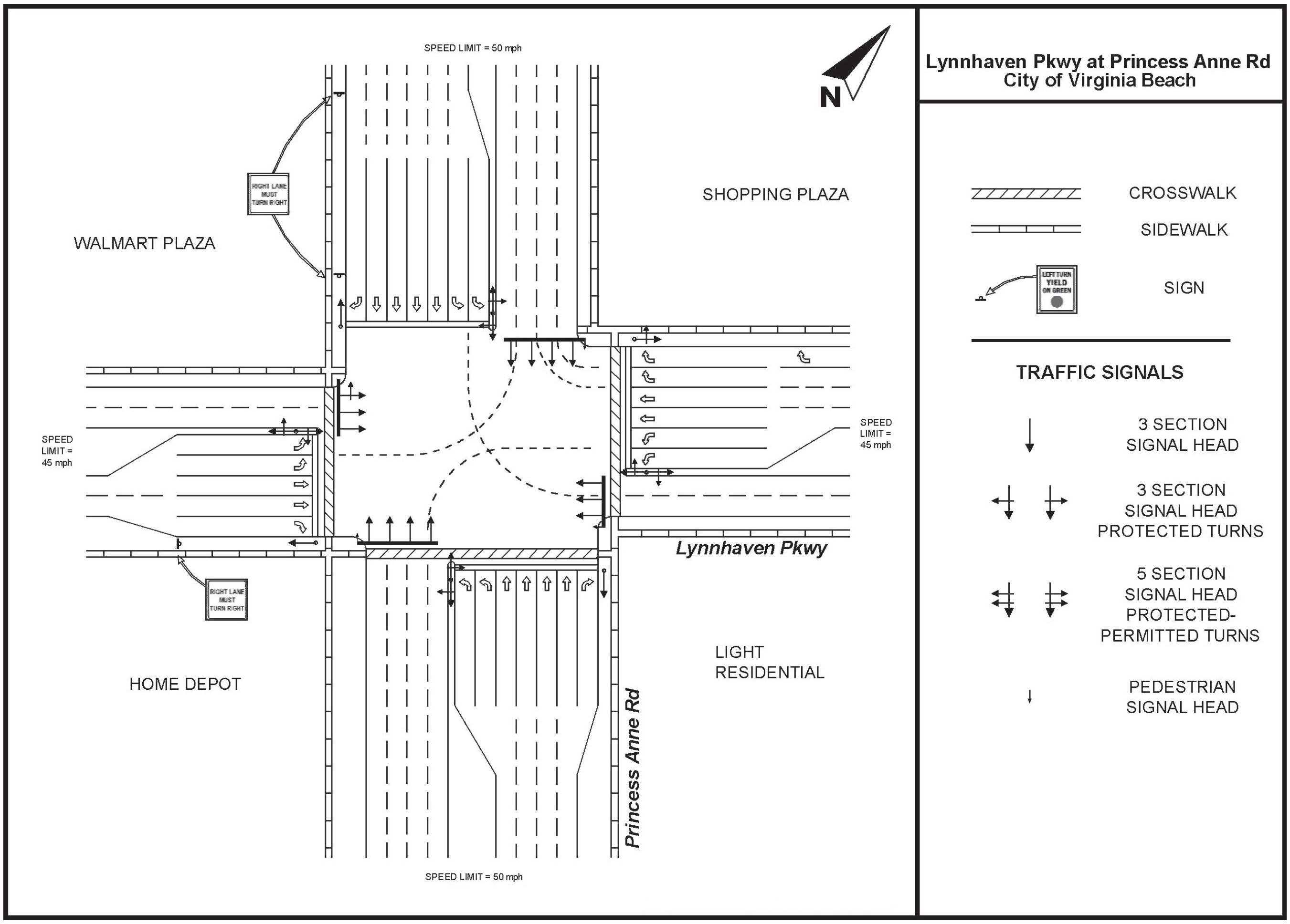

FIGURE 6 Intersection example, roadway geometry (not to scale; locations of signs and symbols are approximate).

of the crashes involved drinking. Driver error was by far the major factor contributing to these crashes.

Specific recommendations were made for each of the top high-crash intersection locations analyzed for each jurisdiction. Common remedies and solutions included increasing roadway capacity to reduce congestion, adding turning lanes, and improving signal timings. Other study recommendations included improving signage, increasing enforcement, and providing additional driver education with emphasis on the hazards of drinking and driving.

CONCLUSIONS

Previous safety considerations in Hampton Roads took place at the project level and on individual facilities through traffic engineering improvements. Education and enforcement efforts are still largely developed and conducted outside the traditional planning process. TEA-21 required that safety considerations be incorporated into the planning process with systemwide and multimodal perspectives. The U.S. Department of Transportation is working to incorporate safety into the next surface reauthorization package. This effort will focus on several core principles and values, which includes making substantial improvements in the safety of the nation's surface transportation system.

As roadway safety remains a nationwide focus and priority, the HRPDC staff, as part of its CMS program, initiated a comprehensive regional safety study in 2001 to identify causes and solutions for high-crash locations in Hampton Roads. Location maps, roadway geometry diagrams, collision diagrams, crash data summaries, observations, and remedies were provided for the top-10 high-crash Interstate segments by EPDO crash rate as well as the top intersection by EPDO crash rate and the top intersection by number of crashes for each Hampton Roads jurisdiction.

The regional safety study revealed the following as main causes of traffic crashes in Hampton Roads:

- Congestion;
- Driver actions:
  - Failure to yield the right-of-way,
  - Following too closely,
  - Driver inattention,
  - Disregarding traffic signals and signs;
- Rear-end or right-angle collisions;
- Weather;
- Access management;
- Driving under the influence; and
- Speeding.

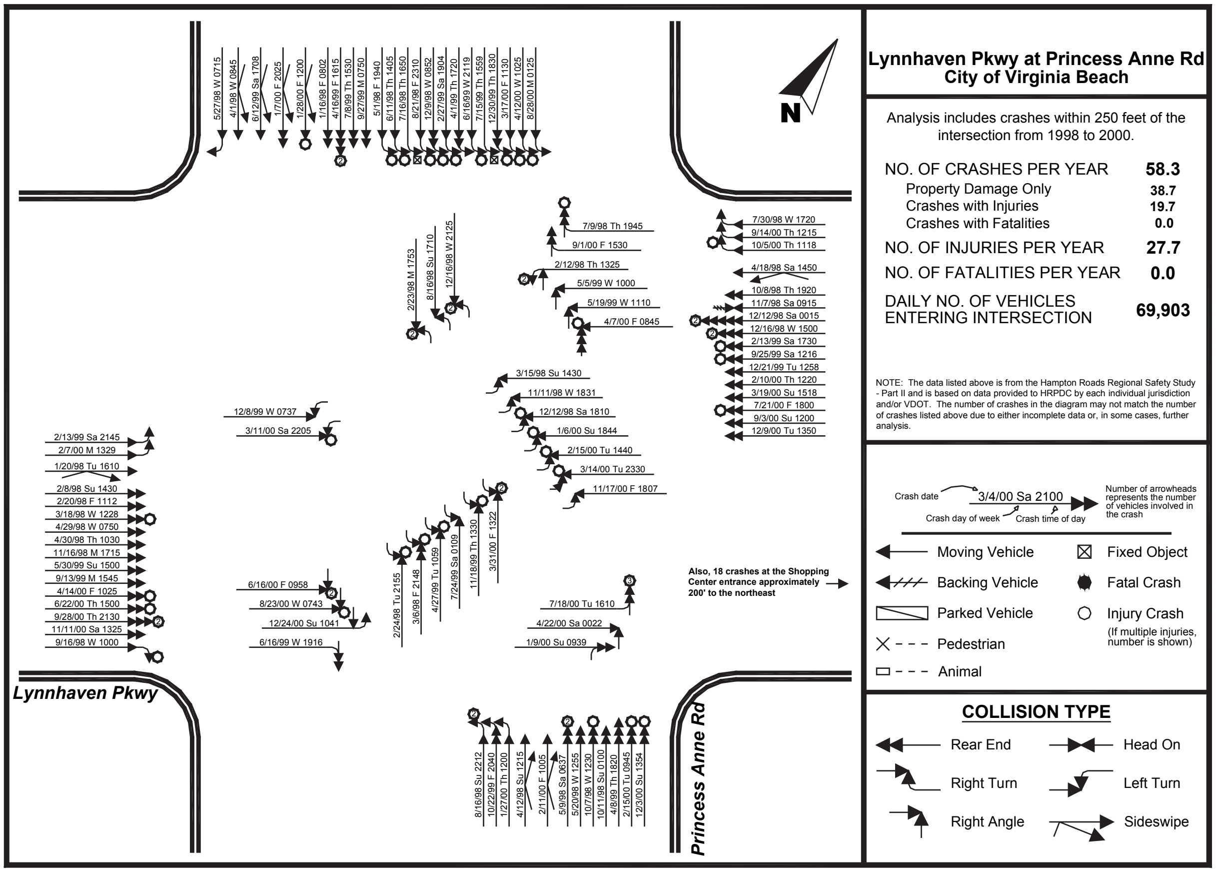

FIGURE 7 Intersection example, collision diagram.

ACCIDENT SUMMARY

	1998	1999	2000	Total
Total Crashes	61	61	53	175
PDO Crashes	38	43	35	116
Injury Crashes	23	18	18	59
Fatality Crashes	0	0	0	0
Injuries	35	23	25	83
Fatalities	0	0	0	0

Average Number of Crashes	58.3	
Average EPDO	97.7	
Average EPDO Rate	3.83	
Volume Entering Intersection	69,903	
% of crashes involving heavy vehicles - Segment		1.9%
- Regionwide		5.3%

DRIVING UNDER INFLUENCE

Crashes with...	Intersection	Regionwide
Nobody Drinking	95.6%	92.8%
Drinking Without Impaired Ability	2.2%	1.5%
Drinking - Not Known if Impaired	0.0%	1.3%
Drinking with Impaired Ability	2.2%	4.4%

DRIVER VISION

Crashes with...	Intersection	Regionwide
No vision obscured	81.5%	83.8%
Obscured by rain/snow	0.0%	3.1%
Obscured by trees/hills	0.0%	0.6%
Obscured by other vehicles	10.0%	4.5%
Obscured by sun/glare	0.0%	1.3%
Other	2.3%	2.9%
Not stated	6.2%	3.8%

TIME OF DAY

Time of Day	Intersection	Regionwide
6:00 - 8:59	6.3%	11.4%
9:00 - 14:59	38.9%	35.4%
15:00 - 18:59	32.0%	29.9%
19:00 - 5:59	22.9%	23.3%

DAY OF THE WEEK

Day of Week	Intersection	Regionwide
Monday	6.9%	14.2%
Tuesday	10.9%	14.8%
Wednesday	10.3%	14.7%
Thursday	17.1%	15.2%
Friday	15.4%	17.2%
Saturday	23.4%	14.0%
Sunday	16.0%	9.9%

WEATHER

Weather	Intersection	Regionwide
Clear/Cloudy	83.2%	80.5%
Fog	0.0%	0.4%
Rain/Mist	14.5%	17.8%
Snow/Sleet	1.2%	0.7%
Other/Not Stated	1.2%	0.6%

MOST PREVALENT DRIVER ACTION

Driver Action	Intersection	Regionwide
Hit and run	27.1%	4.6%
Following too closely	18.1%	15.7%
Did not have ROW	15.5%	18.5%
Disregarded stop - go light	9.7%	5.4%
Driver inattention	5.8%	17.8%

CRASH TYPE

Crash Type	Intersection	Regionwide
Rear End	41.2%	40.6%
Right Angle	39.9%	39.5%
Head On	0.7%	2.1%
Sideswipe	7.2%	7.7%
Fixed Object	7.8%	7.1%
Pedestrian	0.0%	1.4%
Animal	0.0%	0.2%
Other	3.3%	1.4%

PRIMARY CRASH MOVEMENT

Crash Type and Direction	Percent
Type #1 ↓	16.5%
Type #2 →→	15.5%
Type #3 ←←	14.4%

MAJOR FACTORS

Major Factors	Intersection	Regionwide
Miscellaneous	1.8%	2.7%
Driver Asleep/Handicap	0.0%	1.8%
Driver Under the Influence	2.7%	3.9%
Driver Speeding	0.0%	1.8%
Driver Error	84.1%	80.6%
Vehicle Defective	0.0%	0.5%
Weather/Visibility	9.7%	6.4%
Road Defective	0.0%	0.2%
Road Slick	0.9%	1.4%
Not Stated	0.9%	0.7%

FIGURE 8 Intersection example, crash analysis data and statistics: Lynnhaven Parkway at Princess Anne Road, city of Virginia Beach.

Intersection: **Lynnhaven Pkwy at Princess Anne Rd**
Jurisdiction: **City of Virginia Beach**

EPDO Rate per Million Entering Vehicles:	3.83
Regional Rank by EPDO Rate:	19 of 468
Number of Crashes Per Year	58.3
Number of Injuries Per Year	27.7
Number of Fatalities Per Year	0.0
Daily Number of Vehicles Entering Intersection	69,903

OBSERVATIONS:

- 10% of crashes occurred where other vehicles obscured the driver's vision. Regional average was 4.5%.
- Many crashes occurred during the weekend. 23.4% of crashes occurred on Saturday (regional average: 14%), while 16% occurred on Sunday (regional average: 9.9%).
- The primary crash movement and type were southeast bound rear end collisions along Princess Anne Rd from Ferrell Pkwy (16.5%).
- Most prevalent driver actions resulting in the crash were hit and run (27.1%), following too closely (18.1%), and disregarding stop-go light (9.7%).
- In addition, 18 crashes occurred at the shopping center entrance approximately 200 feet to the northeast of the intersection along Lynnhaven Pkwy as a result of backed up traffic.

REMEDIES AND COUNTERMEASURES:

- Consider adding 'Slow down signal ahead' sign along Princess Anne Rd from Ferrell Pkwy. Currently, traffic moves very fast at this southeast approach.
- Educate the public of the dangers involved in following too closely and not paying attention when traveling through a busy intersection.
- Consider the use of automated safety enforcement, such as red-light cameras, to guard against crashes resulting from red-light running.
- Review signal clearance times at the intersection.
- Review signal phasing and timing at the intersection.
- Consider access management solutions or the addition of a traffic signal (if feasible because of the close proximity with the intersection) for the shopping center entrance along Lynnhaven Pkwy.

FIGURE 9 Intersection example, crash analysis summary, 1998–2000.

Analysis of the top high-crash location resulted in the following general remedies and countermeasures to initiate safety improvements in Hampton Roads:

- Increase roadway capacity;
- Reduce demand by encouraging transportation demand management strategies;
- Add turning lanes at intersections;
- Improve signal timing, phasing, and, particularly, clearance times;
- Improve signage;
- Improve enforcement; and
- Provide additional driver education.

Since the completion of the regional safety study, several localities have used the results to initiate safety grant applications and secure funding for projects to be included in the region's TIP. Furthermore, the study results are being reviewed by VDOT for programming safety funds. Finally, the cities are using the results of this study to educate and inform their citizens through civic organizations and other local groups.

The MPO staff will maintain and update the regional crash database, which was created as part of this comprehensive study, and will release updated reports periodically. This task will be accomplished through the triennial update of the regional CMS plan.

LESSONS LEARNED

Safety planning has taken on a much higher profile nationally since HRPDC initiated this study in July 2001. During preparation of the methodology for this study, few other metropolitan areas were conducting safety planning studies, and national standards were difficult to find. The next surface transportation reauthorization package is likely to further emphasize safety planning, and a new highway safety manual is being created, and so standards for conducting safety studies should become more prevalent.

Collecting data for individual crashes on a regional level was a more arduous task than was expected. Although VDOT maintains a database with substantial data for each crash throughout the state, locations of crashes are not included for non-Interstate roadways within cities. Most jurisdictions maintain their own crash databases, but the quantity and quality of the data within these databases varies greatly. For example, certain jurisdictions listed a crash in their databases as occurring at the intersection although it may have occurred in an adjacent parking lot. Other jurisdictions listed crashes in their databases only if they occurred within the intersection; crashes that were listed as being as little as 5 ft away may not have been included in the database. It is therefore difficult if not impossible to compare intersection crash rates on a regional level by using jurisdictional crash databases.

That crash locations are not included for non-Interstate roadways in cities lessens the value of the VDOT crash database as a safety planning tool. Incorporating locations for each crash, regardless of whether in a county or a city, would improve the reliability of data in regional safety studies and significantly reduce the time it takes to complete similar studies (and therefore provide more up-to-date results).

NEXT STEPS

The Hampton Roads regional safety study was the first step in integrating safety into the transportation planning and programming process. Identifying high-crash locations and ensuring recommended projects are to be included in the region's TIP are critical elements of this process. The HRPDC staff will maintain, update, and improve periodically the regional crash database as part of its CMS program. Ongoing efforts and cooperation are under way to incorporate all crash locations, regardless of jurisdiction, into the VDOT statewide crash database. This would drastically improve the collection and quality of data for all cities within Hampton Roads. The statewide safety management system (SMS) was established in Virginia as a result of the Intermodal Surface Transportation Efficiency Act of 1991. Although not required by TEA-21, the state continued to discuss safety issues through the SMS, and the committee has met regularly in the last 4 years. Since the completion of this regional safety study, the SMS committee, consisting of representatives from various public agencies, has expanded its membership to include the HRPDC. A goal of the SMS committee is to create and maintain a high-quality, integrated data system for evaluation and analysis. Local governments and VDOT are encouraged to secure funding to incorporate high-crash location projects into the next TIP. Results of this study will be used in the development of the region's 2030 long-range transportation plan. Finally, HRPDC will continue to implement safety planning steps as proposed in the next reauthorization.

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Developing Operational and Safety Guidelines for School Sites in Texas

Scott A. Cooner

The objective of a two-year study was to recommend school site planning guidelines for transportation-related elements such as site selection, general site requirements and design, bus operations, parent drop-off and pickup zones, driveways, turn lanes, signing and marking, parking, and pedestrian and bicycle access. The research team based these guidelines on a comprehensive review of existing guidelines and the results of field studies at school sites in Texas. Examples are provided of good practices and of practices to avoid for three of the more prominent guidelines. The guidelines are focused on transportation design, operations, and safety within school sites—with a particular focus on the parent drop-off and pickup zones. A site plan review checklist based on the 21 consensus guidelines approved by the project advisory panel is provided. Texas Department of Transportation engineers, field crews, architects, and school district personnel can use this checklist to coordinate efforts and improve the safety and efficiency of school site access and traffic flow

The state of Texas, particularly in the large urban areas, has experienced considerable recent population growth. This growth has produced new schools on sites near highways originally designed for low volumes and high speeds. Another trend is the higher proportion of children being transported to schools in private vehicles. These realities make it important to consider the design of roadways within and around schools. Equally important is the consideration of the location and design of the school site, preferably during the planning stages, to establish safe and efficient operations.

The Texas Department of Transportation (TxDOT) has focused attention on these issues through its Precious Cargo program (*1, 2*). Precious Cargo allows TxDOT to review school site plans and make recommendations before construction. TxDOT has assisted independent school districts (ISDs) through application of transportation principles and fundamentals, but its efforts sometimes have been limited by the lack of knowledge of the specific problems associated with school transportation needs and the lack of acceptable guidelines. This research addressed these limitations and provided an opportunity to enhance Precious Cargo by providing TxDOT staff, ISD personnel, and the other stakeholders with guidelines and good examples for the design and operation of roadway facilities around schools (*3–5*).

This paper summarizes a 2-year study by the Texas Transportation Institute (TTI) to develop recommended school site planning guidelines for transportation-related elements such as site selection, general site requirements and design, bus operations, parent drop-off and pickup zones, driveways, turn lanes, signing and marking, parking, and pedestrian and bicycle access (*4*). The research team based these guidelines on a comprehensive review of existing guidelines and the results of field studies at school sites in Texas. The paper provides a few examples of good practices and examples of practices to avoid for several of the more prominent guidelines. The guidelines are focused on transportation design, operations, and safety within school sites with a particular focus on the parent drop-off and pickup zones.

STUDY METHODOLOGY

Initially, researchers used a variety of methods to review existing guidelines for transportation-related elements at school sites. Second, the research team interviewed architects, consulting engineers, and ISD personnel about current school site planning methods and resources. Researchers also used surveys to identify current site plan review practices used by TxDOT and local municipalities. Next, the research team performed observational studies at 14 schools to assess different school site designs and to refine data collection methods and procedures (*3*).

In the second year of the project, researchers conducted field studies at 20 school sites throughout the state to collect detailed operational and safety data to assess various site designs and loading zone strategies (*5*). The research team developed guidelines and good examples for the design and operation of transportation-related elements within and around school sites.

STUDY FINDINGS

Review of Existing Guidelines

Researchers used published documents, Internet searches, survey instruments, and direct correspondence to gather information on existing guidelines. This effort produced some key findings

- Much of the state of the practice is found in state department of transportation and local ISD Internet sites.
- Two state departments of transportation, North Carolina and South Carolina, dedicate units for review of school site plans, and they have developed guidelines based on experience and study of existing sites (*6, 7*).
- The most universally cited guideline is separation of modes (auto, bus, and pedestrian).
- A recent TRB study indicated that school buses are the safest form of transport for getting children to and from school (*8*).

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• Some existing guidelines conflict (e.g., engineering guideline to provide adequate on-site stacking space versus architectural practice to place school building near front of the site).

Results of Interviews and Surveys of School Transportation Stakeholders

Architects

Architecture firms normally are the lead entity on most school construction projects. To gain a better understanding of challenges, issues, and methods used to plan and design educational facilities, researchers conducted interviews with six architecture firms with considerable school design experience. The following list presents three key finding from this effort:

• Most resources do not provide any substantial guidance on transportation-related issues.
• Only three of 10 participants indicated an awareness of Precious Cargo; however, half had at least one site plan reviewed by TxDOT before construction of a new school campus.
• Most participants (70%) stated that the most challenging problem with traffic access and circulation at schools was separating vehicle, bus, and pedestrian traffic.

School District Personnel

According to data collected by *School Planning & Management* magazine, the state of Texas has led the nation in the development and renovation of school campuses, spending more than $19 billion on construction of K–12 facilities between 1992 and 2000 (*9*). These data indicate that Texas ISDs are building a large number of schools, and increasing numbers are being located on or near state-maintained roadways. Researchers conducted interviews with eight ISD personnel about transportation-related elements of school projects. The following are three key findings from this effort:

• Separation of traffic types (vehicles, buses, and pedestrians) was the highest-rated problem area at all campus types (elementary, middle/junior, and high schools).
• Slightly more than half (56%) were aware of Precious Cargo; 40% had at least one site plan reviewed by TxDOT before construction of a new school.
• Demographics (i.e., locations of existing and future students) were the most important factor in the selection of future land parcels for development of new school campuses.

Consulting Engineers

Civil or transportation engineers support architects on many projects with traffic-related elements. A member of the research team interviewed two consulting engineers with extensive experience in school projects regarding coordination issues with architects and the design principles they commonly use. This effort produced two key findings First, the integration of traffic circulation with the school building's location was important, but consulting engineers typically were brought in late in the process and in some cases were called on after construction to devise solutions to access and circulation problems. Second, design guidelines for parent zones were sketches or in-house sources (no written guidelines).

TxDOT and Municipal Engineers

Researchers mailed a survey to each TxDOT district and most of the major municipalities in Texas. The survey gathered information on how school site plans were reviewed and also identified good (and not-so-good) examples for design and operation of transportation facilities at schools. The following are key findings from this effort:

• TxDOT and cities preferred to be involved very early in the school site planning process.
• When reviewing a school site plan, TxDOT and cities overwhelmingly use the *Manual on Uniform Traffic Control Devices* (MUTCD) (*10*) and engineering judgment. TxDOT staff also used the *Roadway Design Manual* (*11*), and cities used local guidelines.
• TxDOT has no requirement in place for school sites to have a traffic impact analysis (TIA); however, four of the nine city respondents required a TIA.

Observational Case Studies

In the first year of the project, researchers performed observational case studies at seven elementary, five middle, and two high schools (*3*). Some key findings from this effort are as follows:

• The average amount of time spent dropping off or picking up was significantly more variable in the afternoon than in the morning.
• There was a wide variety of design, operational, and traffic control practices (lack of uniformity).
• Some schools used innovative practices, such as placement of traffic cones and use of students and staff for on-site traffic control, to improve safety and traffic flow

Researchers also observed several typical problems. Examples of these problems are the following:

• Lack of sufficient on-site stacking length, which caused the queue of vehicles trying to access the school site to spill back onto adjacent roadways;
• Undesirable behaviors, such as circumvention of traffic control (e.g., "Do Not Enter" and turn restriction signs) and use of nondesignated areas for loading (e.g., parking lots);
• Signs and pavement markings that were not consistent with accepted MUTCD standards;
• Lack of supervision of on-site loading zones, particularly during morning drop-off;
• Low proportion at many sites of students arriving by bus or walking, which contributed to the high volume of vehicles vying for access to the campus.

Field Studies

The research team conducted in-depth field studies at 20 schools and focused on elementary schools and parent drop-off and pickup zones (*5*). Researchers concentrated on collecting data at elementary sites because they are the most prevalent type of school (almost 60% of public schools in Texas) and are frequently cited for having problems. Some of the key field study findings for elementary sites are as follows:

- Student enrollments ranged from a low of 400 students to a high of 1,087 students, and many schools were beyond their design capacity.
- The percentage of students arriving at school by private vehicle ranged from a low of 34% to a high of 92%, with an average of approximately 60%.
- The percentage of students arriving at school by bus or day care vans ranged from a low of 1% to a high of 55%, with an average of approximately 33%.
- The percentage of students arriving at school by walking or cycling ranged from a low of 0% to a high of 21%, with an average of approximately 7%.
- On average, almost twice as many vehicles arrived in the morning as in the afternoon; however, maximum queues were twice as long in the afternoon because departure times were less variable.

General findings based on the pedestrian–vehicle conflict data collection included the following:

- Elementary sites experienced a relatively low rate of conflicts primarily because of good on-site supervision by staff members.
- Sites with two or more lanes for loading and unloading of students had more conflicts than those with single-file queue lanes.
- Middle school sites had significantly more conflicts compared to elementary sites, which was attributed to less supervision in the loading zone.

RECOMMENDED GUIDELINES

On the basis of the findings and lessons learned during the project activities, the research team developed recommended guidelines for the design and operation of transportation-related elements within and around schools. Researchers organized the guidelines into nine categories:

- Site selection criteria;
- General site requirements and design;
- Bus operations;
- Parent drop-off and pickup zones;
- Bicycle and pedestrian access;
- Driveways;
- Turn lanes;
- Traffic control, signing, and pavement markings; and
- Parking requirements and design.

The 21 recommended guidelines are provided in detail in Project Report 4286-2 (*4*). This project report also contains best practices and good examples of school site design and operations for these 21 guidelines, which were developed with the approval of the project advisory panel. The guidelines are focused on transportation design, operations, and safety within school sites, with a particular focus on the parent drop-off and pickup zones.

GUIDELINE 1

Guideline 1 says that school buildings should be set back on the site a sufficient distance from the adjacent roadways to ensure safe and adequate site storage for stacking of loading and unloading vehicles.

Building Setback Requirements

The review of existing guidelines for building setback requirements showed that no agencies had specific values for how far back from the roadway the school building had to be placed. Building setback is an important consideration because the placement of the building significantly affects the traffic circulation and amount of on-site space for stacking of vehicles. One agency had a general guideline that school buildings are to be set back on the site a sufficient distance from the adjacent roadways to ensure safe and adequate site storage or stacking of loading and unloading vehicles.

Best Practice for Application of Guideline 1

Figures 1 and 2 show examples of school sites located in the same ISD. Both schools are elementary schools that used the same prototype design for the school building.

Example to Avoid

The school site shown in Figure 1 was the first prototype elementary school built in the suburban ISD. In this case, the architect placed the school building near the front of the site, set back approximately 150 ft from the adjacent two-lane roadway. At this site, the queue of vehicles in the front loop driveway regularly spilled back out onto the adjacent roadway during morning drop-off and afternoon pickup operations, blocking through traffic.

Example of Good Practice

On the basis of this experience, the ISD built the next prototype elementary on a similar site but placed the building approximately 350 ft farther back on the site (see Figure 2). The increased setback distance provides more on-site stacking space and has resulted in better operations at the school.

GUIDELINE 2

Guideline 2 states that the physical routes provided for the basic modes (buses, cars, pedestrians, and bicycles) of the traffic pattern should be separated as much as possible from each other.

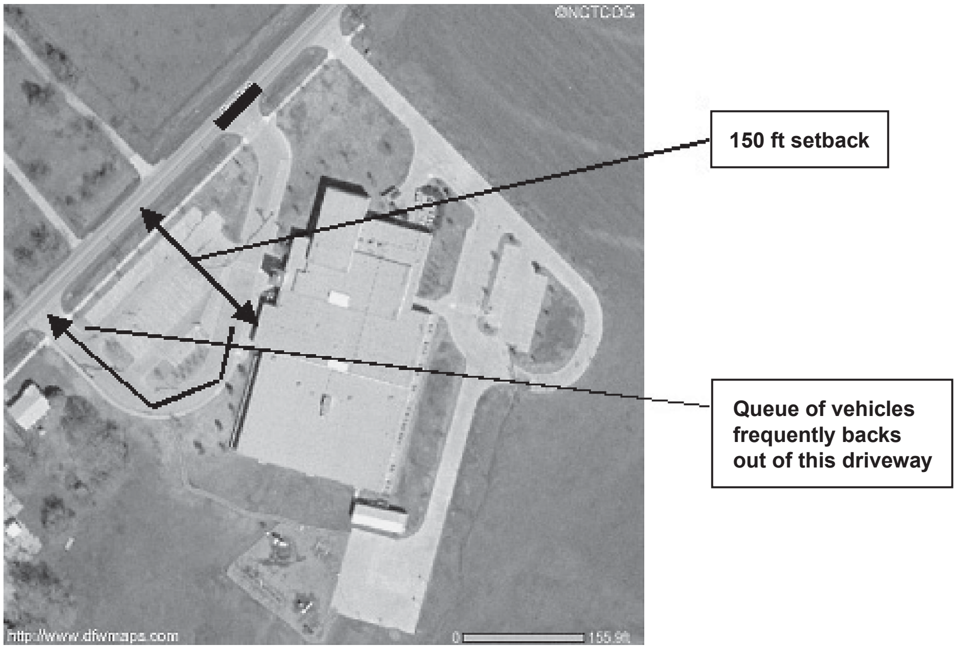


FIGURE 1 School building located near front of site: frequent queue spillback (*12*).

Separation of Modes

For the research team, perhaps the most universal guideline involving design and operations at schools is summarized in Guideline 2. Almost every source, whether from the architecture, transportation, or educational professions, had some guidance on providing for separation of the basic modes of travel for students within the school site. Providing for physical separation of the basic modes is both a design issue (e.g., layout of separate driveways, loading areas) and an operations issue (e.g., enforcement of bus-only zones, supervision of crosswalks). This guideline advocates separation of modes because it is important to limit exposure to modal conflicts that can lead to possible crashes, increased delay, and a general sense of chaos that can promote noncompliance with intended traffic access and circulation patterns.

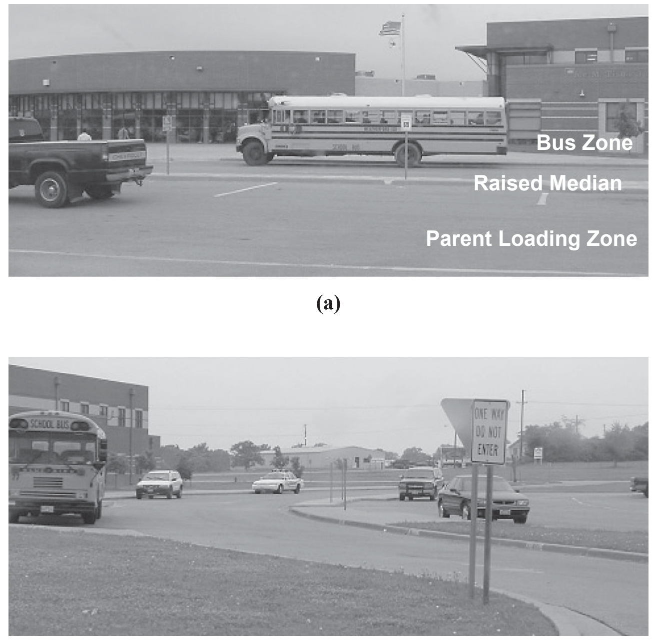
FIGURE 2 School building pushed back on site: better operations (*12*).

Example of Good Practice

Most sites included in the field studies had good separation of the basic arrival modes. Figure 3 is an aerial photo of an elementary school site that shows a good example of separation of parent vehicles, school buses, and pedestrians and bicyclists. The basic design of this school site provided for good separation; however, an operational change from the original layout improved the function of the site from the perspective of separating the basic modes of the traffic pattern.

The school principal made the operational change from the original layout because the queue in the loop driveway in front of the school frequently stacked out onto the adjacent roadway. The operational change involved closing this loop driveway to parent traffic and making it a pedestrian- and bicycle-only zone. The driveway on the south side of the school, labeled 1 in Figure 3, was then opened to be the parent drop-off and pickup zone. This site had a higher than average percentage, just over 20% of students, arriving by walking or cycling, which is at least partly attributable to the system of sidewalks and bicycle racks and the creation of the pedestrian- and bicycle-only zone. The driveway labeled 2 in Figure 3 serves as the entrance and exit for all the school buses.

FIGURE 3 School with good physical separation of basic modes of traffic pattern (*12*).



(a)

(b)

FIGURE 4 Site with layout to avoid—adjacent bus and parent zones.

Examples to Avoid

Figure 4 shows photographs of a junior high school site with a design that should be avoided. This site has some physical separation of modes—bus and parent zones are separated via a raised median and have separate entrance driveways. The layout at this site has the bus zone adjacent to the school entrance, and the parent zone is separated via a raised concrete median. Although these loading zones are physically separated, students dropped off in the parent zone have to cross the bus zone driveway to access the school entrance. This layout promotes pedestrian–bus conflicts. The other element of this site that did not work well and that violates the guideline of trying to separate modes is that parent vehicles and buses use the same exit driveway. In this case, the use of the same exit driveway creates unnecessary on-site congestion, particularly in the afternoon, when buses and parent vehicles are trying to exit at the same time.

Vehicles and buses using the same exit is not a design that necessarily should always be avoided. Separation of modes can be achieved through physical separation, time separation, or both. Several schools included in field studies had a design similar to that of the school shown in Figure 4 with a shared exit driveway; however, they were able to achieve time separation by having buses drop off early in the morning and then exit before parent vehicles in the afternoon. Note that the types of operation a school uses (e.g., one-way traffic pattern, time restrictions, loading supervision) are as key to safety and efficiency as the physical site design and layout.

GUIDELINE 3

Guideline 3 states that the design should provide an adequate driveway for stacking cars on site.

On-Site Stacking Length

The research team found several examples of guidelines similar to Guideline 3. Having adequate on-site stacking length to accommodate parent vehicles during the morning drop-off and afternoon pickup operations is important so that traffic flow and safety on adjacent roadways is not negatively affected. A corollary to Guideline 3 is the need to provide for an alternative vehicle routing or driveway expansion if the school has an increase in student population or car ridership percentage or inadequate driveway length is calculated or constructed.

A primary focus of the field studies during the 4286 project was to examine geometric design and operational practices in parent drop-off and pickup zones. Researchers concentrated on collecting sufficient data at elementary schools in Texas to be able to validate the existing South Carolina (*6*) and North Carolina (*13*) guidelines for on-site stacking length.

The data collected during the 4,286 field studies validated the school traffic calculator (*13*). It is good practice to use the afternoon pickup data to predict the maximum queue of vehicles. The maximum queue length is then used to design and appropriately size the length needed in the parent driveway for lining up cars on site. The analysis of the average, maximum, and 95th percentile queue data at Texas schools did not produce any statistically significant models based on a regression analysis (*5*). The data did show that the observed maximum queue lengths were often well below the recommended on-site stacking lengths given in Table 1 and those predicted by the school traffic calculator (*13*).

It appears that the South Carolina and North Carolina recommended on-site stacking lengths were more conservative than the Texas data. On the basis of this finding, the research team believes that the recommended on-site stacking lengths for Texas schools can be decreased and will still meet the objective of Guideline 3—providing an ade-

TABLE 1 South Carolina DOT Guidelines for On-Site Stacking Length (6)

School Type	Student Population	Loop Drive Stacking Length (linear feet) (m)
Elementary	200–600	900–1200 (274.5–366)
	600–1400	1200–1500 (366–457.5)
Middle	200–600	900–1200 (274.5–366)
	600–1200	1200–1500 (366–457.5)
High	400–800	800–1200 (244–366)
	800–2500	1200–1500 (366–457.5)

quate driveway for stacking cars on site. Although no statistically significant models were developed on the basis of queue length, the research team had sufficient data to formulate recommended on-site stacking lengths for Texas elementary and middle schools. From the data from this project, researchers recommend the on-site stacking lengths for high schools contained in Table 1 for Texas because no new field data were collected at Texas high schools (*5*). Table 2 provides the recommended on-site stacking lengths for Texas schools. (Note that for high school populations of greater than 2,500 students, two separate student pickup and drop-off loops should be considered.)

Examples to Avoid

During the case studies and field studies, the research team observed many sites that did not provide adequate on-site stacking length. The inadequate on-site space to accommodate the queue led to spillback on adjacent roadways. Figure 5 shows an intermediate school site where both lanes of the northbound direction of the adjacent roadway were blocked by the queue of vehicles that backed up from the parent drop-off and pickup zone driveway. Figure 6 shows another example of queue spillback at an elementary school site.

SITE PLAN REVIEW CHECKLIST

Table 3 provides the site plan review checklist, which contains the 21 consensus guidelines approved by the project advisory panel. The guidelines in the checklist are put in the form of questions that can be used to determine if a school site meets the recommended guidelines. TxDOT engineers, field crews, architects, and school district personnel can use this checklist and other guidelines to coordinate site

TABLE 2 Recommended Parent Drop-Off and Pickup Zone On-Site Stacking Length for Texas (4, 6)

School Type	Student Population	Loop Drive Stacking Length (linear feet) (m)
Elementary	Less than 500	400–750 (122–229)
	500 or more	750–1500 (229–458)
Middle	Less than 600	500–800 (153–244)
	600 or more	800–1600 (244–488)
High (*6*)	400–800	800–1200 (244–366)
	800–2500	1200–1500 (366–458)

FIGURE 5 Queue spillback from school site.

plan review efforts for existing or new school sites and to improve the safety and efficiency of school site access and traffic flow

RECOMMENDATIONS

- Increase promotion of the TxDOT Precious Cargo program to school districts and architecture firms to increase awareness and usage. This promotion can be accomplished by coordination with professional organizations such as the Council of Educational Facility Planners International and the American Institute of Architects.
- TxDOT district staff use the guidelines, good examples, and review checklist produced during the project to ensure a uniform approach to review of school site plans.
- Further research is needed regarding development of methods and techniques to increase the proportion of students getting to school by bus, walking, and biking versus private vehicles.

ACKNOWLEDGMENTS

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FIGURE 6 Another queue spillback from school site.

TABLE 3 School Site Plan Review Checklist Based on Recommended Guidelines

Guideline #	Review Question	Answer: Yes	Answer: No	Comments
1	Is the building setback a sufficient distance to provide adequate site storage?			
2	Is the school site located on a high-speed roadway? (If yes, please comment.)			
3	Is access provided from more than one direction to the immediate vicinity of the site (i.e., from at least two adjacent streets)?			
4	Is the school site situated where the road alignment provides good visibility?			
5	Are the physical routes provided for the basic modes (buses, cars, pedestrians, and bicycles) separated from each other on the site?			
6	Does overhead cover or soffit protect all primary building entrances for students?			
7	Have the school site and proposed plans been reviewed by the proper road agency?			
8	Are school buses going to be staged single-file right wheel to the curb in the loading zone?			
9	Is there adequate driveway stacking length for lining up cars on site (see Table 2)?			
10	Are students loaded and unloaded on the right side directly to the curb/sidewalk in the bus and parent loading zones?			
11	Are the short-term parking spaces located past the student loading area and near the building entrance?			
12	Is parent loading occurring only in designated zones? (If not, please note non-designated zones in comments section.)			
13	Are the student safety patrols and loading supervisors well trained and outfitted with reflective safety vests?			
14	Are traffic cones or other channelizing devices used within the site to minimize pedestrian/vehicle conflicts			
15	Are safe crosswalks with crossing guards provided on site and off site to minimize pedestrian/vehicle conflicts			
16	Are there standard and well-maintained sidewalks and/or a designated safe path leading to the school?			
17	Are there wider paved student queuing areas at major crossings and "stand-back lines" to show where to stand while waiting?			
18	Are facilities for bicycle access and storage provided at this campus?			
19	Do the school driveways conform to TxDOT design and access management guidelines for number, spacing, location, and layout?			
20	Does this school site have existing or planned left- or right-turn lanes? Do they meet existing TxDOT design guidelines?			
21	Do all site and regulatory signs and markings within the site comply with the *Manual on Uniform Traffic Control Devices*?			

served on an advisory panel, for their assistance and guidance: Mark Cantebury of the Keller Independent School District; Larry Colclasure of TxDOT, Waco District; Wade Odell of the TxDOT Research and Technology Implementation Office; Craig Reynolds of BRW Architects; Steve Taylor of Carter & Burgess; and Scott Young of the City of Frisco. The research team thanks the school principals and other district administrators who allowed researchers access to their campuses to observe and collect data.

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Bus or Car?

The Classic Choice in School Transportation

Tori D. Rhoulac

School-related traffic congestion causes increased commuter travel times in many communities because of high volumes of passenger cars on school campuses that often queue onto adjacent streets. To change modal choices and behaviors ultimately to prompt a decrease in this recurring congestion, research was done to gain a better understanding of the student, household, and trip attributes and behaviors that influence school transportation mode choice for students living beyond walking distance of school. When home-to-school distances make nonmotorized modes infeasible, families typically must choose between the automobile and the school bus for travel to and from school. School transportation mode choice models were developed to estimate morning and afternoon modal split between these two modes for a North Carolina school district. The factors that exhibited statistical significance in estimating mode choice for kindergarten to eighth-grade students included the total number of students in those grades living in a household, student grade, household income, and subjective variables that attempted to quantity the convenience of each modal alternative and parents' perceptions of modal safety. Two models were developed because the variables related differently to morning and afternoon school trip mode choice. In comparison with traditional mode choice models, the school transportation mode choice models developed as part of this research exhibit several similarities as well as distinct differences. Household income, for example, was found to be inversely proportional to the probability that a student will travel by automobile for morning or afternoon school trips.

Mode choice modeling is an essential component of the transportation planning process. This process, known as travel demand forecasting, comprises four primary steps—trip generation, trip distribution, mode choice, and network assignment. Trip generation determines the expected number of trips that will result from traffic generators, such as the home, workplace, and retail core. Trip distribution assigns an origin and destination to each trip, and mode choice models assign trips to the available modes. In the final component, network assignment, trips are assigned to specific links (i.e., roadways, bikeways, sidewalks) in the appropriate network.

The focus of this paper is mode choice modeling, which in the majority of cities, municipalities, and regions of the United States involves only two choices: transit or automobile. Although pedestrian and bicycle are other customary modes of transportation, they are often omitted in analyses because of the negligible quantity of these trips compared to transit and automobile modes. As traffic conditions worsen on roadway networks across the country, automobile trips remain the majority, with transit ranking a far second. Of the 2.7 trillion vehicle miles traveled in the United States in 2000, transit accounted for 45.1 billion passenger miles, with an average occupancy of 12 persons per vehicle (*1*). Mode choice modeling research has determined that the primary influences on a commuter's choice of mode are travel time, travel cost, and some measure of convenience or relative attractiveness of the modal alternatives. In response to mode choice modeling results, numerous strategies have been developed to increase the attractiveness of modes other than the automobile in an effort to decrease vehicular demand on roadways and associated congestion and other negative effects. The classic choice of car or bus thrives as commuters decide between the perceived convenience of their personal automobiles and the alternatives provided by transit buses and rail.

PROBLEM IN SCHOOL TRANSPORTATION

Each year in the United States, more than 5 billion student trips are made by using the school bus (*2*). School bus trips outnumbered the total transit bus trips in 2000, when transit bus ridership exceeded 5 billion passenger trips nationally, because school bus ridership is counted 5 days a week, approximately 10 months a year, whereas transit trips are tallied 7 days a week, 12 months a year (*3*). Millions of student trips are made daily by other school transportation modes, such as walking, biking, driving automobiles, and riding day care program vans, which suggests that school trips are significant contributors to peak period travel patterns in the United States. With school trips making up a relatively large quantity of peak period trips, a mode choice modeling effort applied specifically to school trips may be a beneficial step toward determining well-founded approaches to easing school-related traffic congestion on roadway networks.

Normal school transportation hours are from 6:00 to 9:00 a.m. and 2:00 to 5:00 p.m., whereas the generally accepted peak travel periods for commuter traffic are 7:00 to 9:00 a.m. and 4:00 to 6:00 p.m. in most areas. Although afternoon school trips do not fully coincide with the typical evening commuter peak, both the morning and the afternoon school travel peaks cause local traffic congestion problems in many communities. Traffic queues from school driveways and parking lots overflow onto adjacent streets because of high demand and inadequate supply of vehicle storage on school campuses. The problem is of such severity that in North Carolina, the state department of transportation (NCDOT) established the Municipal and School Transportation Assistance Group in its Congestion Management Section to deal with school-related traffic congestion problems. This group participated in a school campus circulation study in 2002 in which student loading and unloading procedures and vehicle queuing patterns were observed at 20 schools in eight North Carolina counties. The results further substantiated the impact of school trips on traffic

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patterns as about 50% of the schools experienced queues in the afternoon that exceeded their on-campus vehicle storage space, causing spillback onto the adjacent street; many of these queues began before the dismissal bell rang (*4*). A subsequent impact of such recurring traffic congestion is the disruption to normal traffic patterns, so that it becomes necessary to install turn lanes or widen roads in an attempt to enhance mobility in areas adjacent to schools.

"Nationwide, there appears to be a systemic modal shift from school transportation modes that are relatively safe to modes that are causing operational and safety problems in and around school areas" (*5*). Only 25% of all student trips nationally are made by using the school bus (*2*), although school buses are characterized as "the safest form of transportation for children" (*6*). Between 1991 and 1999, school bus crashes accounted for only 4% of all student injuries and 2% of all student fatalities in the United States. Passenger vehicles, including cars, light trucks, sport utility vehicles, and vans being used for school transportation, accounted for 84% of all injuries, 75% of all fatalities, and 59% of all trips (*2*). Given these statistics, why do so few students use the school bus service? One reason may be that "parents and students often do not consider the associated risks [of a travel mode] and choose or encourage the use of school travel modes for reasons apart from maximizing safety or minimizing risk [like] convenience, flexibility, or cost savings" (*7*). This paper presents and discusses factors that have been determined to influence choice of mode for school trips.

RESEARCH OVERVIEW

The fundamental objective for the school transportation mode choice research was to develop, calibrate, and validate a school transportation mode choice model for a selected school district with the intent of inferring the results to similar school districts nationwide. The scope of the project was determined by establishing the following research constraints:

- A reduced set of three modal alternatives was considered: nonmotorized (bicycle and pedestrian), school bus, and automobile. This decision was made primarily because of small sample sizes for other modal alternatives.
- Data were collected in North Carolina, where many resources were available, including the School Transportation Group, dedicated solely to multimodal school transportation issues, and the NCDOT Municipal and School Transportation Assistance Group. The organization of the public school transportation system also favored North Carolina because of the regular reporting required by the state of each school district on pertinent school bus operational statistics.
- Public school students in kindergarten through eighth grade (K–8) were the focus. Private school policies differ concerning the provision of bus transportation, as do many preschool programs, so limiting the study to public school students would ensure that all students were governed by the same transportation policies. Also, preliminary research showed that high school students (grades 9 through 12) exhibit notably different mode choice patterns than elementary and middle school students because the driving mode becomes available around the age of 16. The decision was made to collect further data from only those students in grades K–8, to create a data set with the same modal alternatives.
- Only those school trips occurring during normal school transportation hours were included in the analysis.

MODEL DEVELOPMENT

Data Collection

Mail-out–mail-back household surveys were used to collect data for mode choice model development in accordance with two primary objectives: (*a*) to collect data on individual student mode choice and associated student and household characteristics and (*b*) to assess the parent's perception of problems associated with the school bus service and other available school transportation modes. (Copies of the survey can be obtained by contacting the author.) The survey, which achieved a response rate of approximately 25%, involved just fewer than 800 K–8 students of the Wake County, North Carolina, public school system (WCPSS).

Statistical tests, such as the Wilcoxon two-sample test, were conducted to determine the effects of nonresponse bias on the sample. Results suggested that students from higher-income households were overrepresented, as may have been expected. Further comparison of sample statistics and known population parameters, however, led to the determination that adjustments to the data set would not be necessary because several sample values were statistically equivalent to population parameters, including the morning school bus modal spilt. The sample was therefore considered representative of families in the WCPSS.

In addition to use of a representative sample for model development, there was concern about the range of inference that would be possible for the study results, given the county where data were collected. The North Carolina Department of Public Instruction, Transportation Services Division, characterizes school transportation primarily on the school bus usage statistics of total expenditures, number of students transported, and number of buses for each school district. Average values for the state in the 2002–2003 school year were $2,504,445 in total expenditures, 7,150 students transported, and 134 buses. Wake County is the second-largest school district in the state and reports values of $27,813,014, 55,441 students transported, and 731 buses. As such, Wake County may not represent the typical school transportation conditions for a North Carolina district. Instead, the models developed through this research are expected to apply to the typical large, suburban school district in the southeastern United States, like Charlotte–Mecklenburg in North Carolina, where 1,132 buses were used in the 2002–2003 school year to transport more than 67,000 students, or DeKalb County Schools in Georgia, where more than 900 buses are reported to be in use, providing transportation for more than 78,000 students. A larger-scale study will be recommended to confirm the range of transferability for the models developed.

Survey Results

Student Choice of Mode

Mode choice by grade level and time of day is given in Table 1, categorized according to the four primary school transportation modes. High school students are included in Table 1 because these data were obtained in preliminary research efforts that involved grades K–12. School bus and automobile are clearly the two primary modes chosen for school trips among all grade levels. Nonmotorized modes (pedestrian and bicycle) accounted for less than 5% of all morning and afternoon school trips. Travel by contracted vans and transit made up about 1% of the morning and afternoon trips. Given the small sample

TABLE 1 Mode Choice by Student Grade Level

Grade Level	School Travel Mode: Automobile	School Bus	Van–Transit	Ped.–Bike	Total
Elementary—a.m.	42%	52%	1%	5%	100%
Middle—a.m.	34%	63%	0.3%	2.7%	100%
High—a.m.	63%	35%	0.5%	1.5%	100%
Total—a.m.	46.3%	50%	0.6%	3.1%	100%
Elementary—p.m.	30%	62%	2%	6%	100%
Middle—p.m.	17%	77%	1%	5%	100%
High—p.m.	54%	43%	1%	2%	100%
Total—p.m.	33.7%	60.7%	1.3%	4.3%	100%

sizes of students that use contracted van and public transit, these two modes were omitted from model development analyses.

High school students exhibited significantly different mode choice patterns from other grade school students, most likely because of the availability of driving as a modal alternative. Nearly 25% of high school students drive to and from school. Another 20% ride with a family member other than the parent or with friends who drive, adding to the total number of high school students transported by automobile for school trips. Research conducted in the early 1990s studied trip generation rates for North Carolina high schools and found that North Carolina high schools generate more automobile trips than the national average (*8*). The research scope was therefore narrowed to students in grades K–8, whose available model alternatives are the same.

Perceived Modal Safety

Parents' perception of the safety of a mode was considered for its possible impact on the mode a child uses for school trips. Survey results found that many parents believe driving their children to school is safer than having their children ride the school bus. Fatality statistics support the contrary, confirming that "motor vehicles are the leading cause of death for school-age children" (*6*). Of the 800 school-age children killed in the United States during normal school transportation hours between 1991 and 1999, automobiles accounted for 75% of the fatal crashes, and approximately 55% of these fatal crashes involved a teenage driver (*7*). Still, most parents believe that their children are more secure when driven by a parent, and this is likely why the majority of responding parents indicated that they believe driving their children to school was safest. Nearly 65% of parents rated the automobile as "very safe," whereas only 35% rated the school bus as "very safe."

Home-to-School Distance

Grade school transportation in North Carolina involves three groups of students, each with a unique set of available modal alternatives. According to provisions in North Carolina state law, students live either in a no-transport zone, a base attendance area, or outside the base attendance area. A school district is not required to provide school bus transportation for students living within a 1.5-mi radius of school, unless potentially hazardous conditions exist, such as a railroad crossing (*9*). Within this no-transport zone, two modal alternatives are available: nonmotorized and automobile. The second group of students lives inside the base attendance area for a school, beyond the no-transport zone. School bus service is provided within this boundary, but nonmotorized modes are not considered feasible because of lengthy home-to-school distances. The available modes are therefore school bus and automobile. The final group of students must use an automobile or public transit for school trips because they live outside the base attendance area for their school and school bus service is not provided nor are distances feasible for using nonmotorized modes.

To characterize and compare influences to mode choice in school transportation and traditional, commuter transportation, only those students in the base attendance area, outside the no-transport zone, were involved in modeling. Less than 2% of WCPSS students in the K–8 sample live outside the base attendance area for their respective schools. Approximately 10% of the students live in a no-transport zone, but the factors influencing decisions to walk or cycle to school, such as availability of sidewalks or trails, vary greatly by community. Nonmotorized mode choice models should include a pedestrian environment variable, which takes into account factors like sidewalk availability, ease of street crossing, street connectivity, and availability of bicycle infrastructure (*10*). The task of creating a single model to estimate the modal split for the many no-transport zones that make up a school district was beyond the scope of this research because of the large variability in walking and biking conditions. Therefore, only those students with a choice of automobile or school bus for school trips were modeled. A mode choice model that includes nonmotorized modes has been developed, however, although the emphasis is on school siting implications (*11*).

School Bus Service Convenience

More than 70% of responding WCPSS parents characterized bus arrival as "on time." Overall, WCPSS school bus service appears to be convenient for users in terms of punctuality. Still, less than 50% of the WCPSS student population used the school bus service. This supports further the need to determine those factors that most contribute to choice of mode in school trips.

Spatial Analysis

To obtain socioeconomic data for inclusion in model development, surveyed parents were asked to identify the zip code boundary in which they lived. By using geographical information systems software, the 105 Wake County census tracts from 1990 and their associated median household incomes were merged to form the 32 zip code boundaries. Median income values by census tract were averaged to obtain median household incomes by zip code boundary. This value was used as a surrogate variable for actual household income.

Defining Variables

The following variables were formulated from survey and geographic information system data and considered in model development:

- AMMode—binary variable representing the mode a student uses for travel to school in the morning (0 = school bus, 1 = automobile);
- PMMode—binary variable representing the mode a student uses for travel from school in the afternoon (0 = school bus, 1 = automobile);
- K8HH—total number of children in grades K–8 in a household;
- Income—average median household income for the zip code in which a student lives ($\times 10^4$);
- Gender—binary variable to represent male or female students (0 = male, 1 = female);
- Distance—student's home-to-school travel distance as perceived by the parent;
- Grade—integer ranging from 0 for kindergarten to 8, indicating a student's grade for the 2002–2003 school year;
- SafeMode—numerical representation of the mode a parent perceives to be most safe [0 = school bus, 1 = nonmotorized, 2 = motorized (i.e., the parent believes that the school bus and automobile tie for most safe, outranking a nonmotorized mode), and 3 = automobile];
- AUConv—numeric value used to describe automobile convenience for a household based on student schedules, automobile ownership, and ability to chain trips and carpool (see Table 2); and
- SBConv—numeric value used to quantify whether parent work schedules, safety, or other concerns promote or constrain school bus usage (see Table 3).

The school bus and automobile convenience variables, although highly subjective, were included in the analysis in an attempt to quantify factors that might otherwise be accounted for only in the resulting model constant. For example, without collecting data on scheduled school bus arrival times for individual households, the school bus convenience term expressed whether the scheduled bus arrival times in the morning or afternoon affected the modal decision.

Linear Regression Analyses

Simple linear regression was the initial tool used to determine which variables were statistically significant in estimating school transportation mode choice. Data on 611 students living in the base attendance area for their school, outside the no-transport zone, were analyzed. By using 0 and 1 to represent the school bus and the automobile, respectively, binary linear regression analysis was performed. Results indicated that two mode choice models were needed because the variables found to significantly influence mode choice were different for the morning and afternoon cases. Table 4 displays the regression analysis results, considering only those variables exhibiting significance at a 95% confidence level.

TABLE 2 AUConv Variable Components

AU Convenience Components	Add One (+1)	Subtract One (−1)
Automobile ownership	If yes	If no
Carpool opportunity	If yes	If no
Trip chaining opportunity	If yes	If no
Extracurricular activities	If AU transport required	

Neither student gender nor home-to-school distance appeared to significantly affect school trip mode choice. In the morning case, only the subjective variables, SafeMode, SBConv, and AUConv, appear to influence mode choice. With AMMode included as a possible predictor variable for afternoon mode choice, AMMode exhibited significance, along with K8HH, Grade, Income, SafeMode, SBConv, and AUConv.

Forward and backward stepwise linear regression analyses considered not only the variables listed in Table 4 but also variable interactions and quadratic effects. Although several interactions exhibited significance in the estimation of morning and afternoon mode choice, and two quadratic terms were suggested for inclusion in the afternoon model, improvements to the models' overall fit were on the order of only one-hundredth to six-thousandths and were therefore considered inconsequential.

The morning model was adjusted, however, in an attempt to develop morning and afternoon models with similar explanatory variables. K8HH, Grade, and Income were added to the morning model and analyzed. The resulting model helps to explore the relationships between explanatory variables and choice of morning mode, compared and contrasted with those same relationships and choice of afternoon mode.

The final school transportation mode choice models are given in Equations 1 and 2. The dependent variable, $P(\mathrm{AU})_x$, represents the probability of a student using the automobile for the x = morning or afternoon school trip. The standard errors for each coefficient are given in Table 5.

$$\begin{aligned} P(\mathrm{AU})_{\mathrm{AM}} = {} & 0.338 - 0.028\mathrm{K8HH} + 0.009\mathrm{Grade} \\ & - 0.027\mathrm{Income} + 0.034\mathrm{SafeMode} \\ & - 0.253\mathrm{SBConv} + 0.056\mathrm{AUConv} \end{aligned} \tag{1}$$

where $R^2 = 0.49$, $\sigma^2 = 0.125$, and $N = 611$.

TABLE 3 SBConv Variable Components

SB Convenience Components	Add One (+1)	Subtract One (−1)
Scheduled arrival time	If parent work schedules require SB	If time does not fi household schedules
Bus stop concerns (including punctuality and behavior)	—	If yes
Bus operation concerns (including driver and student behaviors on board SB)	—	If yes

TABLE 4 Statistically Significant Contributors to School Transportation Mode Choice

Variable	a.m. Mode Choice Significanc	p.m. Mode Choice Significanc
Intercept	Yes	Yes
K8HH	No	Yes
Grade	No	Yes
Gender	No	No
Distance	No	No
Income	No	Yes
AMMode	—	Yes
SafeMode	Yes	Yes
SBConv	Yes	Yes
AUConv	Yes	Yes

$$P(\mathrm{AU})_{\mathrm{PM}} = 0.453 - 0.039\mathrm{K8HH} - 0.023\mathrm{Grade} - 0.053\mathrm{Income} + 0.185\mathrm{AMMode} + 0.024\mathrm{SafeMode} + 0.026\mathrm{AUConv} - 0.162\mathrm{SBConv} \quad (2)$$

where $R^2 = 0.46$, $\sigma^2 = 0.109$, and $N = 611$.

Logistic Regression Analysis

Traditional mode choice models use logistic regression, or logit models, to estimate modal split. Among the benefits of logistic regression is the inability to generate probabilities less than zero or greater than one. Linear regression was the primary analysis tool in the development of the school transportation mode choice models, with extreme values of zero and one assigned to the range of possible probabilities. Logistic regression, however, was also used to analyze the data for comparison with the linear models' results.

The basis for logistic analysis in the school transportation case was the difference in utility between the school bus and the automobile in order to estimate the probability that a student will use the automobile for school trips. The resulting logistic mode choice models were evaluated, along with the linear models, by using Brier scores (BS) to determine which would estimate the modal split for school trips most accurately. Brier scores are commonly used to measure the accuracy of probabilistic forecasts and are calculated by using Equation 3, where o is a binary observation equal to zero or one and p is the forecasted probability (*12*). The preferred Brier score is the lowest. Brier skill scores (BSS) are then computed by using the calculated Brier scores and the Brier scores of a reference probability value ($\mathrm{BS_{ref}}$); Equation 4 gives the formula. To compute the Brier skill scores for the school transportation mode choice models, mean values for the actual morning and afternoon mode choice were used as reference probabilities. Table 6 gives the Brier scores and Brier skill scores for the linear and logistic model results.

Although the Brier scores for the logistic-based models are slightly lower than those for the linear-based models, the difference is not significant. Therefore, because both effectiveness and simplicity are desired for the models, the linear models were selected. Validation tests confirmed the effectiveness of the linear models.

$$\mathrm{BS} = \overline{(o - p)^2} \quad (3)$$

$$\mathrm{BSS} = \frac{\mathrm{BS_{ref}} - \mathrm{BS}}{\mathrm{BS_{ref}}} \quad (4)$$

TABLE 5 School Transportation Mode Choice Models' Parameter Standard Errors

Model Parameter	a.m. Linear Model Standard Error	p.m. Linear Model Standard Error
Intercept	0.101	0.095
K8HH	0.021	0.019
Grade	0.006	0.005
Income	0.021	0.019
AMMode	—	0.038
SafeMode	0.012	0.011
AUConv	0.009	0.009
SBConv	0.013	0.016

Sensitivity Analysis Results

Sensitivity analyses were performed to observe how model outcomes change relative to changes in the explanatory variables. Each predictor variable was evaluated to determine the impact of slight changes in model parameters on school transportation mode choice. No change in the K8HH, Grade, or SafeMode variables prompted a change in mode, as evidenced by changes in resulting probabilities of a student using the automobile for school trips. Changes in automobile and school bus convenience, however, do prompt significant changes in mode choice. School bus convenience proves to have the largest effect, overall, on school transportation mode choice.

SCHOOL TRANSPORTATION MODE CHOICE AND TRADITIONAL COMMUTER MODE CHOICE

The fundamental theory of mode choice modeling, which has been used by transportation planners for decades, is this: "[T]he probability that an individual will choose a particular alternative is a function of the characteristics of the individual and of the overall desirability of the chosen alternative relative to all other alternatives" (*13*). In traditional mode choice modeling, travel cost, although a modal characteristic, represents an individual attribute—socioeconomic status or ability to pay. Other common variables in traditional mode choice models are trip characteristics, travel time, both in- and out-of-vehicle, and relative attractiveness, which is difficult to quantify and is usually accounted for in the model constant. Two of these variables are also significant for estimating modal split for school trips.

TABLE 6 Brier and Brier Skill Scores

Model	Brier Score	Brier Skill Score
a.m. linear	0.11	0.53
a.m. logistic	0.10	0.57
p.m. linear	0.11	0.47
p.m. logistic	0.10	0.49

Neither distance, which is inversely proportional to travel time, nor gender appears to have a significant impact on whether a student uses the automobile or the school bus mode for school trips. One might argue that each of the students whose data made up the model development database lives within a similar distance from the school, being outside the no-transport zone and inside the base attendance area. The range of perceived home-to-school distances represented in the data set, however, was quite substantial, extending from less than 1 mi to more than 20 mi. Distance was, therefore, a formidable variable in the model development process that simply did not exhibit significance in estimating school trip modal split.

Household socioeconomic attributes were captured by average median household income according to zip code boundary in the school trip models. Perhaps the most interesting variable relationship involves this model parameter. In both the morning and afternoon school transportation mode choice models, income is assigned a negative coefficient, suggesting that the higher a household's income, the more likely the students of that household are to ride the school bus. In traditional mode choice models, travel cost is thought to be directly proportional to socioeconomic status or income. The higher an individual's income, the more likely he or she is to use a more costly mode of travel. Intuitively, one would think that a family with a higher income would have more opportunities to drive their children to school, because of higher vehicle ownership rates, more flexible work hours, and so forth, but these data suggest that higher incomes make a student less likely to ride in an automobile for morning or afternoon school trips. This is compounded by the fact that school bus transportation is fully subsidized and no direct cost is assessed to the parent. School siting processes may have an effect because more affluent neighborhoods, where families of higher socioeconomic status typically live, are placed farther from schools; thus these students are more likely to ride the school bus because of the relatively long distance to their respective schools. However, model development analyses suggest that home-to-school distance does not have a significant impact on choice of mode for school trips. Socioeconomic status, as evidenced by average median household income, does exhibit statistical significance in estimating school trip mode choice, but in an unexpected way. According to the results of this research, the higher the income of a student's household, the more likely he or she is to ride the school bus for school trips.

A second similarity between school trip and traditional mode choice models is the importance of modal convenience. To quantify convenience for the school transportation models, so that convenience is represented by more than a constant so that variable relationships could be explored, subjective variables SBConv and AUConv were defined and included in model development analyses. The components of these variables are explained in detail in Tables 2 and 3. Results confirmed what traditional mode choice models have found to be true—mode choice is highly dependent on relative modal convenience. Both SBConv and AUConv were significant variables in the morning and afternoon models. School bus convenience, which accounted for parental safety concerns on board school buses and at bus stops, was determined to contribute most to school trip mode choice in the morning and ranked a close second to AMMode in the afternoon. Variable coefficients were intuitively reasonable with SBConv being negative and AUConv positive, meaning that larger values of school bus convenience made a student less likely to use the automobile for school trips, whereas larger values of automobile convenience made a student more likely to use the automobile.

The only other subjective variable, SafeMode, also exhibited significance in both morning and afternoon models. Parents were asked to rank the four most common school transportation modes (school bus, automobile, bicycle, and pedestrian) in order of safety; the mode perceived to be most safe by each parent was quantified and included in model development analysis. Results confirmed that the children of parents who believe the automobile to be most safe are more likely to ride in an automobile for school trips, whereas students whose parents believe the school bus or nonmotorized modes to be most safe are more likely to ride the school bus.

Other explanatory variables were the total number of children in a single household in grades K–8 (K8HH), student grade, and AMMode in the afternoon model. For morning and afternoon school trips, students in households with more than one child in grades K–8 are more likely to ride the school bus. This is reasonable given the magnitude of parental safety concerns and perceptions as evidenced in the model results; parents appear to feel more comfortable if their child can wait at the bus stop and ride the school bus with other siblings.

The impact of student grade differs in the morning and afternoon cases. In the morning, students in higher grades are more likely to ride in an automobile, the Grade variable having a positive coefficient. Students in higher grades are more likely to ride the school bus in the afternoons, however, according to the negative coefficient assigned to this variable in the afternoon model. This may be explained by the frequency of before-school activities or quantity of instructional support materials, like large-scale projects, for middle school students, relative to elementary, that would require arriving at school early or that would make riding a school bus infeasible. Another reason might be that parents are more likely to allow older children to ride the school bus and be at home by themselves in the afternoon, whereas parents who would rather a younger child have supervision usually will have the child picked up directly from school, typically in an automobile.

Finally, morning mode choice is a significant explanatory variable of afternoon mode choice, meaning that a student is most likely to use the same mode for morning and afternoon school trips. This is intuitively reasonable, especially from the perspective of the parent with school bus safety concerns. If that parent believes the school bus to be unsafe for travel to school in the morning, he or she will likely believe the same for travel home from school in the afternoon.

RESEARCH CONCLUSIONS AND RECOMMENDATIONS

Although other issues, such as model inference and transferability, were investigated in this research, the intent for this paper was to introduce the factors that exhibit statistical significance in estimating school transportation modal split between the automobile and the school bus and to compare these factors with traditional, commuter mode choice model variables. There are both important similarities and differences in the variables and variable relationships of both model types.

Conclusions

- Origin-to-destination distance (or travel time), although significant in traditional mode choice models, does not appear to influenc choice of mode for school trips. Perceived home-to-school distances were gathered in the data collection survey because decisions are made on the basis of perceptions, and obtaining actual distances for the more than 600 students involved in the analysis was infeasible. This perceived distance variable did not exhibit significance in estimating morning or afternoon modal split for school trips.

- Socioeconomic status, as represented by average median household income, is inversely proportional to the probability that a student will be driven in an automobile for school trips. The models suggest that students from higher-income homes are more likely to use the school bus service, which is free of direct charge to the parent. One proposed explanation relates to school siting and the longer home-to-school distances that may be characteristic of more affluent neighborhoods, but this is not supported by results, which suggest that distance is not a significant predictor of mode choice. Additional research would greatly benefit this discussion.
- Relative modal convenience is important in both commuter and school trip mode choice. This is apparent in the magnitude and sign of the coefficients for the subjective variables that attempted to quantify the convenience of the automobile and school bus service. Increasing the convenience of the school bus service is then critical to prompting a modal shift to the school bus to decrease instances of recurring congestion on and around school campuses.
- Parental perception of modal safety is paramount. If there is to be systemic change resulting in modal shifts and decreased school-related traffic congestion, addressing parents' perceptions about the relative safety of the school bus and automobile is critical. Most parents believe the automobile to be a "most safe" means of transporting children to and from school, although injury and fatality statistics suggest otherwise.
- Linear mode choice models can be used to estimate aggregate modal shares. Although logistic models are conventionally used for estimation of modal share, linear models also produced effective estimates of school transportation mode choice.

Recommendations for Future Research

Further study of the important decisions and behaviors that affect mode choice for school trips would be beneficial to the school transportation, traffic engineering, and urban planning communities. The results of such research could enhance congestion mitigation, air quality, and similar efforts in many communities. Other issues, beyond the scope of this study, that might be considered in future research include the following:

- Verification of model applicability in other large, suburban school districts in the southeastern United States;
- Evaluation of the decisions and behaviors that govern mode choice and automobile safety for high school students, including an assessment of the injuries and fatalities reported of automobile collisions involving teenage drivers during normal school transportation hours;
- Formulation of school bus and automobile convenience terms that involve only the components that indicate a significant relationship with school trip mode choice and assigning different weights, as appropriate, to those components;
- Investigation into the relationship between school trip mode choice and income, since income demonstrates inverse proportionality to the probability that a student will use the automobile for school trips.

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Rural School Vehicle Routing Problem

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The school bus routing problem traditionally has been defined in an urban context. However, because of the unique attributes of the problem in rural areas, traditional heuristic methods for solving the problem may produce impractical results. In many cases, these characteristics also provide the opportunity to investigate what size and mix of vehicles, whether large or small buses, conforming vans, or other modes, are most efficient. In addition, these vehicles may be further differentiated by the presence of equipment for transporting students with special needs. To address this situation, a mathematical model of the problem was constructed and a new heuristic was developed. This heuristic consists of two parts: constructing the initial route and then improving it by using a fixed tenure tabu search algorithm. This rural routing heuristic, in addition to several existing ones, is then applied to a randomly generated school district with rural characteristics. For the relevant measure, a function of student ride time, the new heuristic provides a set of routes superior to those produced by existing methods. Because ride times produced by the new heuristic are lower than those for routes generated by existing methods, the likelihood of injury to students may decrease. Also, with the cost of operation for each route calculated in dollars, a comparison of solutions in financial, as well as temporal, terms is possible.

The efficient routing of vehicles is one of many challenges facing rural school districts. Declining enrollments and a dwindling tax base further intensify the already difficult duties of administrators that include ensuring the safe transportation of students to and from school in an economical and timely manner. A large geographic area and small student population are often combined with an aging and inappropriately sized fleet, consisting of large, underutilized, high-cost buses. In many cases, it may not be efficient to use buses at all, and instead conforming vans or smaller vehicles may form a portion of the fleet. This results in a problem much different from that defined in the traditional school bus routing literature. Consequently, the situation is referred to as the rural school vehicle transportation problem. The unique characteristics of the rural school vehicle transportation problem are discussed, and a general model is constructed. Then the new heuristic is applied to a simulated school district, and the results from it and other common methods are compared. Because the heuristic focuses on minimizing a function of student ride times, the likelihood of injury to students may decrease.

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UNIQUE ATTRIBUTES OF RURAL SCHOOL VEHICLE TRANSPORTATION PROBLEM

The school bus routing problem, and the related assignment and scheduling problems, traditionally have been approached from an urban standpoint. Because a number of important differences exist between the problem in a rural as opposed to an urban setting, alterations and additions must be made to the conceptual framework in the literature to adequately address the problem. Also, because the computational size of the rural problem is considerably smaller than its metropolitan counterpart, it is possible to incorporate other aspects of school vehicle management beyond that in previous studies.

The most noticeable attributes of the rural school vehicle routing problem are the small numbers of students, stops, vehicles, transfers, and schools, and in many cases the longer lengths of routes. In many small or rural school districts, there are fewer students to be transported than the number of buses in large ones. This difference in magnitude makes the problem much easier to solve computationally because one can make use of a number of existing heuristic approaches. In fact, because of its small size, a solution to the rural school vehicle routing problem found by hand may be optimal.

As work in the field has noted, providing transportation manager flexibility in choosing among alternatives, as well as the ability to impose route constraints, is important (*1*). Firsthand knowledge of the system cannot be replaced, although it may be complemented, by an algorithm. This is especially true in rural districts where unique road attributes may be known only by members of the local community. Thus, it may not be efficient for implementing technologically advanced solutions in situations in which a great deal of effort must be used to tailor fit computer-generated routes to relatively small, real-world problems.

Although the literature has focused on the urban school bus routing problem, some studies have identified and addressed specific issues faced in rural areas. Foremost is the importance of the routing problem relative to that of scheduling in rural areas, where in many cases vehicles never reach full capacity (*2*). In the case of a single school in a district, the scheduling problem is nonexistent. Some work in the field has included separate routing algorithms for urban and rural scenarios (*3, 4*). The assignment problem in rural settings is also much simpler because of both the small number of students and the fact that door-to-door service in outlying areas is common, eliminating the need for solving an assignment problem.

Fleet composition typically is more important in rural areas than urban ones, because vehicle capacity constraints rarely bind in rural settings (*1*). The choice of which bus or other vehicle, such as a conforming van or car, to use on which route becomes important. In urban areas, the large number of densely located students ensures that buses will reach capacity, and the question becomes one of how many large buses are needed to meet a district's need. Note that rural school districts typically own and operate their fleets, whereas urban districts often lease their fleets or contract out their entire transportation operation to vendors.

Time constraints may need to exhibit some flexibility in remote areas. In a number of parts of the country, the time needed to travel directly from home to school may exceed a time constraint that in most other areas would appear appropriate. In urban districts, however, short routes are the rule, and the time constraint seldom binds.

The primary difference between urban and rural routing problems arises because of geography, for which cost-minimizing algorithms provide impractical results. For example, a traditional cost-minimizing algorithm may find an optimal route servicing three students that picks up an in-town student, a rural student, and then another in-town student before reaching the school. However, the first child will spend a relatively long time in route, which may detract from the real-world value of the algorithm. In a general rural context, such algorithms may generate extremely high ride times and increase the likelihood that the students will be riding to or from school when an accident occurs.

In rural as in urban areas, students with special needs usually are transported separately in appropriately equipped buses. This frequently results in ride times that are much longer than those for other students, because students who live long distances apart must be assigned to the same route to effectively make use of the available equipment and deliver students in time for school. However, in rural school districts spread over large areas, this may lead to rides that are much longer than an hour.

MATHEMATICAL MODEL

The system consists of a network, students, vehicles, and schools. The directed network, G, consists of a set N of n nodes, which may be student homes, schools, or intersections, and a set A of m undirected arcs, which are the streets and roads traversed by vehicles transporting students to and from school. In this paper, a student's home is the equivalent to his or her stop, as is often the case for rural routes. Alternatively, this can be seen as the location and assignment portions of the problem being previously solved.

There is a set S of s students, each of whom is assigned a stop, $\pi(s)$, and school, $\sigma(s)$, and occupies one unit of space in his or her assigned vehicle. The school district owns a set V of v heterogeneous vehicles, which differ by age, capacity, and fixed and variable operating costs. Vehicles may be further differentiated by the presence of equipment to transport students with special needs. These vehicles are grouped together by these attributes into K types.

The mathematical representation of the problem is described as follows:

$$f_1 = \sum_{k=1}^{K}\sum_{v=1}^{V} f_k x_k^v + \sum_{k=1}^{K}\sum_{v=1}^{V}\sum_{i=0}^{n}\sum_{j=0}^{n} c_{i,j}^k x_{i,j}^{v,k} \tag{1}$$

$$f_2 = \sum_{\pi=1}^{\Pi}\sum_{i=0}^{n}\sum_{j=0}^{n} t_{i,j} x_{i,j}^{\pi} \tag{2}$$

minimize (f_1, f_2)
subject to

$$\sum_{k=1}^{K}\sum_{i=0}^{n} x_{i,j}^k = 1 \qquad \text{for } j = 1,\dots,n \tag{3}$$

$$\sum_{i=0}^{n} x_{i,p}^k - \sum_{j=0}^{n} x_{p,j}^k = 0 \qquad \text{for } k = 1,\dots,K;\ p = 1,\dots,n \tag{4}$$

$$\sum_{s=1}^{S} o_{i,j}^{s,v} \le c^v \qquad \forall v \in V;\ i,j = 1,\dots,n \tag{5}$$

$$\sum_{i=0}^{n}\sum_{j=0}^{n} t_{i,j}\delta_{i,j}^s \le t_{\max} \qquad \forall s \in S \tag{6}$$

where

f_k = fixed cost of owning a vehicle of type k,

$$x_k^v = \begin{cases} 1 & \text{if vehicle } v \text{ is of type } k \\ 0 & \text{otherwise} \end{cases}$$

$c_{i,j}^k$ = cost of a vehicle of type k to travel between nodes i and j;

$$x_{i,j}^{v,k} = \begin{cases} 1 & \text{if vehicle } v\text{, of type } k\text{, travels between nodes } i \text{ and } j \\ 0 & \text{otherwise} \end{cases}$$

$t_{i,j}$ = time needed to travel between nodes i and j

$$x_{i,j}^{\pi} = \begin{cases} 1 & \text{if route travels between nodes } i \text{ and } j \text{ after visiting stop } \pi \\ 0 & \text{otherwise} \end{cases}$$

$$x_{i,j}^{k} = \begin{cases} 1 & \text{if a bus of type } k \text{ travels between nodes } i \text{ and } j \\ 0 & \text{otherwise} \end{cases}$$

$$x_{i,p}^{k} = \begin{cases} 1 & \text{if a bus of type } k \text{ travels between nodes } i \text{ and } p \\ 0 & \text{otherwise} \end{cases}$$

$$x_{p,j}^{k} = \begin{cases} 1 & \text{if a bus of type } k \text{ travels between nodes } p \text{ and } j \\ 0 & \text{otherwise} \end{cases}$$

c^v = capacity of vehicle v;

$$o_{i,j}^{s,v} = \begin{cases} 1 & \text{if student } s \text{ occupies a seat on vehicle } v \text{ between nodes } i \text{ and } j \\ 0 & \text{otherwise} \end{cases}$$

$t_{\max}$ = maximum student ride time;

$$\delta_{i,j}^{s} = \begin{cases} 1 & \text{if student } s \text{ travels from node } i \text{ to node } j \\ 0 & \text{otherwise} \end{cases}$$

The objective for the rural school vehicle routing problem is to minimize the financial cost of transporting students to and from school in a timely matter, as defined by Equations 1 and 2. Although there is no method with which to simultaneously optimize the functions, one could make use of a subjective weighting function (*5*). Equation 1 and all constraints except that regarding time closely follow the seminal work in fleet mix and vehicle routing by Golden et al. (*6*), except that both fixed and variable costs vary by vehicle. It is also assumed, without loss of generality, that vehicles are purchased and not leased.

Whereas a change in the composition of the fleet will result in additional revenue or expense as vehicles are purchased or sold, this aspect of the problem is ignored, because we focus instead on annual operating costs. Also, because fixed costs are accrued on the basis of ownership, not use, the fleet may need more vehicles than are scheduled for daily routes, as would be the case for special trips or to ensure availability of vehicles in the case of breakdown. However, the number and types of vehicles for these purposes are not addressed in this paper. Whether these vehicles are used is unimportant; the fixed cost of ownership would arise regardless.

The first term in Equation 1 calculates the fixed cost of operating by summing the fixed cost of a type k vehicle across all vehicles and types. The second term determines the variable cost of fleet operations by summing the cost of a type k vehicle traveling between nodes i and j across all arcs, vehicles, and types. The second objective function in Equation 2 addresses the time aspect of the routing portion of the problem, similar to the service objective (*7*). However, here the sum of the time it takes for a vehicle to reach its destination from each stop is minimized, as opposed to weighing the function by the quantity of students picked up at each spot (*7*).

Constraints 3 and 4 ensure that each stop is visited only once and that a vehicle that arrives at a given stop also leaves from it. There are also capacity and ride-time constraints (*5, 6*). Although not included in the model, in many cases there is also a financial constraint that affects how many vehicles may be purchased or sold during a certain period.

RURAL ROUTING HEURISTIC

The rural routing heuristic is constructed to address the impractical routes that are often generated when traditional heuristics are applied to large areas with small numbers of students. The rural routing heuristic aims to minimize Constraint 2 subject to its constraints in a computationally efficient manner. The algorithm consists of two phases. The first produces an initial route, which is then improved in the second phase. The first phase begins by sorting the homes by distance from the school in descending order. The furthest location is designated the start of Route 1 and is eliminated from the list. Next, all stops that are in the shadow of the selected location are removed, and the next location on the list is selected. The shadow algorithm first determines the angle formed between each stop and the school. The shadow of a selected location is defined as the space whose angle with the school lies within a range above and below that of the angle between a reference location and the school. This is repeated until all routes, the number of which is predetermined before the algorithm is run and which may coincide with the number currently in use, have been assigned a starting location. This may result in some routes violating capacity constraints, a situation remedied in the second phase. This violation occurs when the number of riders on a given route exceeds the capacity of the largest vehicle. Time constraints may also be imposed, but they are ignored in the first phase as well.

Once all starting points have been found, the cost of inserting all remaining homes between existing stops and the school is calculated, and this with the minimum value is added in the appropriate location and that home is removed from the available homes list. This is repeated until all homes have been assigned to routes. Although this technique will seldom provide optimal solutions, it is successful for clustering stops that likely will be assigned to the same route, and it positions those homes that are furthest from the school as starting points, which likely will be the case when the final solution is determined. As a result, it defines a good starting point for the improvement algorithm used in Phase 2.

Phase 2 consists of using a tabu search algorithm with a 1-interchange mechanism and fixed tenure (*8*). A tabu search algorithm stores previous generated routes to prevent the program from cycling. In practice, this often takes the form of changes in the route ordering becoming temporarily fixed, by placing them on the "tabu" list, for a predetermined number of iterations. For the rural school vehicle routing problem, the cost function on which comparisons are made takes the form of Equation 2, rather than one comparing total distance traveled or a similar variant, which would normally be the case. Also, because of the simple nature of the rural school vehicle routing problem, the powerful apparatus provided by the tabu search algorithm may not be fully utilized in some instances and likely could be replaced with a more primitive but equally effective improvement algorithm, such as a Lin 2-opt, which alters the initial route by changing the order of two stops (*9*).

A set of routes is retained when one of three events occurs. First, a set is retained if the time travel measure is less than that of the current set. Otherwise, if any current route violates either the capacity or the time constraint and its newly generated replacement route decreases the magnitude of the violation or eliminates the violation altogether, it is retained.

SIMULATION

To test the effectiveness of the rural routing heuristic, a simulation is run. A two-dimensional space, 25 mi^2, is populated with 40 homes, each of which has between one and five randomly generated riders, and a centrally located school is provided. In total, 131 students are generated. It is assumed that five large buses, with a per-unit capacity of 40 students, service the area. Each large bus costs $4,500 per year in fixed costs and $1.20 per mile. The school district may also procure small buses, which can transport 20 students at $3,500 per year and $1.12 per mile. Either size bus travels at 40 mph across any arc. These values approximate those that prevail in rural school districts in the upper Midwest. It is assumed that each student, regardless of grade, occupies one seat on the bus. This coincides with existing practice in areas with harsh winters, where appropriately dressed children comfortably fit two to a seat.

It is not necessary, however, that the vehicles used by the districts be traditionally configured buses. Any motorized vehicle that conforms to regulatory standards and whose fixed and variable costs of operation and capacity are known can be used. Nor is the problem limited to comparing only two vehicle types, as done in this scenario for simplicity.

With this information, three commonly used algorithms as well as the rural routing heuristic are applied. The first is a Lin 2-opt applied to a nearest-neighbor route through all points (*10*). Next, a modified Clarke and Wright technique that allows for reversals and makes use of a Lin 3-opt for route improvement is applied (*11*). Finally, the more modern randomized location-based heuristic (rLBH) is used (*1*). For each of the three existing algorithms, two scenarios are run. The first has a time constraint with a maximum of 60 min of ride time for any student and 75 min for the second scenario.

TABLE 1 Simulation Results

	Fleet Composition	Annual Cost ($)	Total Daily Travel Time (min)	Sum of Times from Stops (min)	Time Constraint (min)
Lin 2-opt	4 large, 1 small,	94,573	632	1,139	60
	4 large	93,010	626	1,561	75
C&W	3 large, 2 small,	99,981	674	1,421	60
	4 large	90,437	602	1,490	75
rLBH	4 large, 1 small	93,796	610	1,131	60
	4 large, 1 small	87,290	578	1,325	75
RRH	3 large, 2 small	94,387	624	983	(60)

C&W = Clarke and Wright; RRH = rural routing heuristic.

RESULTS

The results of the simulation are presented in Table 1. By using the traditional measure of fit, the total daily drive time, the rLBH method clearly provides the lowest values for both scenarios, as expected. The annual cost of providing transportation to the students by using the rLBH method falls from $93,796 to $87,290 when the time constraint is raised by 15 min with each solution using four large buses and one small bus. The annual cost for the routes generated by Lin 2-opt is $94,573 with a 60-min time constraint, a value that drops roughly $1,500 when the time constraint rises to 75 min. The related values for routes found by using the Clarke and Wright heuristic are $99,981 and $90,437, respectively. The best set of routes determined by the rLBH when a 75-min time constraint is imposed is shown in Figure 1. This route is similar to the others produced by using existing methods in that there is a loop appearance to the routes.

The rural routing heuristic is the most effective algorithm of those considered for meeting its intended goal of minimizing the "sum of times from stops," which accounts only for the time when students are in transit. This is done at an annual cost of $94,387 and requires the use of three large and two small buses. The measure for the new heuristic, at 983 min, is more than 13% less than that of the next-

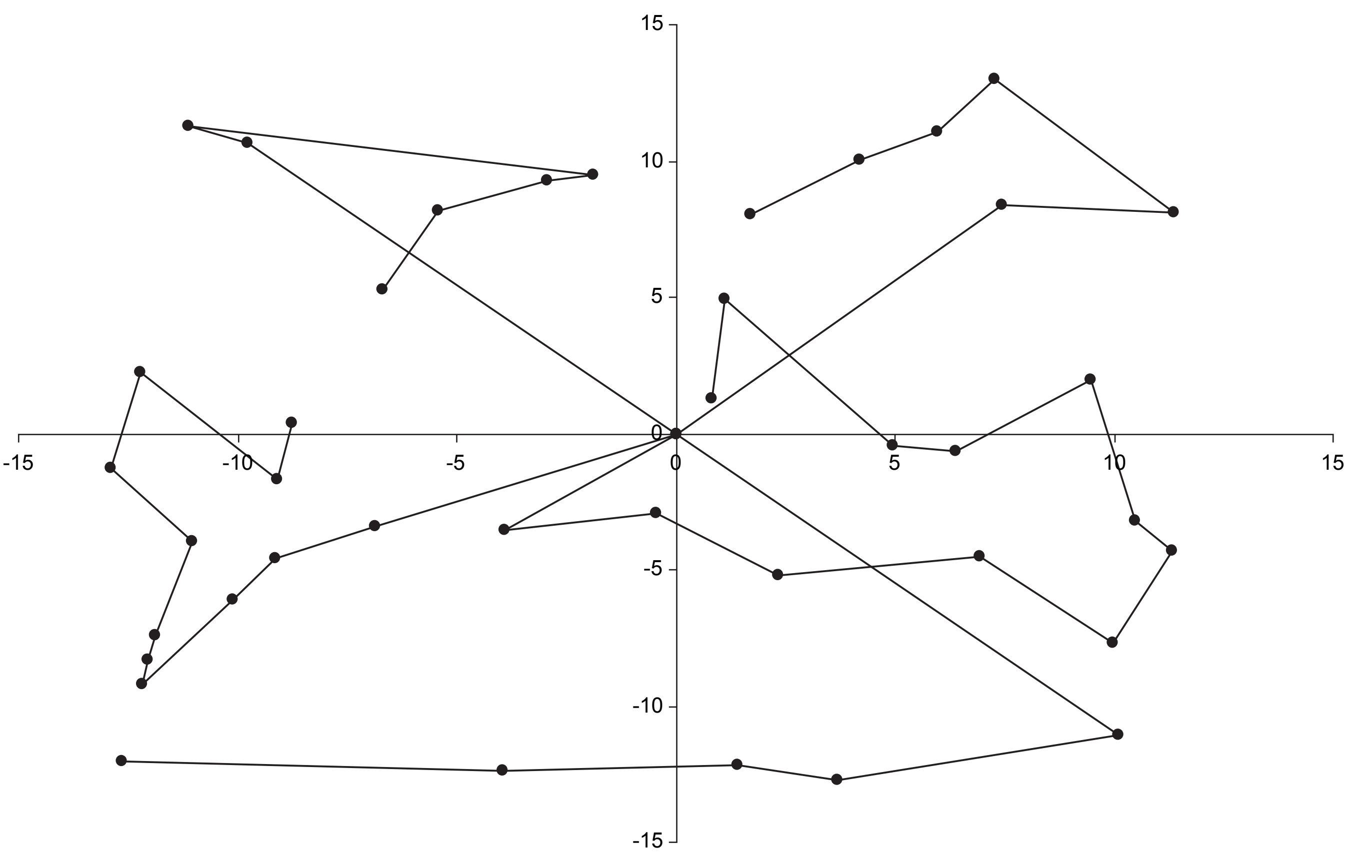

FIGURE 1 rLBH routes.

best solution, at 1,131 min, generated by the rLBH with a 60-min ride constraint.

For the tabu search algorithm, a number of combinations of values for length of tenure and the number of interchanges were used. However, because of the small size of the problem in computational terms, these differing parameters resulted in no change in the best solution. Also, although not imposed because of the structure of the rural routing heuristic, the best solution would meet a 60-min time constraint, as the longest period any student spends on the bus is just over 57 min. The rural routing heuristic routes are depicted in Figure 2. This figure visually differs from the first in that routes now begin at distant points and then return, somewhat erratically, to the centrally located school.

Implementation and Opportunities for Further Research

Because the objective for the rural routing heuristic, a function of riding time, is fundamentally different than for previous algorithms, it is difficult to estimate what effects its implementation might have for a particular school district in either cost of operation or safe transport of students. This is especially true because although one can assert, other things being equal, that a shorter trip is safer than a longer one, other phenomena, such as road type and condition, traffic, and rail crossings, are not accounted for in the model. The rural routing heuristic is valuable because it provides a more logical solution to the school transportation problem in a rural setting than other heuristics in a way that minimizes student travel time. However, to isolate the safety component of school transportation completely while continuing to utilize techniques from operations research, a model fundamentally different from the one presented here or elsewhere in the previous literature would need to be constructed.

Students with Special Needs

For students with special needs, two options for using the rural routing heuristic are available. The first, which is essentially the technique commonly done by hand, is to optimize two or more problems, distinguishing between students with special needs and those without, and assigning suitable vehicles to each set of routes. Alternatively, commingling students may be feasible for school districts that already own suitable vehicles or for those that can procure them. This can be done by altering the rural routing heuristic by keeping tally of vehicle occupancy and capacity, classified by seat type, for each vehicle and also by classifying students according to their needs. For example, if there are 20 conventional seats, seating two students each, and two spaces with wheelchair tie-downs, a total of two students in wheelchairs may be accommodated as well as 40 other students. Then as a student is assigned to a route, given that proper space is available, a seat matching his or her needs is filled, and the opposite occurs when a student is removed. Although the up-front costs to implement such a system is high, its annual cost of operation, especially when it eliminates one or more buses, may make it practical.

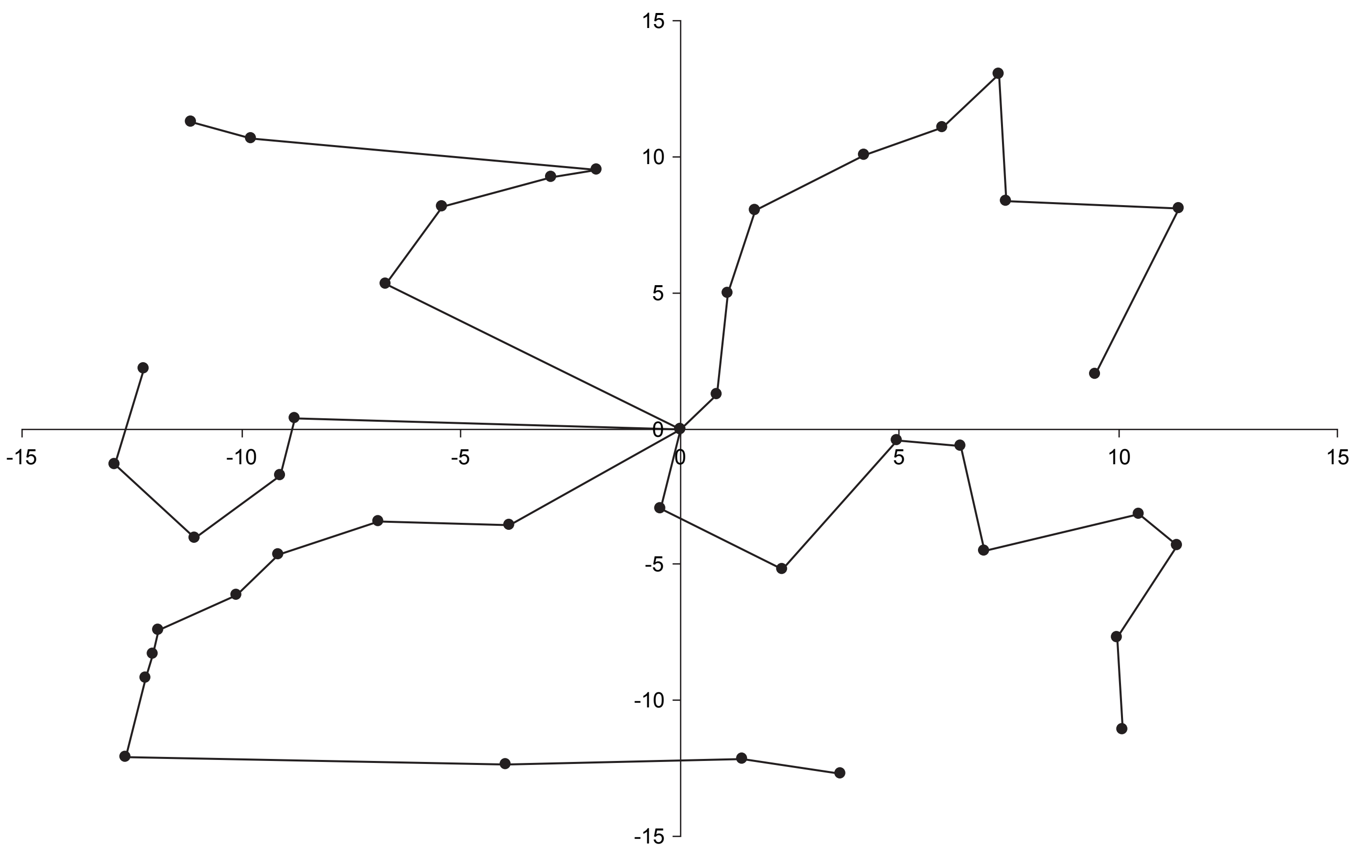

FIGURE 2 Rural routing heuristic routes.

SUMMARY

Although the rural routing heuristic provides poor estimates for the traditionally posed school bus routing problem, it may produce viable alternatives for school districts concerned about their students' ride times and safety. It also provides information about what fleet mix, including nontraditional vehicles, meets a district's needs and can be easily altered to accommodate routing students with special needs. However, because there is a definite trade-off between the cost of operation and the level of service, measured as a function of ride time, the determination of which heuristics and routes best meet the needs of a given district can be made only by administrators.

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Integrated Planning for School and Community

Jeff Tsai and Mike Miller

The Operations Research/Education Laboratory (OR/Ed. lab) at the Institute for Transportation Research and Education, North Carolina State University, has a long history of providing school systems with data-driven solutions for school population forecasting, school attendance studies, and determination of new school locations. These planning processes, known as Integrated Planning for School and Community (IPSAC), provide school districts with mathematically optimal solutions that minimize transportation distance. The OR/Ed. lab works closely with school districts in politically and emotionally charged environments involving school locations and attendance district changes. The success of IPSAC lies in its approach to enumerate school planning needs and school population growth impression through the use of data. Furthermore, through the operations research optimization techniques, favorable solutions are achieved to satisfy the constraints, needs, and policies of the school district. Recent national studies in active school travel have reported that distance to school and built environment have a significan influence on how children travel to school. These research finding prompted the OR/Ed. lab to investigate ways to enhance IPSAC so that school districts may obtain solutions to include multimode school transportation as one of their considerations. This paper describes the IPSAC process, challenges faced by school districts, and areas in which the integration between school planning and transportation planning deserve explorations.

The location of new schools has gained national attention in recent years among researchers from environment, land use, public health, and transportation sectors (*1–3*). Research findings indicate that the location of schools affects future land use and the methods by which school-age children travel to school. Smaller neighborhood schools that are well-connected by sidewalks and bike paths promote active travel.

The Operations Research/Education Laboratory (OR/Ed. lab) at the Institute for Transportation Research and Education (ITRE), North Carolina State University, has a long history of providing school systems with data-driven solutions for school population forecasting, school attendance studies, and determining new school locations. These planning processes, known as Integrated Planning for School and Community (IPSAC), provide school districts with mathematically optimal solutions that minimize transportation distance.

Through close working relationships with many school districts, the OR/Ed. lab is well attuned to the specifics involved in school planning, selecting school locations, and school attendance district changes. This paper illustrates some of the realistic challenges faced by school districts.

The approach, adopted by IPSAC, is similar to travel demand modeling. ITRE is conducting research to integrate travel demand attributes into the school planning process.

IPSAC PROCESSES

An initial step in the IPSAC process is to develop school-level enrollment forecasts for each year of the ensuing decade. This first step consists of three phases. The first phase is backward looking, in which a modified cohort survival technique is applied to 6 years of historic enrollment data and 10 years of live-birth data. The second phase is forward looking, in which a land use study is conducted, as explained in this paper. At the final phase, these two phases are integrated to form the final enrollment forecast, called an out-of-capacity worksheet.

The school district is divided into regions called planning segments by using various criteria. Each planning segment contains 50 to 100 students; all students in one segment must attend the same school, and neighborhood boundaries are preserved whenever possible. The planning segments will form the fundamental building blocks of school attendances and will provide the units of analysis for the optimization process. Consequently, the construction of these planning segments is critical and must occur early in the project, along with the cooperation of the district. The school transportation departments are intimately involved in the construction of these planning segments, which reflects the importance of school transportation in the process of school location and attendance boundary decision making.

However, school transportation officials are primarily concerned with busing issues, and the design of the planning segments reflect this priority. Major arterials are often used to divide planning segments. Known roadway hazards, such as railroad grade crossings, narrow bridges, or other features that could present obstacles for school bus routes, are generally used to delineate planning segments. In addition, large subdivisions that contain a large school-age population may require more than one bus and are often divided into multiple planning segments. An example of planning segments and their relationship to streets and parcels is illustrated in Figure 1.

Land Use Study

IPSAC is supplemented by a comprehensive land use study of the geographic area encompassing the school district. The objective for the land use study is to quantify future growth by school attendance districts. The land use study includes two components: community interviews and geographic information systems (GIS) data analyses.

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FIGURE 1 School planning segments (shaded), school attendance (heavy line), and streets.

The community interviews allow the OR/Ed. lab to compose an impression of future growth of the study area by interviewing planners, town managers, mayors, utility works, realtors, developers, and so forth. Community involvement in this study is essential, as these interviews provide an understanding of infrastructure development plans (transportation, water, and sewer), recent subdivision permits, residential zonings, available land for development, and comprehensive plans developed by local government agencies. The community interviews build credibility for the study by including key stakeholders' knowledge and include information on future growth that is often not captured during the GIS data study.

The GIS data analysis is the technical aspect of the land use study. The GIS parcel data of the study area often contain information concerning the types of structures existing on a parcel, which allows the OR/Ed. lab to determine if a specific parcel is vacant or contains a residence. By using the geocoded students and occupied residential parcels (lots occupied with homes), the lab can calculate student generation rates (SGRs) for a given region.

Subdivisions are often good indicators of concentrated rapid growth and can significantly contribute to student population growth. As long as a subdivision key exists in the GIS data, this technique provides the lab with a valuable tool for the calculation of student generation rates for each subdivision.

The total number of students that may be generated from this subdivision at build-out is calculated by multiplying the SGR to the number of remaining lots. This approach of forecasting student growth potential also applies to planned subdivisions that attract similar residential demographics.

In the example shown in Figure 2, 47 students reside in 176 parcels containing structures. Thus, the SGR for this area is .267. Assumptions can be made that approximately 27 students will be generated for every 100 houses built in this subdivision.

Although the land use study can be time intensive, it results in the generation of highly accurate data, which adds to the credibility of the study when it is presented to the public.

Out-of-Capacity Worksheet

The out-of-capacity worksheet displays the enrollment forecast for each school building and shows the relationship to the design capacity of the buildings. The spreadsheet is color coded to show the years in which a school is above or below capacity. An example of an out-of-capacity worksheet is shown in Table 1.

Often the lab works with the school architecture firm to develop a comprehensive school facility needs study. The calculation of school building capacity is a difficult topic because it is a function of program and how each classroom is used. This involves laborious interviews with school administrators and is best done by school architects.

Optimal School Sites and Attendance Boundaries

With the understanding of anticipated growth through the land use study and a forecast of school enrollment through the out-of-capacity worksheet, the forecasted school population is disaggregated to the individual planning segments. Data are now available on number of school-age children and their grade levels for each planning segment and for each of the 10 years within the forecast.

For growing school districts and those that are seeking voter approval for new school construction, the optimal location and size of new schools are determined by using a nonlinear mathematical programming model. The typical objective is to minimize the total transportation distance, and the typical constraints are designed to implement school board policies that pertain to such matters as school size, grade structure, and demographic and socioeconomic balances.

As the optimization algorithm produces scenarios for new school locations, it also defines attendance boundaries by aggregating planning segments. If the new school location has been determined, the algorithm can also define attendance boundaries by using the known sites. The school assignments given to each planning segment will produce student populations that meet the conditions set by the school district while ensuring minimum transportation costs.

The optimal school location produced by the OR/Ed. lab is a mathematical optimal. In the real world, it may or may not be a practical location because of many considerations, discussed in the next section. Thus the location is presented as a target area on which a school district can focus its search for suitable property.

Demographic and Socioeconomic Indices

Many school districts face the challenge of constructing new schools that contain a balanced demographic and socioeconomic population. Responding to these needs, the OR/Ed. lab analyzes an assortment of data, some of which include GIS parcel, census, student demographic, achievement scores, and tax data. From these analyses, the OR/Ed. lab can assign demographic and socioeconomic indices to each plan-

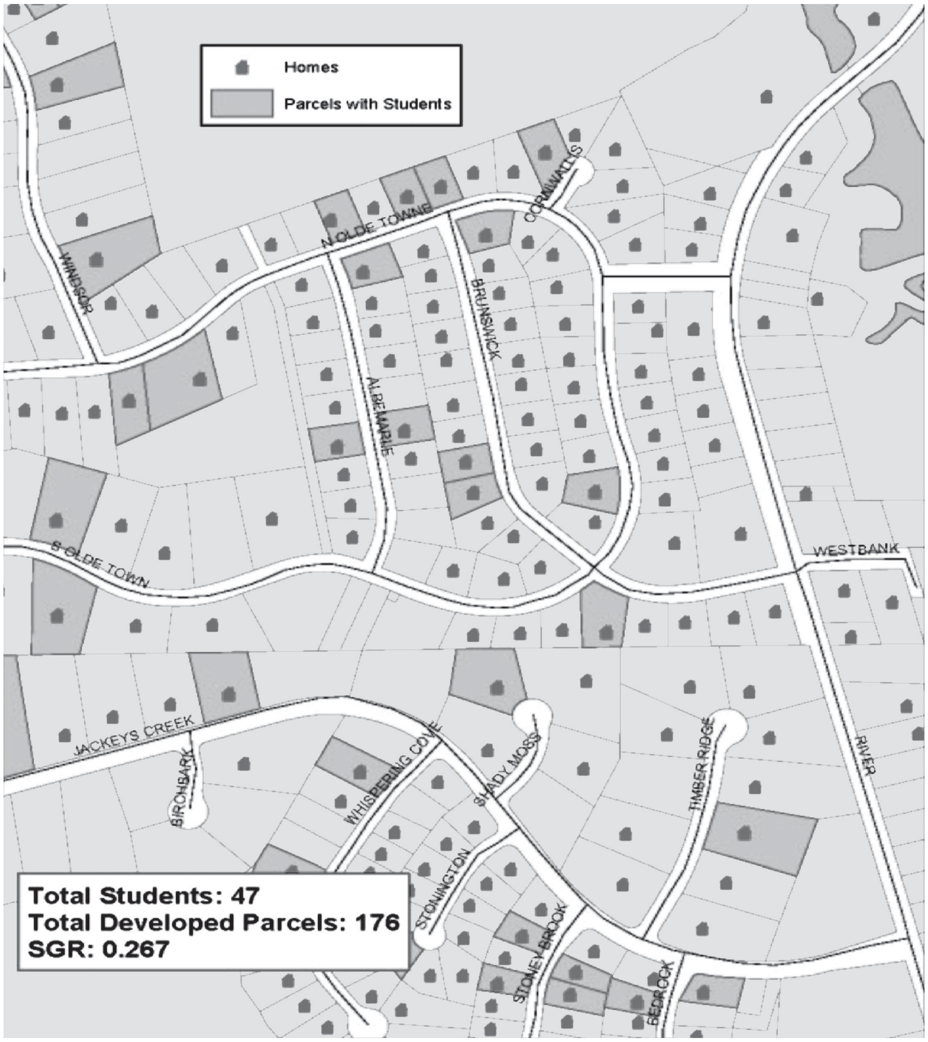

FIGURE 2 Calculation of SGRs.

ning segment on the basis of the general and school populations that reside in the planning segment. The school districts then know the districtwide averages for each index and can establish upper and lower ranges to achieve the desired balance.

At the request of school districts, the lab has created indices to measure minority percentages, academic performance, the percentage of students eligible for free or reduced-cost meals, and housing costs. The school district can define the variability of each index within each affected school. The district may, for instance, choose to concentrate on balancing the socioeconomic index among affected schools. Or, the district may decide that contiguous attendance areas are of primary importance and prefer to allow more flexibility in the socioeconomic or demographic balance of the students.

In a recent project, the lab was commissioned to construct high school attendance boundaries that would yield a balanced student population. Because of a prior lawsuit, the district could not use the percentage of minority students as the priority means to defin these boundaries. The OR/Ed. lab constructed several indices, including the percentage of homes with a value above or below fixed thresholds, the percentage of students eligible for free or reduced-cost meals, and the percentage of students scoring below a cutoff score in a standardized test, in addition to the minority percentage.

Nineteen solutions were produced by using multiple constraints, maintaining a balance on two of the indices simultaneously. This allowed the school district to simulate a range of scenarios and ultimately to use the optimal boundaries to fashion suitable boundaries that satisfied its requirements. For this school district, as seen in Figure 3, the need to achieve demographically balanced schools superseded minimizing transportation costs. Satellite boundaries, though unpopular and impractical for transportation, are necessary to meet school policies for demographically balanced schools.

CHALLENGES AND OPPORTUNITIES

School Site Selection Realities

Through the IPSAC projects, the OR/Ed. lab works closely with a variety of school district personnel, including superintendents, cabinet-level administrators, school board members, and transporta-

TABLE 1 Out-of-Capacity Worksheet

	Capacity				Sept. 2003	Projected Enrollment								
	2003–04	2004–05	2005–06	2006–07	2003–04	2004–05	2005–06	2006–07	2007–08	2008–09	2009–10	2010–2011	2011–12	2012–13
Elementary														
Marshville	700	700	700	700	523	524	526	528	529	531	534	536	538	540
Union	550	550	550	550	384	384	384	384	384	384	384	384	384	384
Wingate	700	700	700	700	749	762	775	790	806	825	845	863	883	902
Benton Heights (y.r.)	750	750	750	750	745	756	766	779	793	809	825	840	857	873
East (y.r.)	750	750	750	750	755	766	776	789	803	819	835	850	866	882
Walter Bickett (y.r.)	750	750	750	750	574	585	595	608	622	638	654	669	685	701
Prospect	550	550	550	550	530	530	530	530	530	530	530	530	530	530
Waxhaw	750	750	750	750	559	576	593	613	635	661	687	710	737	763
Western Union	650	650	650	650	643	669	694	724	757	796	835	870	910	949
Fairview	750	750	750	750	566	590	613	641	671	707	743	775	813	848
Hemby Bridge	725	750	750	750	626	656	685	720	758	803	848	888	935	979
New Salem	325	325	325	325	298	308	318	330	343	359	374	388	404	419
Unionville	750	750	750	750	723	759	796	838	885	939	995	1,045	1,102	1,156
Indian Trail	750	750	750	750	765	799	833	873	916	968	1,020	1,066	1,119	1,170
Sardis	750	750	750	750	980	1,058	1,136	1,227	1,327	1,444	1,564	1,669	1,791	1,907
Shiloh (y.r.)	750	750	750	750	1,072	1,174	1,277	1,395	1,526	1,680	1,837	1,975	2,135	2,287
Marvin	750	750	750	750	1,002	1,203	1,406	1,640	1,899	2,203	2,513	2,787	3,103	3,403
Weddington	750	750	750	750	601	632	663	699	739	786	833	876	924	970
Wesley Chapel	650	650	650	650	901	978	1,057	1,147	1,246	1,363	1,482	1,587	1,709	1,825
New ES A		750	750	750										
New ES B			750	750										
New ES D		750	750	750										
Totals	13,100	14,625	15,375	15,375	12,996	13,706	14,426	15,254	16,170	17,242	18,340	19,307	20,425	21,488

(continued)

TABLE 1 *(continued)* **Out-of-Capacity Worksheet**

	Capacity				Sept. 2003	Projected Enrollment								
	2003–04	2004–05	2005–06	2006–07	2003–04	2004–05	2005–06	2006–07	2007–08	2008–09	2009–10	2010–2011	2011–12	2012–13
Middle														
New MS			1,000	1,000										
East Union	1,000	1,000	1,000	1,000	815	819	826	835	842	848	854	865	877	891
Monroe (y.r.)	1,000	1,000	1,000	1,000	847	856	872	892	908	921	935	961	988	1,018
Parkwood	950	1,000	1,000	1,000	933	945	967	992	1,015	1,031	1,050	1,084	1,120	1,161
Piedmont	1,000	1,000	1,000	1,000	1,153	1,181	1,232	1,292	1,344	1,382	1,426	1,506	1,590	1,684
Sun Valley	1,150	1,150	1,150	1,150	1,296	1,357	1,465	1,593	1,704	1,786	1,881	2,053	2,233	2,434
Weddington	800	1,000	1,000	1,000	1,218	1,307	1,463	1,649	1,810	1,929	2,066	2,316	2,577	2,868
Totals	5,900	6,150	7,150	7,150	6,262	6,465	6,824	7,252	7,623	7,896	8,211	8,785	9,387	10,056
High														
New HS			1,200	1,200										
Forest Hills	910	910	910	910	920	929	937	943	949	956	967	977	985	995
Monroe High	840	840	840	840	938	959	976	989	1,003	1,019	1,043	1,067	1,084	1,106
Parkwood High	1,235	1,235	1,235	1,235	1,226	1,254	1,276	1,294	1,313	1,334	1,366	1,397	1,421	1,450
Piedmont High	1,100	1,200	1,200	1,200	1,357	1,422	1,474	1,515	1,559	1,610	1,684	1,757	1,813	1,881
Sun Valley High	1,225	1,225	1,225	1,225	1,428	1,567	1,679	1,766	1,861	1,970	2,129	2,285	2,405	2,550
Weddington High	1,200	1,200	1,200	1,200	1,477	1,678	1,842	1,968	2,105	2,263	2,493	2,719	2,893	3,104
Totals	6,510	6,610	7,810	7,810	7,346	7,808	8,184	8,474	8,790	9,152	9,682	10,202	10,601	11,086
Special														
South Providence					91	91	91	91	91	91	91	91	91	91
Wolf School					68	68	68	68	68	68	68	68	68	68
Career Center					1	1	1	1	1	1	1	1	1	1
Totals					160	160	160	160	160	160	160	160	160	160
System Total	25,510	27,385	30,335	30,335	26,764	28,139	29,595	31,140	32,743	34,450	36,393	38,455	40,573	42,790

Adequate capacity   Two-year warning   Out of capacity
y.r. = year-round; ES = elementary school.

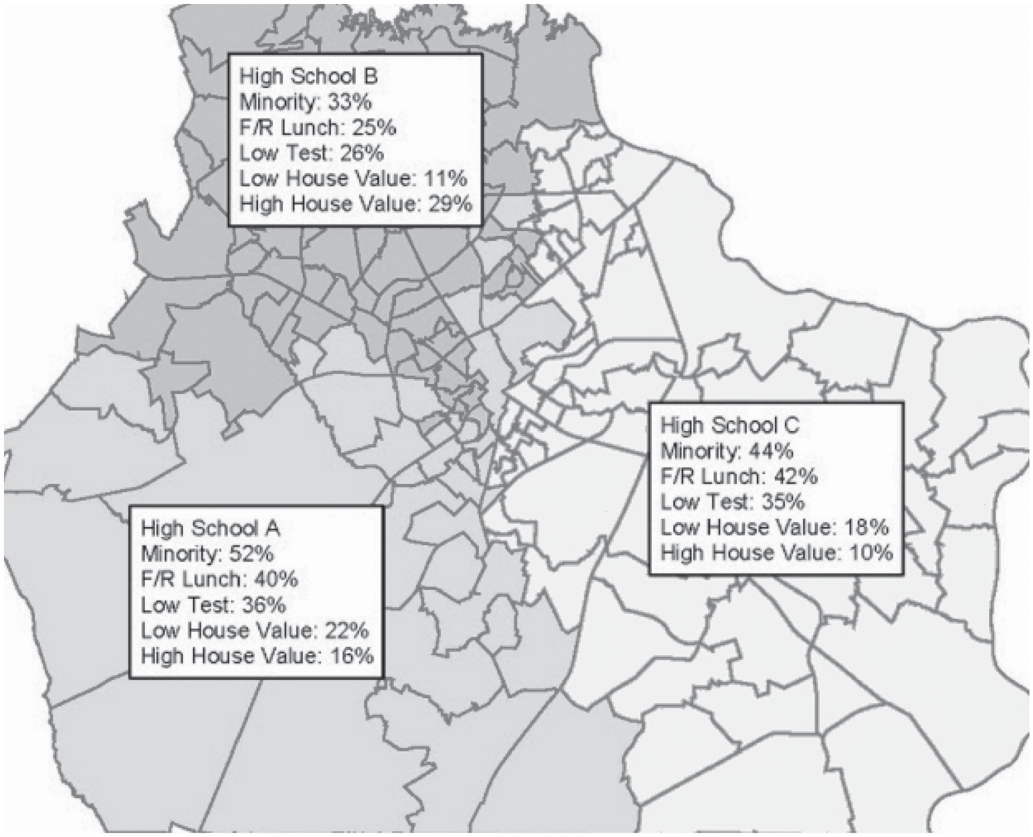

FIGURE 3 Optimal school attendance with satellite boundaries (F/R = free or reduced cost).

tion personnel. Following is a list of considerations usually faced by rapidly growing urban and suburban school districts when selecting school locations and adjusting school attendance:

- The property must be affordable. Properties already owned by the local government receive the first priority of consideration.
- Donated properties are high on the list. Large-scale subdivisions sometimes offer parcels for school sites.
- The property must have access to water, sewer, and power.
- The size of the property must be adequate to meet school needs. Schools serving high school populations will require larger property for athletic fields
- The property location must not present safety and security concerns.
- The location of the school should produce school attendance that minimizes the effect on existing schools. This happens mostly to elementary schools that serve six grades (from kindergarten through fifth grade). It is not unlikely in a high-growth area that multiple new schools were constructed, causing multiple reassignments.
- School attendance should be configured to reduce the need by school buses and private vehicles to cross major highways.
- The school must be large enough to offer a variety of academic opportunities, such as academically gifted programs and art and music classes.
- The school must be small enough to offer quality education.
- The school population must be balanced demographically.
- The student's travel distance to school should be minimized.

Another important consideration within the school planning process is the attentive public. Citizens are highly motivated by issues concerning their children's schooling. Even people who rarely vote, or those who rarely stay abreast of political issues, will attend school redistricting meetings. Consequently, any decisions that pertain to the location of new schools or the reassignment of students must be able to address these issues.

Costs of Multimode School Travel

Cost drives public school decisions. The costs to build a school are well understood, but the transportation impact costs of a school are less clear (*4*). A few questions that should be considered are as follows:

- How much private vehicle traffic will the school generate, and what is the cost to design a campus capable of accommodating these vehicles?
- What is the impact of school-generated vehicular trips on a roadway, and what is the enhancement cost to accommodate additional traffic?
- What is the cost of building sidewalks to promote biking and walking, and what are the costs of hiring rights-of-way and employing crossing guards?
- What are the other long-term societal costs, such as air quality introduced by motor vehicles, and health costs due to inactive travel?

This subject should be a great interest to those who are ultimately affected by the school location by additional traffic. Most of these costs are shared partly, if not entirely, by local and state agencies outside the school district. Until these costs can be articulated and synthesized,

FIGURE 4 Comparison of school planning segment (outlined polygons) and TAZ (shaded polygons).

it will be difficult to convince policy makers to consider all modes of school transportation in the school planning process.

Collaborations Between School Planning and Transportation Planning

Efforts are under way at the lab to integrated transportation-related criteria as the optimization constraints to determine optimal school location. Figure 4 illustrates the initial attempt to assess similarities between the school planning segments that are developed by the school and the transportation analysis zone (TAZ) developed for a regional transportation demand model.

This example consists of a school district located in the outskirts of a rapidly growing urban city. The current school population is 26,700, and the estimated annual growth averages 1,800 students per year for the next 10 years. The annual student increase in this school district is equivalent to opening one new elementary school per year and one middle and one high school every 4 years.

As shown in Figure 3, the TAZs and the school planning segments are somewhat similar geographically with the exception of areas (western part of the school district) with extreme high residential population. In such areas, an SGR of .75 or higher is not uncommon; thus large-scale subdivisions are often divided into several planning segments and assigned to different schools.

The regional transportation demand model and the IPSAC processes have an identical approach that uses smaller planning areas to conduct forecasts. The difference is that the regional transportation demand model's primary concerns are trip generation and trip origin and destination attributed to employment. IPSAC's primary concern is number of students generated by residential growth. Since it is estimated that nearly 30% of the morning peak traffic is school related (*3*), the school sites and school-related traffic should be of great interest to both regional transportation modelers and school planners. Such collaboration is at its earliest development phase.

SUMMARY

School planning is often carried out within complex, multilayered, and insufficiently articulated environments. In addition to identifying affordable locations for new schools, school districts are challenged with reaching compromising solutions to satisfy a variety of requirements. Through the IPSAC processes, the OR/Ed. lab has successfully assisted school districts with school location and attendance boundary decisions through planning solutions that are driven by policy and supported by data.

More research is needed to determine how school-related traffic affects roadway congestion and the public costs of school travel. School transportation departments need to participate in discussions beyond those of school bus transportation. Most important, transportation planners must be sensitive to the realistic challenges faced by school districts. Through mutual understanding of issues, state and local agencies can collaborate in school planning decisions to optimize public investments while achieving local educational goals.

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Modeling and Performance Assessment of Contraflow Evacuation Termination Points

Erick Lim and Brian Wolshon

About 20 hurricane evacuation contraflow segments are planned for use in the United States. When activated, these routes will serve as lifelines for people fleeing the potential destruction of approaching storms. The termination points of these segments are critical because they move traffic from the reverse-flowing lane into the normal flow direction. They are also thought to affect the overall effectiveness of the sections significantl because they can regulate the amount of volume that exits the section. The research effort described in this paper was undertaken to assess and compare the operational characteristics of contraflow evacuation termination point designs that would be used under threat from catastrophic storms. Among the developments of the research was an approach and set of assumptions for using CORSIM to model contraflowing freeway traffic under evacuation conditions. These models were used to assess and rank the planned termination designs comparatively and to identify the factors that made some designs more effective than others, including the effect of reducing traffic volumes before the termination. The quantifi cation of the operations revealed several important concepts relative to the use of contraflow evacuation segments. First, it is advantageous to maintain all lanes through the termination point with split rather than merge designs. Also, it is advantageous to reduce the volume entering the termination point by maintaining exit points along the route. The study suggests that merge zones located after exits, instead of before them, and the use of channelization or separation devices well in advance of forced maneuvers can enhance the quality of the flow through the termination vicinity.

During the past 5 years, highway agencies in nine coastal states threatened by hurricanes have developed plans for the use of contraflo traffic operations on freeways during evacuations. Contraflow involves the use of one or more inbound travel lanes for the movement of traffic in the outbound direction. The use of contraflow is particularly useful for evacuations because the inbound flow during evacuations is very low, whereas the outbound demand often overwhelms the available capacity of the road system. It is also highly cost-effective since significant capacity gains can be made without the need to construct additional lanes.

Although contraflow is widely viewed as a major advancement in the ability of highway agencies to increase the effectiveness of evacuations, it is not without its drawbacks. In fact, these negative aspects are why most states plan to use contraflow only under the most extreme threat conditions and only for the evacuation of major population centers. Among the recognized shortcomings of contraflo evacuations are the following:

- It eliminates inbound movement of traffic into the evacuation zone. This can be a problem because the early stages of evacuations typically involve a mobilization period during which people enter the threat zone to retrieve family members and property as well as to secure homes and businesses. Inbound entry is often required by law enforcement and emergency response personnel and service vehicles that need to tend to roadway incidents on evacuation routes.
- It can be confusing to drivers and can increase the likelihood of dangerous traffic conflicts
- It often restricts the ability of evacuees to make routing choices to reach their destinations, including the closure of exit and entry points along the intermediate contraflow segment.
- It requires increased levels of manpower and material and equipment for implementation and operation of the evacuation as well as the need for longer lead times to configure roadways for its use.

Another limitation of contraflow is the lack of experience in its use for evacuations. Although widely planned, it has been implemented only twice, once as an improvised contraflow evacuation in South Carolina in 1999. The lack of use has meant that there is an absence of field data and analyses on the characteristics of contraflow evacuation traffic streams and a limited number of simulation studies to evaluate its effect at local or system levels.

To better prepare state departments of transportation (DOTs) and emergency management agencies for the use of contraflow, a series of research projects was undertaken. Among these have been efforts to evaluate the characteristics of traffic operation within and near contraflow evacuation segments. This paper summarizes the results of one of these studies in which the effects of the various contraflo termination designs planned for the Atlantic and Gulf Coast states were evaluated.

OBJECTIVES

Like most traffic management strategies, the development of evacuation plans requires trade-offs among a number of factors, including operational efficiency, safety, cost, and manpower requirements. Unlike routine strategies, evacuation can directly affect the life and well-being of tens (perhaps hundreds) of thousands of people in a

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single event. Another problem of evacuation planning is that evacuations are relatively infrequent events, often occurring as rarely as once a decade. As such, they do not necessarily receive the attention of more frequently occurring, although perhaps less critical, problems. To begin to address the need for improving evacuation traffic planning and management, in general, and to better understand contraflow evacuation traffic issues specifically, this study was undertaken to evaluate and assess the operation of planned contraflow evacuation termination points. The overall goal for this effort was to identify the pros and cons of the current plans and look for ways to improve them before the arrival of the next major storm.

Contraflow termini are widely assumed to control the capacity of contraflow sections because they control the number of vehicles that can enter and exit the segment. Whereas a prior study evaluated traffic operations in the vicinity of contraflow entry points (*1*), this study focused on the departure end of these segments. In this study, simulation models for six different types of termination points were created on the basis of the recently developed plans for 13 segments of Interstate freeway in seven hurricane-threatened states. The output data from these models were used to generally quantify the traffic conditions in the vicinity of the termination points and to compare the relative performance and benefits of each of the designs.

The purpose of this study was not to suggest or advocate the use of any design over another. Rather, it was done to identify common characteristics of certain configurations and design elements that can enhance the effectiveness of these segments as well as to suggest some of the reasons this may be the case. Another key element of the study was to compare the performance of the various designs under varying levels of demand. Unlike the fairly predictable demand patterns of routine peak period demand, evacuation volume can vary widely depending on factors such as storm strength, arrival time, and landfall location. The demand that would be present at a contraflo termination point would also be a function of the number and location of exits and volume balance crossovers along the intermediate section.

TERMINATION POINT CONFIGURATIONS

At the time of the study, 21 controlled access evacuation contraflo segments were planned for use in the United States (*2*). The method by which contraflow operations were terminated at the end of each of these segments can be broadly classified into one of two groups. The first were the split designs, in which traffic in the normal and contraflowing lanes was routed onto separate roadways at the end of the segment. The second group was merge designs, in which the separate lane groups were reunited into the normal travel lanes by using various geometric and control schemes. The selection of one or another of these termination configurations at a particular location by an agency was a function of several factors, most importantly the level of traffic volume and the configuration and availability of routing options at the end of the segment.

In general, split designs offer higher levels of operational efficiency of the two designs. The obvious benefit of a split is that it reduces the potential for bottleneck congestion resulting from merging four lanes into two. Its most significant drawback is that it requires one of the two lane groups to exit to a different route, thereby eliminating route options at the end of the segment. In Louisiana, where the 25-mi contraflow segment terminates at the interchange of I-10 and I-55, traffic in the normal lanes will be routed onto northbound I-55 and the contraflow traffic into the normal lanes of I-10 by using a median crossover after the interchange, as diagrammed in the schematic drawing of Figure 1. In some designs, such as the original North Carolina plan shown in Figure 2, the contraflow traffic stream was planned to be routed onto an intersecting arterial roadway. One of the needs for this latter type of split design is adequate capacity on the receiving roadway. It is for this reason that North Carolina Department of Transportation (NCDOT) officials are reevaluating the feasibility of this design.

Merge termination designs also have pros and cons. However, in many respects these costs and benefits are nearly the opposite of split designs in their end effect. For example, most merge designs preserve routing options for evacuees because they do not force vehicles onto adjacent roadways and exits. Unfortunately, the negative side to this is that they also have a greater potential to cause congestion since they merge traffic into a smaller number of lanes. At first glance, it would appear illogical to merge two high-volume roadways into one. However, DOTs in most locations where merges are planned will also maintain exit opportunities along the intermediate segment to decrease the volumes at the end. Other states have placed the termination points at locations where additional lanes become available, such as in Texas, where the termination of the northbound I-37 contraflo out of Corpus Christi occurs on the outskirts of San Antonio.

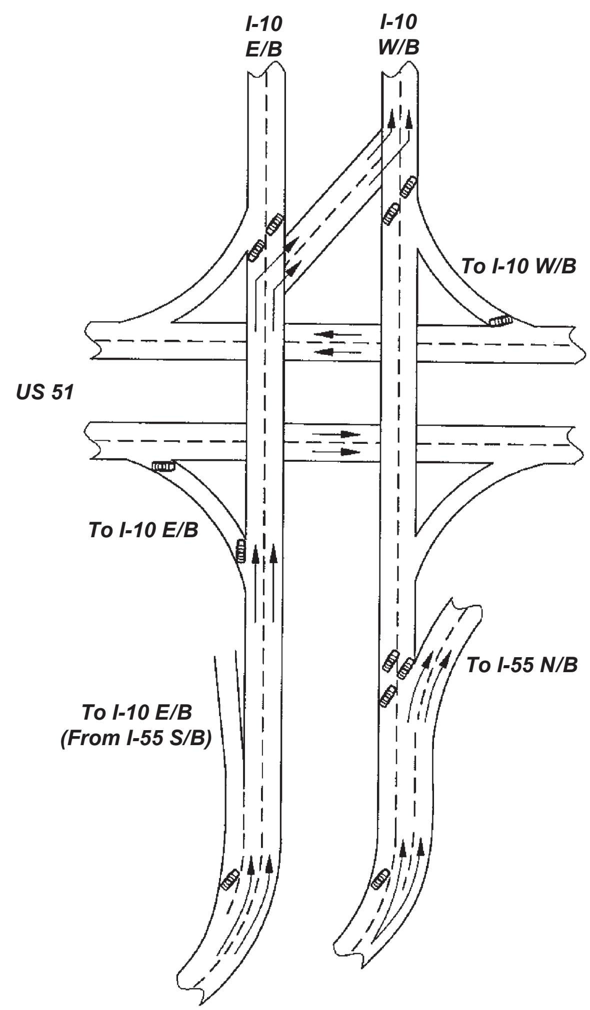

FIGURE 1 Louisiana contraflow termination design (SOURCE: Louisiana State Police).

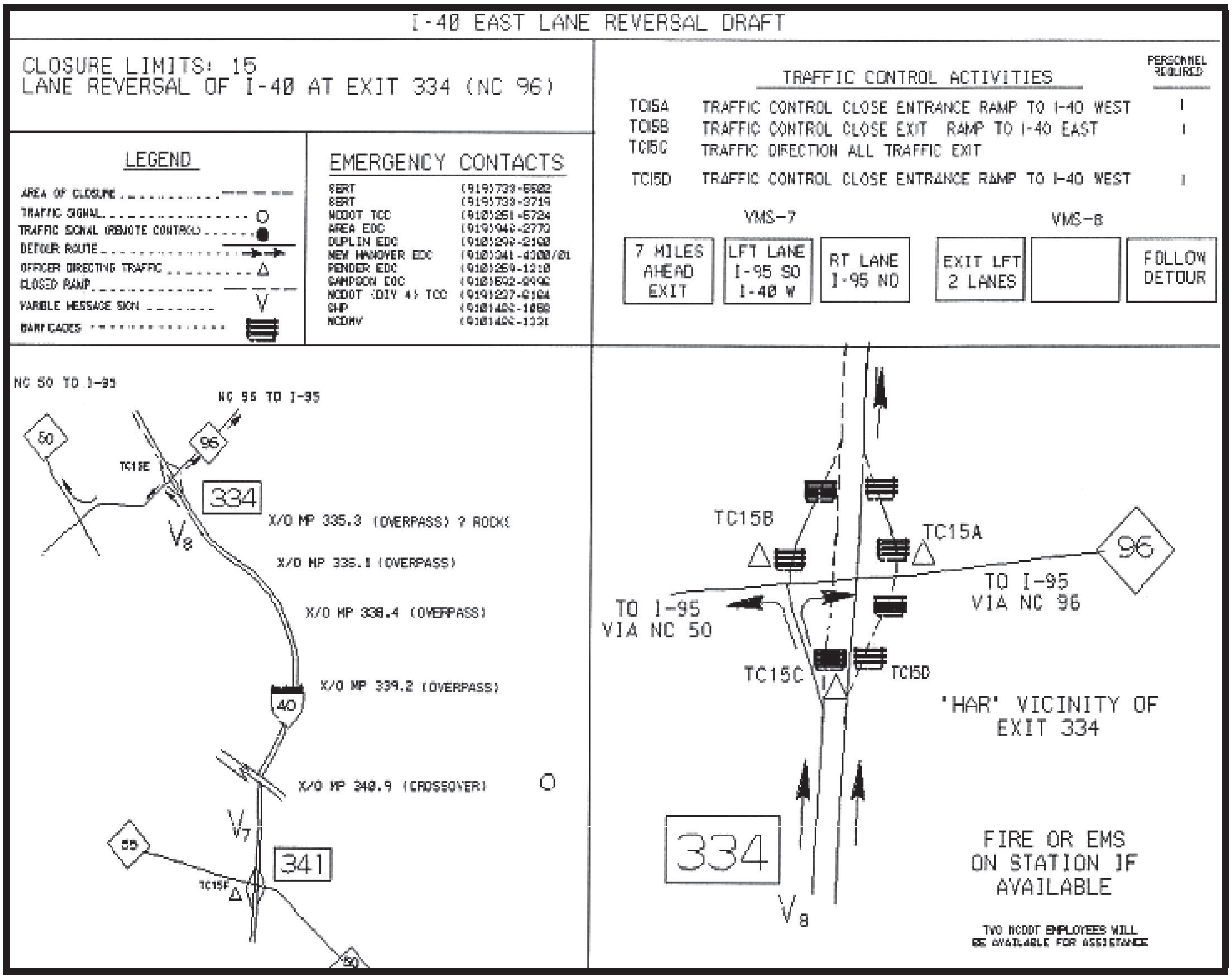

FIGURE 2 North Carolina I-40 contraflow termination design (Source: North Carolina DOT).

In the Georgia merge plan, diagramed in Figure 3, traffic on both sides of I-16 will be merged into single lanes and then, by using a median crossover, merged together into the normal lanes. The expectation is that enough traffic will have exited along the >110-mi segment to allow reasonable flow conditions within the merge vicinity and upstream of this point. In Virginia, a somewhat different merge design is planned. Near the capital of Richmond, traffic from the contraflo side of I-64 will cross back into the normal lanes after traffic in the normal lanes is routed onto the I-295 service road at the I-264 interchange, as illustrated in Figure 4. Any merging that occurs will take place as traffic from the normal lanes combines with westbound traffic continuing on I-64.

The preceding figures also show some of the other variations between the various termination configurations. One difference is the manner in which traffic is returned to its original flow lane or routed to another roadway. Some of the configurations use exit ramps (reversed and normal) to shift traffic to other routes, whereas others use median crossovers to return traffic to lanes of a normal flo direction. Table 1 summarizes the general termination configuratio and transition design characteristics for the 13 termination points that were evaluated in this research. A more detailed discussion of these and other design features and control strategies of hurricane evacuation contraflow segments, along with additional design drawings of many of these segments, can be reviewed in two related reports (*2, 3*).

MODELS, ASSUMPTIONS, AND EXPERIMENTS

The evaluation and comparison of designs such as the one conducted here ideally would have been completed by using actual field data collected at each of these locations. However, the infrequency of contraflow evacuation use, coupled with the absence of field data, made this impossible. As a result, the research relied on the results produced by the CORSIM simulation program. Initially, a macro level model was considered for use; however, the wide acceptance of CORSIM in the transportation community as well as its simplicity, flexibility, and ability to produce the desired measures of effectiveness (MOEs) made it the ideal tool for the project. The following sections describe the development of the CORSIM models along with the underlying assumptions that were required for their use in a proj-

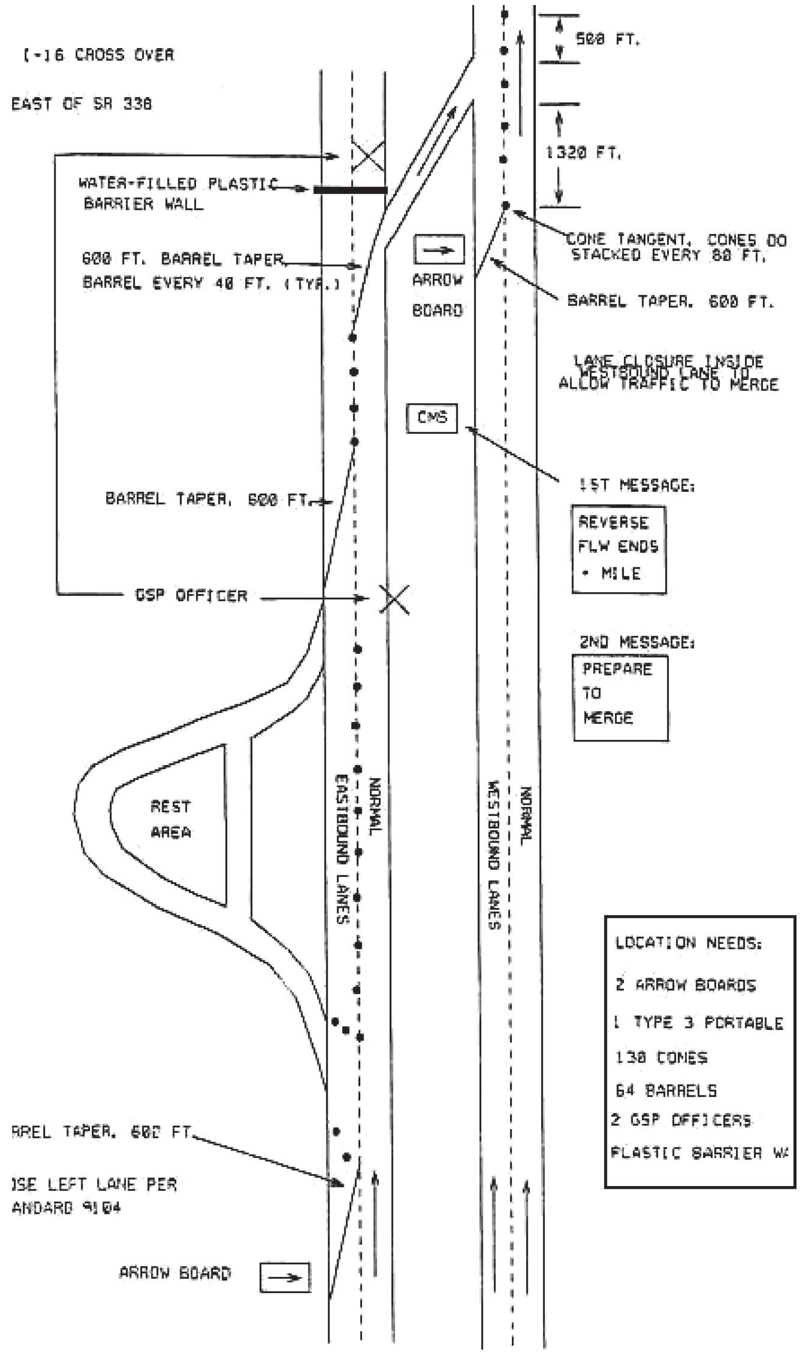

FIGURE 3 I-16 contraflow termination plan (not to scale) (Source: Georgia DOT).

ect such as this. A later section describes the series of experiments that were conducted with the models.

Model Development

Although each of the 13 contraflow termination designs was unique in specific design characteristics and traffic patterns, some characteristics were consistent over many of the designs. These similarities were used to develop six generic configurations capable of representing traffic operations in the vicinity of the termination point. Each of these terminations designs was then joined to the end of a 13-mi tangent approach segment. Schematically illustrated in Figure 5, these generic designs were designated Types A, B, C, D, E, and F, on the basis of the general ranking of the amount of traffic each would be assumed to be able to carry.

All six models incorporated median crossovers, although the Type A model was the only split configuration in the study. It featured a crossover to return contraflow traffic to the normal lanes similar to the plan for westbound I-10 outside of New Orleans. Models B through D were all merge configurations. The primary differences between them were the side to which vehicle exits would be permitted

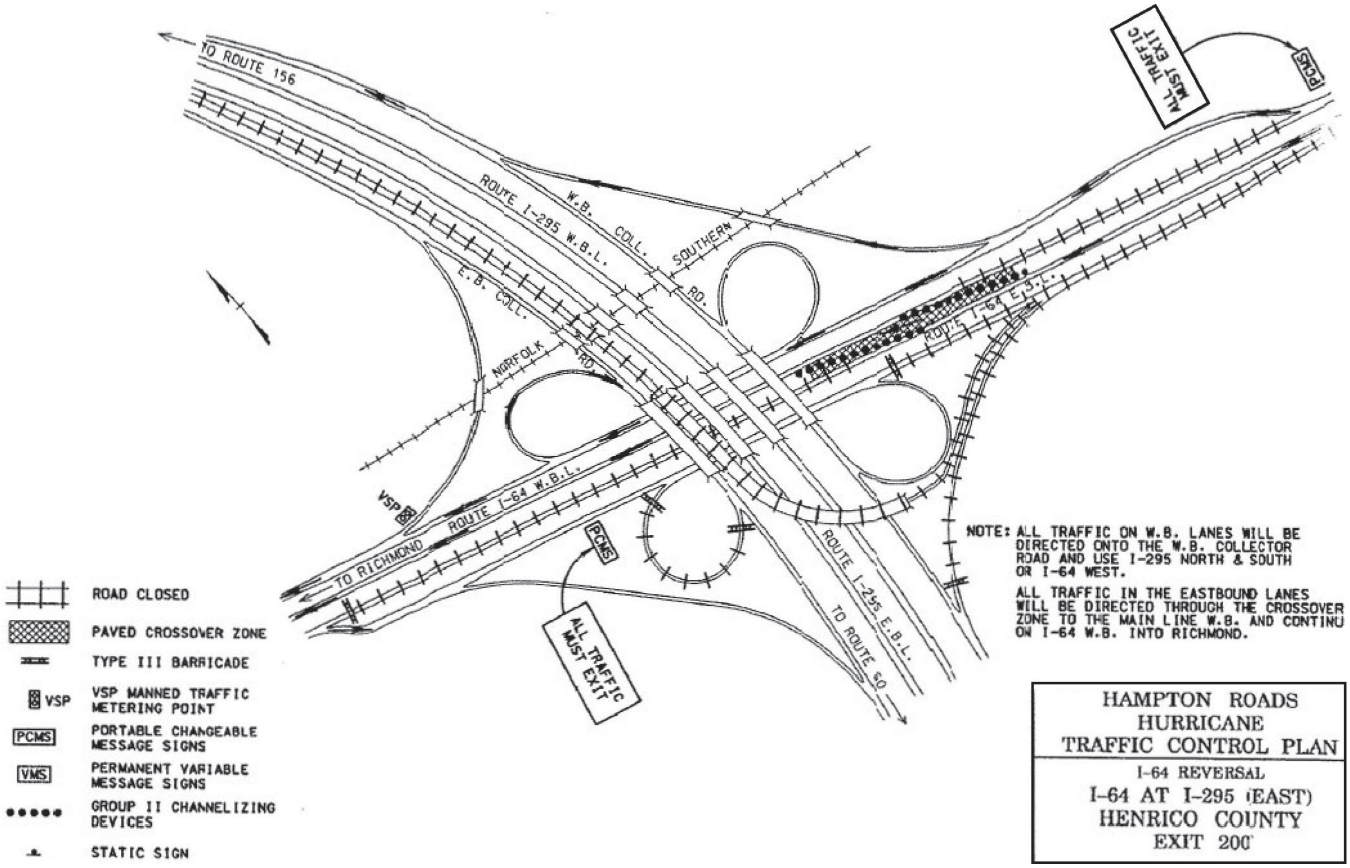

FIGURE 4 I-64 contraflow termination plan (SOURCE: Virginia DOT).

TABLE 1 Contraflow Termination Configuration and Design by Location

State	Route(s)	Termination Configuratio	Design
Virginia	I-64	Merge	Median crossover
North Carolina	I-40	Split	Reversed on-ramp
Georgia	I-16	Merge	Median crossover
Florida	I-10 westbound	Merge	Reversed on-ramp
	I-10 eastbound	Merge	Reversed on-ramp
	I-4	Merge	Median crossover
	I-75 southbound	Merge	Median crossover
	I-75 northbound	Merge	Reversed on-ramp
	FL Turnpike	Merge	Median crossover
Alabama	I-65	Split & merge	Median crossover
Louisiana	I-10 westbound	Split	Median crossover
	I-10/I-59 (east/north)	Merge	Median crossover
Texas	I-37	Merge	Reversed on-ramp

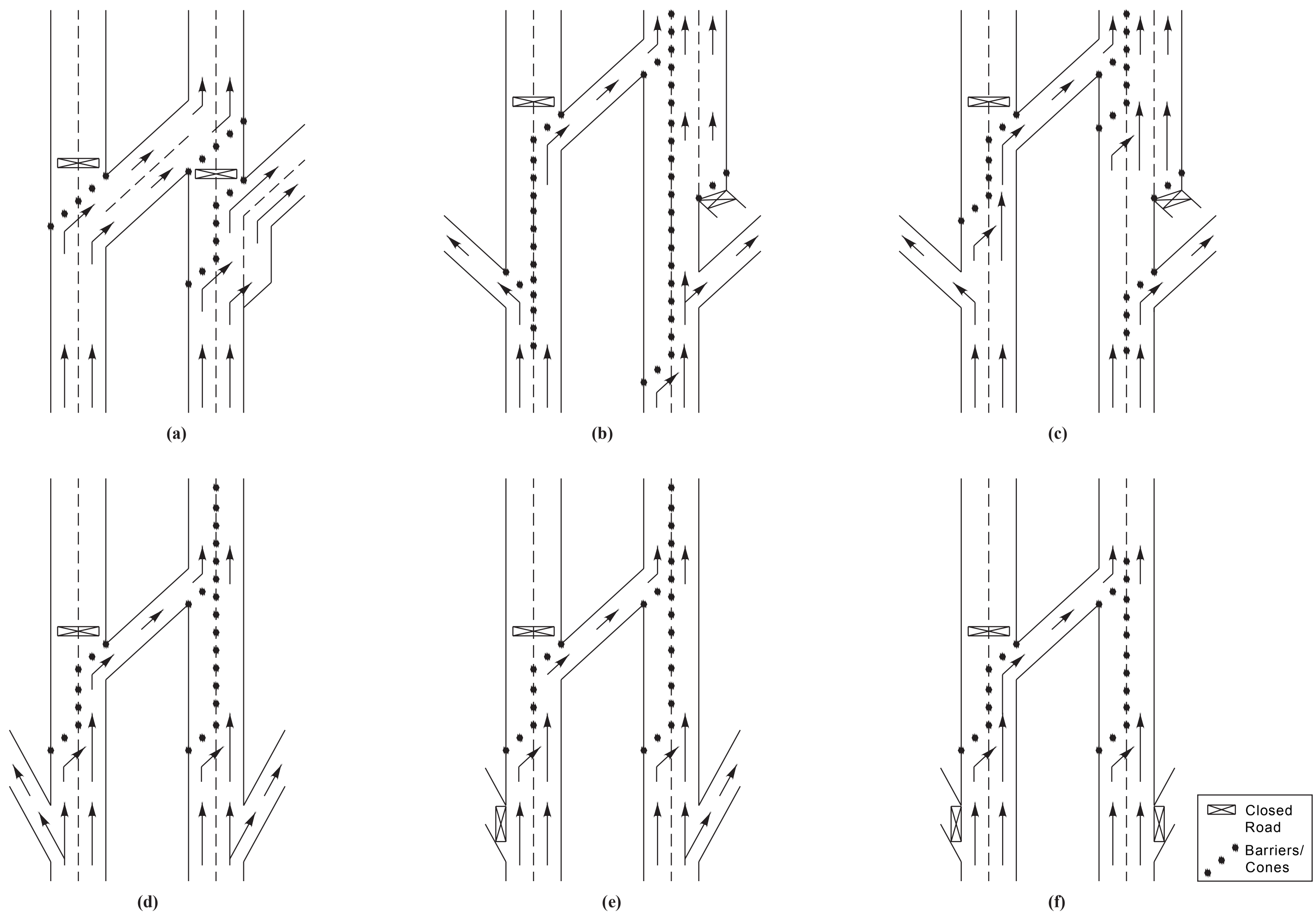

FIGURE 5 Schematic termination point designs (*3*): (*a*) Type A, (*b*) Type B, (*c*) Type C, (*d*) Type D, (*e*) Type E, and (*f*) Type F.

and the distance between the crossover point and the last upstream exit point. Although not apparent in the diagrams, the Types A, B, and C configurations also each had the associated exits within 1 mi of the crossover, and Types D and E had crossover–interchange separations of greater than 6 mi. Type F had no exiting opportunities along its length, so any traffic that entered this type of segment ultimately would have to exit the system at this point. Although this configuration is no longer planned by any state, it was planned for use as recently as 2002, so it was maintained as part of the study for comparison purposes.

With the exception of blocked access or reversed operations on the contraflow ramps, none of the current plans call for any traffic control along the intermediate segment. Traffic merging and exiting at the termini is expected to be controlled by using temporary traffic control devices (typically cones and barrels) supplemented by on-site police or, in some cases, National Guard personnel. To model the traffic operations in these areas, lanes closures were coded into the CORSIM models by using a permanent blockage incident that extended 1,550 ft upstream of the crossover and exit points. Advanced warning signs were also coded into the model to alert drivers of the impending merge 1 mi before these lane drops. Another key assumption of the models was that all the roads that received exiting volume had adequate capacity to accommodate. It is recognized, however, that this may not always be true.

Key Assumptions

To model the configurations, a number of assumptions were needed about the representation of contraflow lanes, the expected actions of drivers during an evacuation scenario, and the enormous volume that would be generated during such an event. CORSIM does not explicitly support the construction of reversible lanes or the behavioral conditions of drivers operating in evacuation conditions. It also has limitations on the volume that could be generated at the source nodes as well as the allowable network size.

To create the contraflow freeway lanes, reverse-flowing ramps, and crossovers, links were oriented to flow in the opposite direction adjacent to the normal outbound lanes. Since the lane drop merge areas near the termination will not be a permanent design feature (they will be created by using temporary traffic control devices), lane blockage incidents were used in CORSIM to model the essence of the operations expected to be present in these areas.

Free-flow speeds were adjusted on these various facilities. It has been widely assumed (although never conclusively proved) that drivers in contraflow lanes will tend to drive more cautiously compared to normal flow lanes because of their unfamiliarity with this type of operation and that this would be manifested by a general decrease in operating speeds. To account for this theory in the models, the free-flow speeds in all contraflow lanes were decreased from 70 to 65 mph. Free-flow speeds were set at 45 mph in the crossovers and 35 mph on the ramps. These speed values were based on design speeds used for the design of these facilities, a prior evacuation simulation in Texas (*4*), as well as studies of reverse-flowing traffic in Louisiana (*5*) and Washington, D.C. (*6*).

The models included several other key assumptions. It has been theorized that drivers will tend to remain in the normal flowing lanes if given a choice. To deal with such possibilities, some states, such as South Carolina, have developed strategies to equalize the loading of evacuation volumes into the two sides of the freeway. In Alabama, intermediate crossovers have been constructed to permit vehicles in the normal lanes to cross into the contraflow lanes along the segment. For these reasons, 55% of the total evacuation traffic was assumed to enter the segment in the normal lanes and the other 45% in the contraflow lanes in this study. Another assumption was a heavy vehicle proportion of 15%. Although FHWA statistics show that the heavy-vehicle (combination trucks with three or more axles) percentages on the Interstate system typically are around 7% to 8% (*7*), the 15% value accounts for the tendency of evacuees to take various recreational vehicles and pull trailers loaded with various property and possessions.

Another important series of assumptions were associated with the level of traffic that would be generated in the models. Empirical observations of evacuation contraflow in South Carolina suggested that a four-lane freeway would have a capacity of 5,000 vehicles/h (vph) (i.e., 1,500 vph in each of the normal flow lanes and 1,000 vph in each of the contraflow lanes). To determine what volume would produce similar conditions in this research, a series of pilot test runs were completed by using entry volumes between 4,000 and 8,000 vph. These showed that an entry volume of 6,000 (rather than 5,000) vph resulted in the speed and queuing conditions observed in prior evacuations. To generate vehicles at this rate, it was necessary to adjust the CORSIM default setting for "minimum separation for generation of vehicles" from 1.6 to 1.4 s to increase the amount of volume entering the system.

MOEs and Experiments

The experiments and MOEs that were developed in this research were developed and selected primarily on the basis of the concerns of emergency management and transportation officials in the locations in which they are planned. In general, these agencies are most concerned with the total number of people that can get out during an evacuation period, how long they are expected to be delayed, and how long congestion queues are expected to extend (temporally and spatially).

To evaluate the effect of varying levels of traffic volume at the termination point, the models were adapted to reduce the volume entering the termination area by one-quarter or one-half from the contraflow lanes, normal lanes, or both. These scenarios, summarized in Table 2, were designated Types B_{25}, B_{50}, C_{25}, C_{50}, D_{25}, D_{50}, E_{25}, and E_{50} on the basis of the level of traffic reduction. Although it would have been ideal to model more specific conditions likely to occur at the study locations, the conditions of where, when, and how much traffic will exit or enter along any of the intermediate contra-flow segments remains unknown. The generalized volume reductions and exit locations used in this study are, however, suitable for broad comparisons of the relative effects of decreased volume at the termination.

The output data produced by CORSIM fit into one of two general categories: those at the systemwide level and those that were link specific. Systemwide statistics were generally used to assess the general performance characteristics of the entire network. In this research, the analyses of networkwide statistics were used to compare the overall operation of the individual configurations at varying levels of traffic levels and comparatively rank them. Link-specific statistics are more useful for analyzing conditions at specific points. Here, they were used to evaluate operations within specific sections of the segment, including the lanes immediately before the crossover, merge areas prior to lane drops, the upstream lanes several miles before the terminal, and the mainline and ramp roads around the exits. Link-specifi data were also used to compare operational differences between the normal and contraflow lanes.

A total of 23 different cumulative link-specific and networkwide average statistics generated by CORSIM were used in the research. These statistics were then used to calculate the total number of vehi-

TABLE 2 Exiting Traffic Percentage at Upstream Interchange (*3*)

Model Type		Number of Lanes on Median Crossover	Exiting Traffic at the Previous Interchange Within 1 m Ahead of Median Crossover (%)		Exiting Traffic at the Previous Interchange Within 6 m Ahead of Median Crossover (%)	
			Reverse Direction	Normal Direction	Reverse Direction	Normal Direction
Type A		2	—	100	—	—
Type B	Type B_{25}	1	50	25	—	—
	Type B_{50}	1	50	50	—	—
Type C	Type C_{25}	1	25	50	—	—
	Type C_{50}	1	50	50	—	—
Type D	Type D_{25}	1	—	—	25	25
	Type D_{50}	1	—	—	50	50
Type E	Type E_{25}	1	—	—	—	25
	Type E_{50}	1	—	—	—	50
Type F		1	—	—	—	—

cles exiting the network, vehicle speed, delay time, volume, density, and move-to-delay-time (M/T) ratio for each configuration model. Each of these sets was then, in turn, used to perform the comparisons between each of the design configurations by using a series of statistical tests that both compared the differences and ranked relevant performance measures. The networkwide MOEs included the total vehicle miles of travel, total time spent moving (vehicle hour), total delay time (vehicle hour), average travel speed (mph), ratio of time spent moving to total time in the system, delay time (minutes per vehicle), and total time spent in each system (minutes).

Thirty separate cases using different seed numbers were executed for each of the 10 models for a total of 300 cases. Each case reflecte a 4-h period and output statistics were collected at the end of 16 15-min periods. The average values from each of these case groups were used for the statistical comparisons summarized in the following section.

RESULTS AND ANALYSES

The following sections summarize the primary findings of the research and highlight some of the particularly interesting results. In this paper, only the results of the comparisons of the models, volumes, and lanes from the 10 models are discussed. A more complete discussion of the study results is available in the full project report (*3*).

Comparison of Alternative Designs and Varied Volume

Models A, C_{50}, and D_{50} consistently outperformed the other models in nearly all performance measures, and C_{50} was the best performer in all categories except total time in the system. On average, the A, C_{50}, and D_{50} models were able to maintain operating speeds at or above 32 mph and kept vehicles moving more than half the time. By contrast, models F, E_{25}, E_{50}, and D_{25} each had average operating speeds below 10 mph and vehicle stoppages more than 70% of the time. These find ing are not surprising and are intuitively logical because these configurations minimize merging prior to the crossover and, in the cases of C_{50} and D_{50}, remove half the traffic volume. Interestingly, however, the performance of models B_{25} and B_{50} was only marginally better than that of the D, E, and F groups, even with a traffic decrease. This appeared to be because merging maneuvers in the B configuratio took place before the exit ramps, rather than after the exit ramps, as was the case in Models C and D, in which densities were lower and merging opportunities greater. This preexit merge meant that traffic queued for some distance before the crossover.

The results of the number-of-vehicles-processed measures were also consistent with the earlier findings. Again, the A, C_{50}, and D_{50} models showed the best performance, and C_{25} was close behind. One of the more interesting results was that although the A model maintained all lanes open, its average hourly flow rate (1,441 vph) was just below that of the C_{50} and D_{50} models (1,463 vph and 1,462 vph, respectively). At the opposite end of the spectrum, the F model, with no exits and lane drop merges on both the normal and contraflo lanes, had average hourly flows of just more than half these rates at 822 vph. Table 3 summarizes the results of the average hourly outflow, the total number of vehicles processed, the ratio of outflowin to inflowing vehicles in each model, and their Tukey ranking.

The values presented in Table 3 are also useful to illustrate two other issues of critical importance during an evacuation. Sum Total Vehicle Out shows the total amount of evacuating volume that exited the segment during the 4-h test period. In addition to ranking the models by this measure, the table shows the advantages of reducing traffic volume to increase the capacity of some of the configurations The Type C and D configurations actually had higher total output volumes (23,444 and 23,313 vehicles, respectively, at 50% of the full traffic volume) than did the Type A configuration (23,087 vehicles at full no-exit volume). Not surprisingly, the Type F configuration which featured merges on both the normal and the contraflow sides of the freeway, had the lowest total outflow rate of 13,307 for the 4-h test period. This meant that nearly half the vehicles entering the test segment in the F model were unable to depart. Assuming an occupancy rate of 2.5 passengers per vehicle and comparing it to the results of the F models, which processed 13,307 vehicles during the simulation period, the C_{50} model would be able to evacuate more than 25,000 more people from a threat area. Such results are hardly insignificant

TABLE 3 Comparison of Model Performance (3)

Tukey's Studentized Range (HSD) Tests								
Sum TP Vehicle Out			Sum Total Vehicle Out			Out/In Ratio		
Tukey Ranking	Mean[a]	Type	Tukey Ranking	Mean	Type	Tukey Ranking	Mean	Type
A	1,463	D_{50}	A	23,444	C_{50}	A	0.98	C_{50}
A	1,462	C_{50}	B	23,313	D_{50}	B	0.97	D_{50}
A	1,441	A	C	23,087	A	C	0.96	A
B	1,334	C_{25}	D	21,611	C_{25}	D	0.90	C_{25}
C	1,190	E_{50}	E	19,210	E_{50}	E	0.80	E_{50}
D	1,136	D_{25}	F	18,869	D_{25}	F	0.79	D_{25}
E	1,089	B_{50}	G	17,437	B_{25}	G	0.73	B_{25}
E	1,087	B_{25}	G	17,429	B_{50}	G	0.73	B_{50}
F	959	E_{25}	H	16,031	E_{25}	H	0.67	E_{25}
G	822	F	I	13,307	F	I	0.55	F

NOTE: Means with the same letter are not significantly different.
HSD = honestly significant difference; TP = time period.
[a]Average number of vehicles that exited out the test road segment during each of 16 h.

Normal Versus Contraflow Lane Comparison

Analyses were used to determine if there were any apparent differences between the normal and the contraflow sides of a freeway. There is a widely held belief that flow conditions in the contraflow lanes will be less efficient than those in adjacent normally flowing lanes. This is based on the idea that drivers will be more tentative because of the unfamiliarity of driving in an unintended direction, and, for example, operating speeds in the contraflow lane will be lower than those of the normal lanes.

Direct comparisons between the two lane groups in this research were somewhat complicated by a number of factors, including the traffic volume disparity between the two sides (45% in contraflo lanes and 55% in the normal lanes) and the control and exit config uration differences between the two sides. However, when the configurations were essentially the same (such as in the A and D models), the contraflow lanes were shown to maintain higher operating speeds and lower levels of congestion. This was not unexpected given their lower volume.

A more interesting set of results was apparent between the B and C models, where different control configurations were used to maintain separation between adjacent lanes near the exit ramps. The results strongly suggested a significant benefit can be gained from using lane channelization for the separation of traffic in adjacent travel lanes in the vicinity of exits. In the B models, lane separation in the contraflo lanes was initiated before the exit ramp and maintained through the length of the ramp. In the normal lanes, a lane drop merge was located before the exit. Under these conditions, average speeds in the contraflow lanes were nearly 60 mph, compared to about 6 mph in the normal lanes. In the C models, separation in the normal lanes was similar to the contraflow side of Model B, and a lane drop merge was present after the exit from the contraflow lanes. In the C models, operating speeds in the normal lanes averaged about 32 mph, whereas speeds in the contraflow lanes averaged nearly 12 mph when volumes were reduced by 25%. When traffic in the contraflow lanes was reduced to half its full volume, average speeds increased to 60 mph. This is illustrative of the significant benefits that can be attained by making even incremental reductions to the traffic volume arriving at the terminus of these segments.

Additional analyses comparing the operation of the lanes at varying volume levels further quantified these trends and showed the effect of the lane drops within the merge areas. The simulation results showed that the closure of a single lane (on either set of lanes) resulted in enormous increases in delay over nonmerge configurations. On average, travel times were increased to 137 min over the 13-mi test segments when a travel lane was eliminated. This represents an approximate 10-fold travel delay increase above the 13- to 14-min travel time in free-flow conditions.

Another quantification was of the gains that would be apparent from decreasing the level of evacuating traffic volume arriving at the termination point. In practice, this could be accomplished in a number of ways. The most practical would be to allow vehicles to use exits along the intermediate segment of the evacuation route. The study results showed that when traffic volumes were decreased by 25% under the highest volume scenario, the travel delay associated with the lane drop merge was reduced between 20% to 60%. The gains that were observed were also lane dependent, with average decreases of 115 min in the normal lanes and 67 min in the contraflow lanes. Although these delay reductions were significant, they nevertheless remain four to eight times higher than for similar volumes in the nonmerge configurations The delay effect of volume was even more pronounced at the 50% reduction level. When arriving volumes were cut in half, the delay associated with the lane drop merge decreased by 80%. However, this delay is still double that of a comparable no-merge configuration

When the general relationship of traffic volume is plotted against its corresponding travel delay resulting from the lane drop merge, it is evident that delays and travel times increase fairly rapidly once traffic volumes begin to exceed half the maximum flow volumes (Figure 6). This strongly suggests that the use of intermediate exits throughout the length of the segment to diminish traffic volumes at the termination of the contraflow evacuation segment would be advantageous.

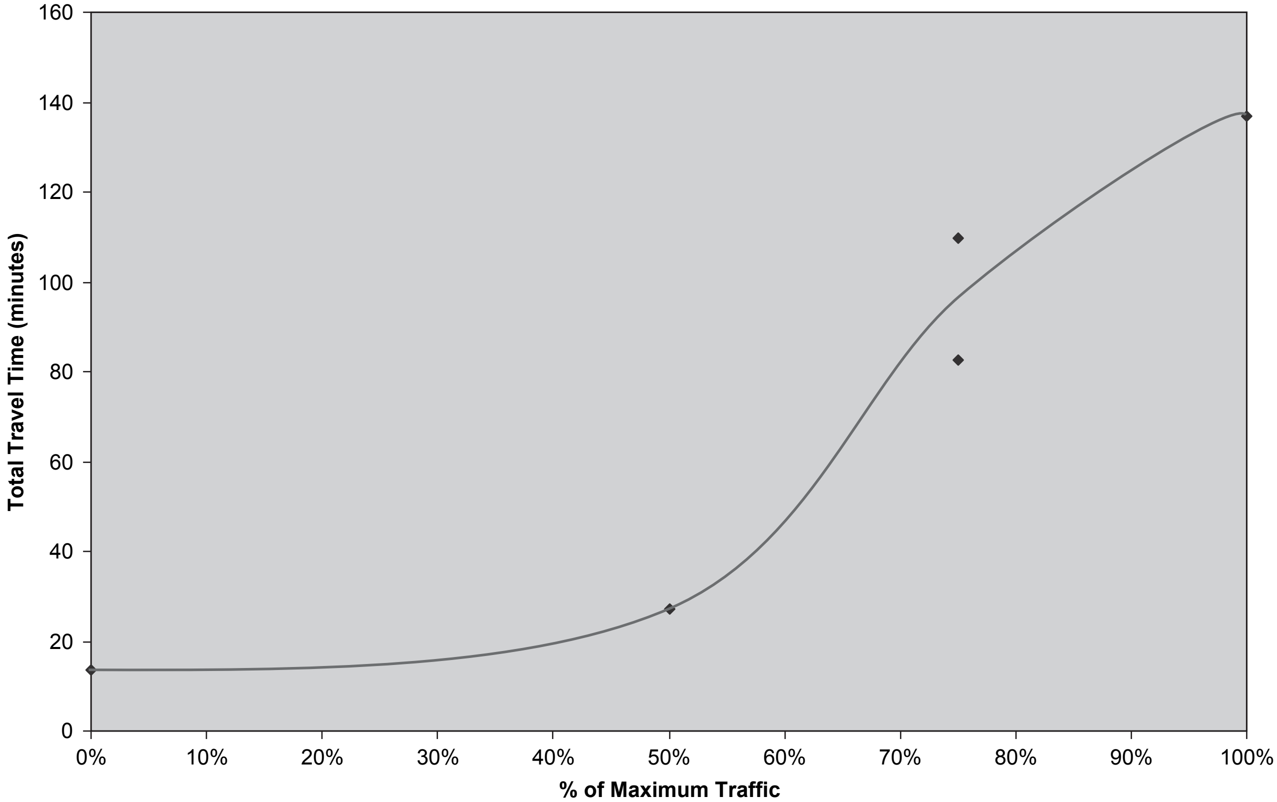

FIGURE 6 Relationship of traffic volume and travel time on test route.

SUMMARY AND CONCLUSION

Twenty evacuation contraflow segments are planned for use in the United States. When activated for use, these routes will serve as lifelines for people as they flee the potential destruction of approaching storms. The termination points are particularly important components of these segments because they can regulate the amount of volume that can exit the section. Interestingly, the design and management of flow in these terminations vary significantly. However, since all but one has never been put into practice, it is unknown how well, or even if, any of them will work. Perhaps even more significant, no one has examined how, or if, they may be improved before they are needed. The research effort described in this paper was undertaken to assess and compare the operational characteristics of contraflow evacuation termination point designs that have been planned although not yet used in the United States.

With these issues in mind, several objectives were developed for the study. The first was to develop an approach that would permit contraflow operations under an evacuation scenario to be coded into a standard traffic simulation program like CORSIM. A key element of the modeling was to develop a set of assumptions to account for reverse-flowing traffic on a freeway under evacuation conditions. The second was to characterize and quantitatively assess the flow and delay characteristics in the vicinity of the termination of these segments at the networkwide level and at key locations, such as near the termination crossovers and ramps. The third and final objective reported in this paper was a comparative assessment of their performance. The comparisons were made not to advocate one design over another but rather to identify or quantify the factors that made some designs more effective than others, including the effect of reducing traffic volumes before the termination.

Ten models were developed by using a number of assumptions, including reductions in operating free-flow speeds within the crossovers and ramp areas. Apart from the reorientation of the contraflo links, very few changes were made to represent the contraflow lanes in the models. All these assumptions were based on experiences under similar or analogous nonemergency conditions, including the use of reversible traffic operations for peak period commuter traffic and planned major events. No allowances were made for driver panic since there is little research or empirical observation to support the existence of such conditions. Since no field data are available from any of the sites, none of the assumptions used in the study could be validated with actual field observation. The experimental results of the study have, however, been presented to numerous traffic professionals in the states in which they are planned, and they have not raised significant objections to the assumptions and methodologies used.

The results of the simulations showed an interesting relationship between volume and delay, including the delay increases associated with merge configurations and the delay reduction that could be expected from incorporating exits before the termination. Not surprisingly, split configurations (Type A) consistently yielded higher levels of performance than the merge configurations. In general, the closure of a single lane on either side of the freeway (normal or contraflow) resulted in an approximate 10-fold increase in the amount of travel delay over the 13-mi test segment. Volume decreases of 25% before the termination reduced the delay associated with the merge lane drop by between 20% and 60%, depending on the configuratio type. While this was a significant improvement, it was nonetheless a four- to eightfold increase over the split (no-merge) configuratio delays. A 50% decrease in traffic volume prior to the termination reduced the merge-associated delay by 80%. This was, however, still a twofold increase over the delay in the split configuration

The reduced traffic volume, and accompanying lowered delay, also helped to increase the capacity of some of the configurations. The C and D configurations had higher total output volumes (23,444 and 23,313 vehicles, respectively) at 50% of the full traffic volume than did the Type A configuration (23,087 vehicles) at full (no-exit) volume. The Type F configuration, which featured forced merges on both the normal and contraflow sides of the freeway, had the lowest total outflow rate (13,307 vehicles) over the 4-h test period. The data also showed that maximum outflows were maintained at average travel speeds between 20 and 30 mph.

The ranking of the various configurations was difficult to assess because of the number of different variables. In general, however, it was apparent that the Type A models consistently outperformed the other configurations. The C and D models also outperformed Type B models. The improved performance appeared to be related to the location of the lane drop relative to the exit. In the C and D configurations the merge took place after an exit location, whereas the merge took place before an exit ramp in the B models. The Type F merge config uration consistently performed worse than all other configurations in nearly every MOE category.

The results of this research suggest some important trends relative to the use of contraflow evacuation segments. First, it is advantageous to maintain all lanes through the termination point by using split designs. Although route choice may be limited by such designs, split configurations eliminate the flow turbulence associated with merge configurations. They can also maintain high levels of effectiveness at high entry volumes. Another finding is that the best way to maintain high operating levels and reduce delays resulting from the termination is to reduce the volume entering the termination point. Although this can be achieved simply by maintaining exit points along the route, the ability of DOTs to accomplish this with few traffic enforcement personnel may be difficult. Two other concepts suggested by the results were to merge traffic after the exits rather than before them in merge designs and that the quality of the flow through the termination vicinity can be enhanced by the use of channelization or separation devices well in advance of forced maneuvers by separating exiting vehicles and reducing conflicting maneuvers.

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Methodology to Establish Hurricane Evacuation Zones

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A review of practice in hurricane evacuation modeling reveals that the criteria determining the delineation of hurricane evacuation zones have not been clearly defined. In addition, there is no recommended procedure with which to establish hurricane evacuation zones once criteria have been accepted. A set of criteria has been adopted in this paper to design a procedure that mechanically establishes a recommended set of hurricane evacuation zones for an area. The procedure, which is based on a geographic information systems platform, is described, and its use is demonstrated for establishing hurricane evacuation zones for the northern part of the New Orleans, Louisiana, metropolitan area on the north shore of Lake Pontchartrain. The procedure can be applied to any area, and although it is specifically directed at identifying evacuation zones for hurricanes, it could be used for any emergency in which flooding is the major hazard, or it could be adapted to other emergency situations for which evacuation is an appropriate response.

Hurricane evacuation zones are areas in which inhabitants are at risk from flooding or high winds caused by a hurricane. Hurricane evacuation zones are used primarily for estimating evacuation demand, but they could also be used by emergency managers to target those at risk while omitting those not at risk, thereby reducing congestion on evacuation routes and limiting the consumption of scarce refuge space.

Hurricanes form from areas of low pressure over warm oceans where surface temperatures generally exceed 80°F (*1*). They form between June and November in the northern hemisphere and are classified by wind speed within the storm. Storms with wind speeds below 38 mph are termed tropical depressions; those with wind speeds between 39 and 73 mph are tropical storms; and hurricanes are identified as Category 1 to Category 5 hurricanes when wind speeds reach 95, 110, 130, 155, and >155 mph, respectively.

Most fatalities during a hurricane are caused by drowning. Flooding can be caused by precipitation, but most inundation in coastal areas is caused by storm surge. Storm surge results from an elevation of the water surface due to the wind and the reduced air pressure at the center of the storm. Storm surge can be estimated with the use of models such as the SLOSH (sea, lake, and overland surges from hurricanes) model or the ADCIRC (a parallel advanced circulation model for oceanic, coastal, and estuarine waters) models. SLOSH uses estimates of the barometric pressure, size, forward speed, track, and wind speed of a hurricane to predict storm surge in the path of the storm. Estimates of water elevation are made in a set of grid cells that vary in size away from the point of landfall. Ground levels must be subtracted from estimated water levels in each grid cell to estimate the depth of inundation. Levees and other topographic features that prevent the free flow of water to the lowest point must be taken into account when flood areas are established.

The maximum water level achieved through storm surge, irrespective of when it occurs, forms an envelope of maximum water surface elevations for a particular storm. The point of landfall of the storm is very influential in determining these elevation. Subsequently, a series of SLOSH model runs are often made in which only point of landfall is allowed to differ while all other characteristics of the storm are kept the same. The maximum water elevation resulting from these runs is called the maximum envelope of water (MEOW) and is used to establish the likely surge levels resulting from a particular storm whose point of landfall is uncertain. By increasing the intensity of the storm at the same time, the maximum of the MEOWs (MOM) is obtained for a storm whose point of landfall and ultimate intensity are unknown.

MEOWs and MOMs are overlaid on digital elevation models (DEMs) to determine areas of inundation and depth of inundation. If hurricane evacuation zones and hurricane evacuation routes are superimposed on these inundation maps, zones that should be evacuated and routes that may be at risk of flooding can be identified However, no clear method exists to establish evacuation zones. The research reported in this paper is aimed at developing a methodology for establishing hurricane evacuation zones.

CURRENT PRACTICE

Hurricane evacuation zones are established with personal judgment based on principles or criteria that ensure that the resulting zones comply with certain requirements. This is similar to the approach used in establishing traffic analysis zones in urban transportation planning (*2*). However, unlike urban transportation planning, there has not been a clear enunciation of the principles or criteria underlying the establishment of hurricane evacuation zones, although some studies do list some. For example, in a study aimed at providing technical assistance to state and local governments, the Federal Emergency Management Agency, the U.S. Army Corps of Engineers, and the National Oceanic and Atmospheric Administration, in collaboration with Delaware state and local emergency management agencies, suggested that the following principles should govern the establishment of hurricane evacuation zones in each county (*3*):

- Zones should relate to expected surge flooding limits (based on MEOWs) for each storm scenario.

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- Zones should relate well to census, traffic analysis zones, or other database units.
- Zonal boundaries should include identifiable natural features, roadways, landmarks, and so forth.
- Rural counties should have no more than 20 zones, and counties with major urban areas should have no more than 35 zones.
- Zones should be set up where possible for use in emergency management operations.

In an earlier study, the Army Corps of Engineers had suggested that evacuation zones should also be established to have relatively balanced populations and should be constructed so that they could be conveniently served by major evacuation routes (*4*). In a later study, the Corps suggested that evacuation zones also should be made to conform to identifiable geographic features such as streets, railways, and other manmade land features (*5*). In a study conducted for Southeast Louisiana, an additional criterion or consideration was that the establishment of hurricane evacuation zones should allow for appropriate transportation modeling (*6*). In the *Treasure Coast Region Hurricane Evacuation Study,* the important criterion that evacuation zones must be easily identifiable by verbal description (so that inhabitants can respond to verbal evacuation orders) was mentioned, together with the requirement that areas isolated by surrounding surge should be included with evacuation zones of the inundated areas (*7*).

PROPOSED PROCEDURE

From information gathered in the literature review, and from discussions with experts (E. J. Baker and D. Lewis, unpublished data), the following principles are suggested to guide establishment of hurricane evacuation zones:

- Hurricane evacuation zones should be areas of uniform elevation.
- One hurricane evacuation zone may not be established within another.
- Hurricane evacuation zones are established for the area extending up to the maximum surge flood limit only.
- Zones should be easily identifiable by verbal or written description.
- Zones should be as homogeneous as possible in their land use.
- Zones should not straddle parish boundaries.
- Zonal boundaries should include features such as major roads and landmarks.

These principles capture the desired features of hurricane evacuation zones identified in the literature. Some desired features are not directly served by the principles because use of technology, such as geographic information systems (GISs) in the establishment of hurricane evacuation zones, makes them redundant. For example, under normal circumstances it is highly advantageous to establish evacuation zones as aggregates of whole census tracts, so that evacuating populations can be easily estimated. However, a GIS readily estimates the properties of bisected areas, thereby making redundant the principle that hurricane evacuation zones should be made up of whole census tracts.

The requirement that hurricane evacuation zones should be easily identifiable by verbal or written description arises from the need of emergency managers to direct evacuation by verbal or written directives. Means of identifying areas through easily understood verbal or written communication include the use of zip codes, telephone exchange areas, and subdivision names. To this can be added orientation to landmarks, rivers, mountains, or major highways. For example, residents would easily understand a command that targeted a certain zip code area south of a major highway or landmark.

The benefit of hurricane evacuation zones that are as homogeneous as possible in their land use is that different land uses have different evacuation potential. At one extreme, land uses involving no human habitation (nature reserves, lakes, marshes, etc.) generate no human evacuation. At the other extreme, residential areas are the main source of evacuation traffic from emergencies with long warning times, such as hurricanes, because people usually return home first from their other activities before evacuating an area.

The listed principles were used to establish a methodology that progressively works toward delineating a set of hurricane evacuation zones in an area while satisfying the principles to the greatest possible extent. The procedure starts with identification of the study area and ends with a set of hurricane evacuation zones. The process is summarized in Figure 1. The first step in the procedure is to establish the flood limits for the area under study. This involves identifying MEOWs for the most severe storms that are likely to strike the area. The surge estimates must be used together with topographical information to establish the limit of the flood area and, subsequently, the study area.

The second step is to use land use information to identify areas within the study area that are uninhabited and therefore should be excluded from the analysis. This includes areas such as lakes, swamps, parks, nature reserves, cemeteries, sports fields, and any other area where humans do not reside or work. Although evacuation from approaching hurricanes is an activity that usually takes place from home because of the relatively long warning time associated with hurricanes, work places are included within evacuation zones as it may be necessary for employers to respond to flooding by moving equipment, securing material against damage, and preventing pollution from materials at the site.

The third step is to identify zip code areas within the study area. Their use is recommended in preference to other areal descriptors because GIS boundary files of zip code areas are readily accessible and they are well known, easily understood, and do not change. In contrast, telephone exchange areas are more difficult to acquire in GIS format, and these change as new exchange areas are established or altered. In addition, subdivision names and boundaries are usually not as well known as a zip code or telephone number by residents.

When zip codes are used to establish recognizable areas within the study area, the five-digit zip code should be used without the four-digit extension, since the extension is not well known. Zip code areas minus the uninhabited areas identified in the second step are considered in the remainder of the analysis.

The fourth step is to identify the main roads in the area and specifically any evacuation routes serving the area. The roads are used to subdivide the zip code areas minus the uninhabited areas into smaller areas that are identified by referring to both a zip code and an orientation to one or more roads subdividing the zip code area. This activity establishes the smallest areas that lend themselves to verbal or written description. The subareas form the basic building blocks from which hurricane evacuation zones are built. This activity also serves to establish evacuation zones that are bordered by evacuation routes

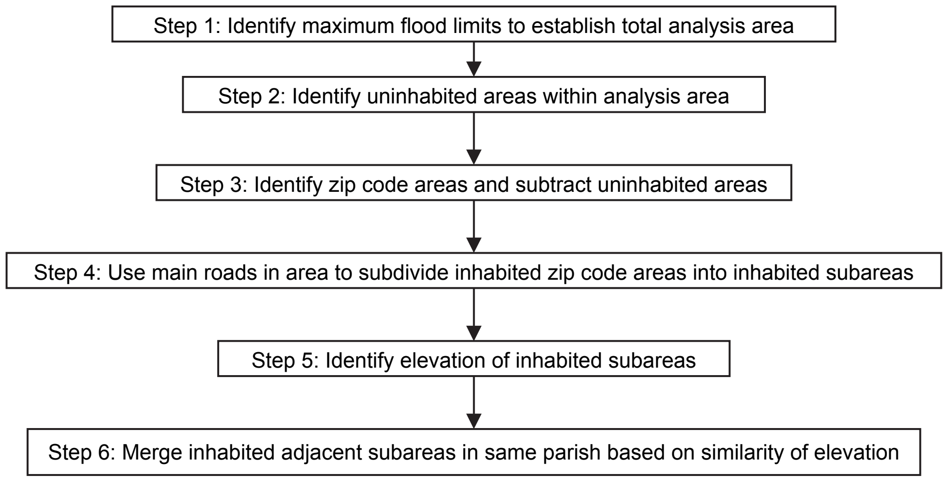


FIGURE 1 Procedure to establish hurricane evacuation zones.

and, thereby, establishes a zonal system that can be conveniently served by the evacuation network.

The fifth step involves identifying the average elevation of each subarea as well as the variance of uniformly positioned elevation points within each subarea. The mean and variance provide the statistic by which subareas are assessed for possible aggregation. It is a process that is repeated as subareas are aggregated into larger areas and the average elevation and variance of these larger areas are recalculated after each merge.

The sixth step is the progressive merging of subareas to form evacuation zones. This is achieved by progressively combining adjacent subareas with the most similar elevations, provided they belong to the same parish. Similarity of elevation is judged on the similarity of the average elevation in each subarea among those subareas that have uniform elevations. This is operationalized by ranking subareas by the variance of their elevations and seeking the subarea pair with the most similar average elevation among the subareas with variances in, say, the lowest decile. Once a subarea pair is identified in this manner, it is combined to form a new zone, and the average elevation and variance of the new zone are determined to begin the next cycle of subarea aggregation. With each iteration, the subareas are progressively aggregated until the required number of evacuation zones is obtained or some closing criterion on the difference in elevation between zones is satisfied

BUILDING THE PROCEDURE ON GIS PLATFORM

The described process has been operationalized on a GIS platform. This facilitates execution and permits convenient graphic output of the results. TransCAD was selected as the main platform on which to build the process. Parts of the process are conducted manually, whereas the iterative portion of the process in the fifth and sixth steps is performed automatically by using a program written specifically as an add-in to TransCAD.

To establish the limits of the study area, both storm surge information and topographic information are required. Storm surge is estimated externally by using models such as SLOSH or ADCIRC. Topographical information can be obtained from several sources but must provide elevation estimates in a regular raster form. Digital elevation models (DEMs) are data files that provide digital representations of cartographic information at regularly spaced intervals. These data files are available from the U.S. Geological Survey (USGS) as part of the National Mapping Program. DEM data for 7.5-min units correspond to the USGS 7.5-min topographic quadrangle map series for all of the United States and its territories except Alaska. Each 7.5-min DEM is based on a 30- × 30-m data spacing with the Universal Transverse Mercator projection (*8*). An alternative source of elevation information is lidar (light detection and ranging), which uses reflected radar signals from a low-flying aircraft to estimate elevations. In many cases, it represents a cost-effective method for acquiring digital elevation data and has a significan advantage over conventional aerial photography in that it can be acquired at night and in cloudy or hazy weather conditions. In a GIS, lidar data can be displayed in grid format or as contour lines. Once lidar data are in grid format, additional products, such as DEMs, can be generated (*9*).

To establish uninhabited areas, land use data files can be downloaded from the U.S. Environmental Protection Agency's (EPA's) website, www.epa.gov. The data describe the environmental land use of the entire country and allow distinction between urban or built-up land from agricultural land, rangeland, forest, lakes, wetlands, barren land, tundra, perennial snow, and glaciers. The urban or built-up land includes all residential, commercial, industrial, transportation, communications, utilities, and mixed urban or built-up areas, and the remainder represents uninhabited areas. If other land use data are available to distinguish residential from industrial and commercial areas, this information is retained for informational purposes but is not used in the remainder of the process to establish hurricane evacuation zones.

Zip code boundary information is usually included in the data accompanying GIS systems. If not, shapefiles of the data can be transferred from other GIS systems or purchased from commercial sources if necessary. Once established, the uninhabited areas are subtracted from the zip code areas manually by using overlay procedures in TransCAD.

Highways are used to subdivide zip code areas into smaller and yet still recognizable subareas. It is also possible to use landmarks or geographic features such as rivers or canals to subdivide zip code areas. This process is conducted manually to permit personal judgment to play a role in the establishment of the subareas. However, the process is greatly facilitated by being able to superimpose the highway, landmark, and other geographic features on the zip code area layer in a GIS while establishing the subareas. Highway, landmark, and other geographic data are available from the data provided with TransCAD software.

Census tract boundaries as well as the population associated with each census tract are available in TransCAD. Census tract boundaries and census values are also available from the U.S. Census Bureau as well as from the websites of prominent GIS vendors, such as the Environmental Systems and Research Institute (www.esri.com). Overlay features in GIS are used to estimate the population in each inhabited subarea.

The inhabited subareas form the basic building blocks for the establishment of the hurricane evacuation zones. They qualify for this role because they are the smallest areas that are easily identified by the public through verbal or written communication. They could be used as individual hurricane evacuation zones, but it is likely that some of them could be combined to form larger evacuation zones of uniform risk and result in a more manageable number of zones. In the procedure suggested in this paper, a process has been suggested that looks at the average and variance of elevation readings in each subarea, combines the two subareas most similar to each other, estimates the average and variance of elevations in the combined subarea, and then iterates through the procedure again to progressively combine the next two most similar areas (including combined subareas from earlier iterations if necessary). Since the estimation of the average and variance of the elevations in each subarea is onerous, and the iterative nature of the process can require many iterations as the initial set of subareas are merged into a reduced number of zones, the process was automated by using the programming language GISDK in TransCAD.

In the program, the elevation point file obtained from a DEM is overlaid on the inhabited subarea layer, and the mean and standard deviation of the elevation points in each subarea are calculated. Zones that have in common at least one point on their periphery are identified, and their joint standard deviations, s, are determined [$\sqrt{\left(s_i^2 + s_j^2\right)/2}$ values for adjacent zones i and j]. So as to combine only those subarea pairs with uniform elevations, the subarea pair with the smallest difference in average elevation is selected from among the pairs within the 10th percentile of joint standard deviation values only. This merges adjacent subareas whose average elevations are most similar provided their variance in elevation values is small. The process is repeated in an iterative fashion, combining two subareas in each iteration, until a closing criterion is attained. In this study, several closing criteria were tested, but the best results were obtained from the average standard deviation of the merged zones.

TEST APPLICATION

A test application of the process to establish hurricane evacuation zones was conducted on the north shore of the New Orleans, Louisiana, metropolitan area. New Orleans, on the southern bank of Lake Pontchartrain, is surrounded by levees, making redundant the establishment of individual areas subject to flooding from different storm surges. A storm surge capable of overtopping the levee system would flood most of the area within the levee system. In contrast, the northern portion of the metropolitan area does not have levees, and the establishment of hurricane evacuation zones is appropriate. The area considered for the test application is shown in Figure 2.

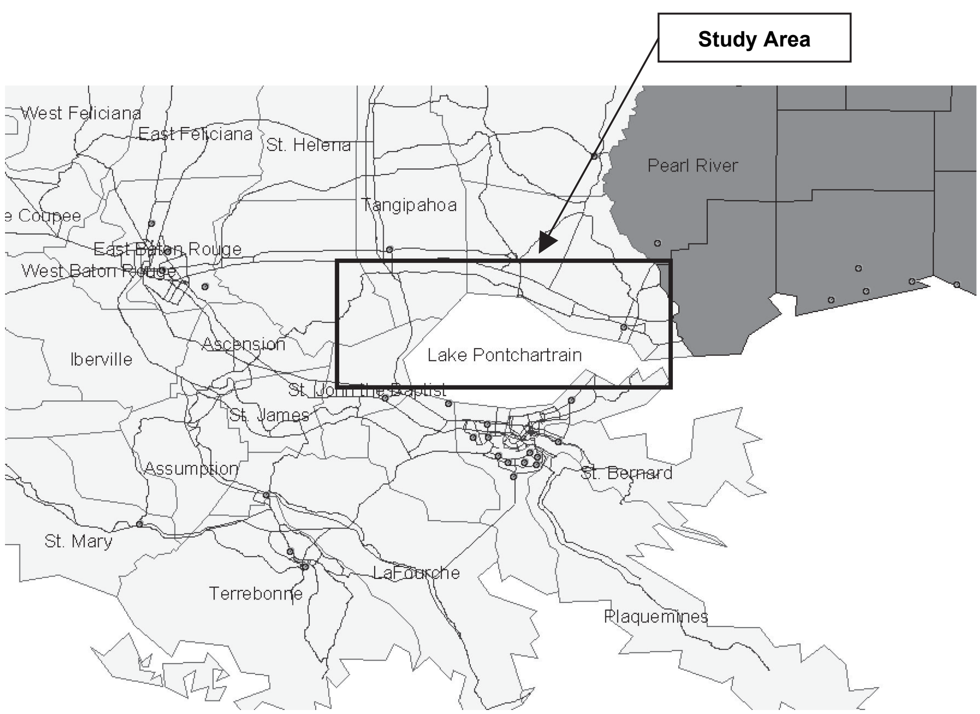


FIGURE 2 Study area.

The SLOSH model was used with several storm scenarios to identify the area that would be flooded as based on the scenarios considered. The scenarios included all combinations of the following storm characteristics:

- Storm Categories 2 and 5;
- Hurricanes approaching on paths from the southeast, south, and southwest; and
- Forward speeds of 5 mph and 15 mph.

Thus, 12 (2 × 3 × 2) storm scenarios were evaluated to identify MEOWS for each scenario. Ground elevations were obtained from USGS DEMs, and it was determined that an area extending 10 to 20 mi inland from the shoreline of Lake Pontchartrain, as shown in Figure 2, should form the study area as the storm surge from the anticipated storms would not flood beyond this area. Land use data were obtained from the EPA website cited earlier. The highway network, zip code, census tract, and parish boundary information was obtained from data files issued with TransCAD. Uninhabited areas in this case consisted of wetlands, state parks, and rivers. These were subtracted from the zip code areas covering the study area, and the resulting areas were then subdivided by superimposing the highway layer on area layer on the reduced zip code areas and manually subdividing them into smaller subareas. The resulting subareas are shown in Figure 3. A total of 52 inhabited subareas were identified in the study area in this manner.

It can be seen from Figure 3 that areas to the west of Lake Pontchartrain, which are wetlands, do not feature as inhabited subareas. Similarly, portions of the north shore and low-lying areas surrounding rivers that flow into Lake Pontchartrain from the north limit the extent of the subareas. Gaps between Subareas 27 and 33 and Subareas 22 and 26 are the effect of uninhabited areas surrounding the Tangipahoa and Tchefuncte Rivers, respectively.

The program written by using GISDK in TransCAD to automatically aggregate the subareas into larger hurricane evacuation zones of uniform elevation was run on the subarea layer created for the northern New Orleans metropolitan area. The process was run through 19 iterations to reduce the 52 subareas to 33 hurricane evacuation zones. The initial 52 subareas and the final 33 hurricane evacuation zones are shown in Figure 4. The process was terminated after 19 iterations, achieving a joint average standard deviation of elevations in the merging subareas of 1.15 ft in this application. The process could have been extended to reduce the number of evacuation zones even further by merely continuing the merging process. However, the 19 merges took approximately 7 h on a Pentium 2 machine in this case, because of the large number of elevation points involved and the complicated file-handling process used. It is possible that the procedure could be streamlined to provide more efficient processing.

Comparison of the two diagrams in Figure 4 shows the subareas that have been combined in the automated merging process. Many of the subareas in the southeast portion of the study area have been combined, as well as those along the shoreline of the lake. However, some combinations have occurred inland, as in the case of the original Subareas 28, 42, and 31 (shown in Figure 4*a*) that extend up to the northern border of the study area. Subarea numbers, obviously, cannot be maintained when a merge occurs. Rather than assign new numbers to merged areas only, new numbers are assigned to all areas on each iteration. Thus, in Figure 4*a,* the subareas are numbered from 1 to 52, whereas they are numbered consecutively from 1 to 33 in Figure 4*b,* without correspondence in area numbers between the two diagrams.

USING EVACUATION ZONES

Once hurricane evacuation zones are established, they can be used together with the storm surge estimates to identify the zones at which the storm surge will exceed the natural elevation of the zone and therefore cause flooding. This can be used to identify which zones should be evacuated and which zones should not. Since storm surge is dependent on various characteristics of the storm (i.e., its track, wind speed, size, forward speed, and barometric pressure), as well as the characteristics of the water body and coastline at the point of landfall (e.g.

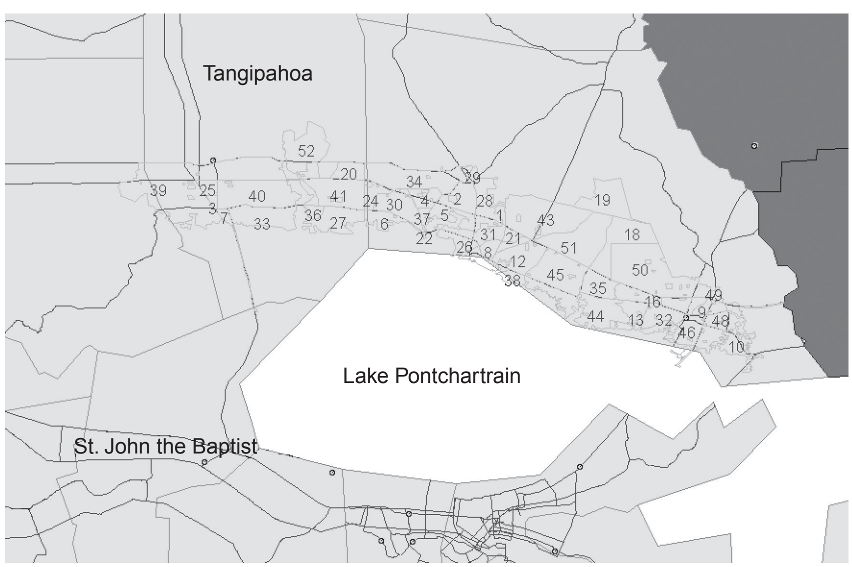
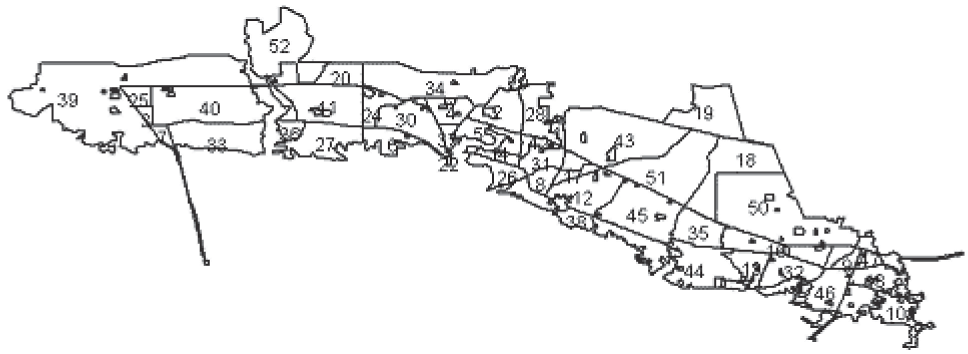
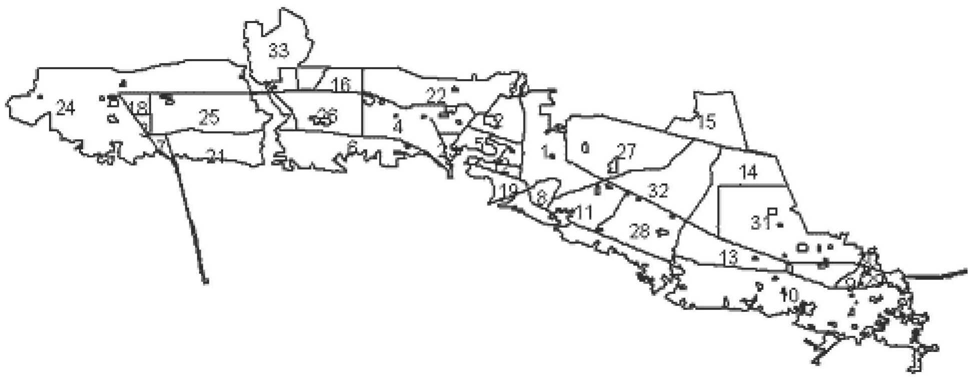


FIGURE 3 Subareas used to establish hurricane evacuation zones.



(a)



(b)

FIGURE 4 (a) Initial subareas and (b) final hurricane evacuation zones.

bathymetry of the coastline, bay and river configurations, and physical restrictions in rivers such as bridges), different flooding patterns emerge from different storms and points of landfall. In the past, the flooding potential of hurricane evacuation zones typically had been associated with the category (wind speed) of a storm only. However, it appears to be much more meaningful to identify the flooding potential of evacuation zones from surge estimates that incorporate many of the characteristics of the storm and its point of landfall. Thus, the use of surge estimates superimposed over a set of evacuation zones is recommended to identify which zones are prone to flooding

To demonstrate the identification of evacuation zones for different storm scenarios, two very different storm scenarios were selected from among the 12 scenarios described earlier. The first was a Category 2 storm moving at 5 mph from the south, and the second was a Category 5 storm moving at 15 mph from the southeast. By using the SLOSH model, storm surge elevations were estimated in the study area. The results are shown in Figures 5 and 6, respectively. The depth of inundation was estimated by subtracting the average ground elevation from the estimated surge height in each zone. The results are shown in Table 1. Ten evacuation zones experienced flooding in Scenario 1, and 29 zones experienced flooding in Scenario 2. In addition, zones on the eastern side of the study area experienced a much higher degree of flooding than those on the west when compared to Scenario 1, because of the difference in direction of the storm. Note that Evacuation Zones 14, 15, 18, 28, and 33 did not experience flooding with either scenario.

Flooded zones can be targeted for evacuation, and those that do not experience flooding can be advised not to evacuate. Thus, only those needing to evacuate will consume valuable road and shelter resources. The potential evacuating population can be estimated with GIS by overlaying census tracts on the zones identified for evacuation and estimating the population from the census figures. Estimates of the population in each zone targeted for evacuation are shown in Table 1. From the table it can be determined that the first scenario affects a total of 63,234 people, whereas the second scenario affects 143,424. Not all those within these zones will necessarily evacuate, but the totals show the potential evacuation population in each case.

SUMMARY AND CONCLUSIONS

The objective for this study was to develop a criteria-based methodology to delineate hurricane evacuation zones. A criteria-based methodology has been developed that focuses on elevation as the primary factor in delineating evacuation zones. However, other factors, such as easy identification of zones, homogeneity of land use, and containment of zones within parishes, are also required. The procedure is initiated by creating an area layer in a GIS based on the MOM for the region in question. This area is then overlaid with the zip code boundaries and land use data to identify uninhabited areas. Only inhabited areas are retained for further analysis, and highways are used to subdivide the remaining portions of zip code areas into

FIGURE 5 Evacuation zones overlaid on storm surge map for Category 2 hurricane from south moving at 5 mph (Stm = Storm; NGVD = national geodetic vertical datum).

subareas. Elevation point file data are overlaid on the subareas to calculate the mean and standard deviation of the elevation of each subarea. Adjacent subareas are merged in an iterative process on the basis of the similarity of the mean and standard deviation of the elevations in each subarea. Both mean and standard deviation were used as criteria for merging because subareas with large variances in elevation do not represent areas of uniform elevation even if their means are similar. The merging process was automated in TransCAD to provide a graphic display of results and to facilitate calculation of the mean and standard deviation of the large number of elevation points involved. In the trial application reported in this paper, the study area encompassed an area of approximately 500 mi^2, resulting in almost 140 million elevation points at 30-m centers in raster format. Computation times of approximately 15 min per

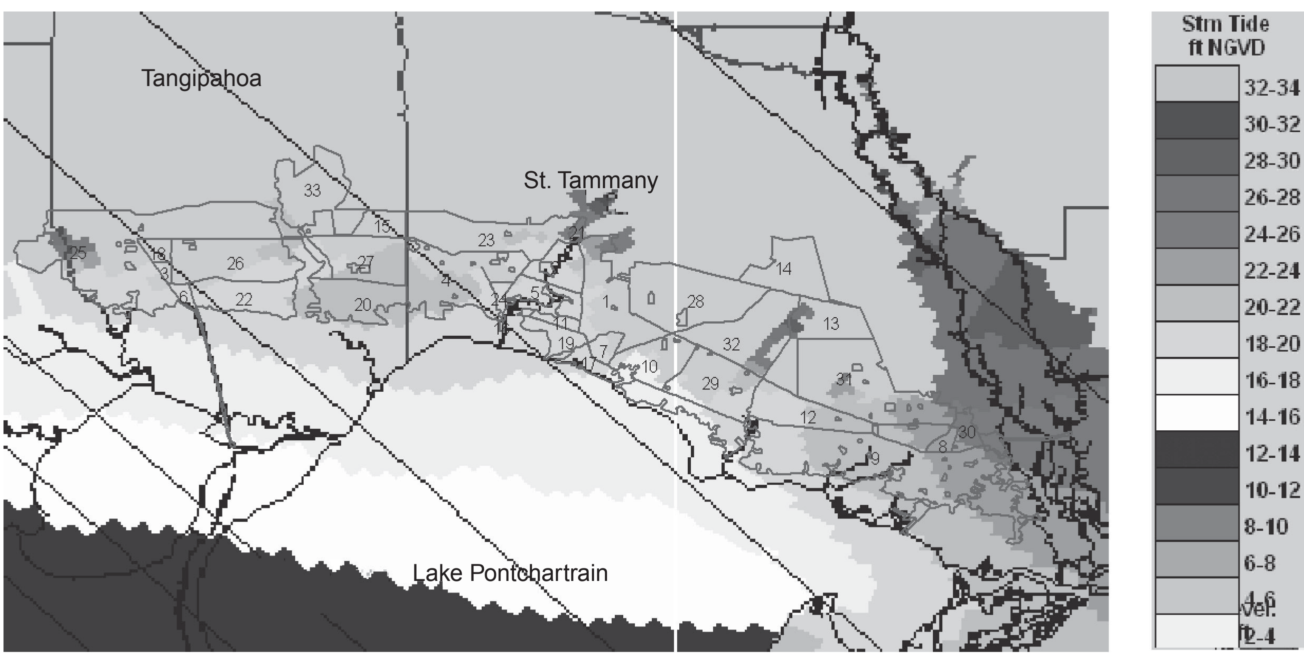


FIGURE 6 Evacuation zones overlaid on storm surge map for Category 5 hurricane from southeast moving at 15 mph.

TABLE 1 Storm Surge Used to Identify Evacuation Zones

		Category 2 Storm, 5 mph, from South		Category 5 Storm, 15 mph, from Southeast		
Zone ID	Average Elevation (X) (ft)	Storm Surge (Y) (ft)	Estimated Inundation (Y-X) (ft)	Storm Surge (Y) (ft)	Estimated Inundation (Y-X) (ft)	Zonal Population
1	6.82	0	0	18–20	13.18	14,200
2	4.31	0	0	18–20	15.69	5,400
3	6.20	0	0	18–20	13.80	520
4	5.29	6	0.71	20	14.71	2,700
5	3.84	4–5	1.16	18–20	14.16	3,400
6	4.09	6–7	2.91	18–20	15.91	121
7	4.76	0	0	18–20	15.24	3,750
8	4.06	0	0	22–24	19.94	5,400
9	2.10	8	5.9	21	18.9	30,800
10	4.81	0	0	16–18	13.19	4,380
11	4.32	0	0	18–20	15.68	2,260
12	4.85	6–7	2.15	20	15.1	10,100
13	9.22	0	0	28	18.78	1,020
16	1.48	5–6	4.52	18–20	18.52	13
17	2.38	5–6	3.62	18–20	17.62	5,040
19	3.27	0	0	18–20	16.73	3,200
20	2.74	6–7	5.26	20–22	19.26	1,170
21	6.41	0	0	24–26	19.59	980
22	3.59	6–7	3.41	18–20	16.41	740
23	7.15	0	0	20	12.85	5,840
24	4.16	5–6	1.84	18–20	15.84	750
25	7.15	0	0	21	13.85	21,000
26	6.55	0	0	20	13.45	5,240
27	5.79	0	0	20–22	16.21	1,300
29	5.93	5–6	0.07	20	14.07	3,875
30	5.20	0	0	24	18.80	3,400
31	6.83	6–7	0.17	22	15.17	4,525
32	8.88	0	0	24	15.12	1,300
33	9.23	0	0	20–22	12.77	1,000

iteration on a Pentium II computer were observed. However, much of this time was needed to tag individual elevation points in the merging process, and it is possible that a more efficient file handling process could be developed.

In the past, hurricane evacuation zones were established manually by using professional judgment. The resulting zones typically have been classified into Categories 1 to 5 to correspond to the category storm that would be needed to flood the zone. However, other factors, such as the track, speed, and size of a storm, are also important in establishing flood levels. Thus, flooding of a particular hurricane evacuation zone is best described as a scenario rather than as a function of the category of a storm alone. Use of a system of zones of homogeneous elevation that are overlaid on a surge map to identify those that will be flooded in each scenario is a more appropriate way to identify which evacuation zones need to be evacuated.

The methodology developed is generic and can be applied in any location to establish hurricane evacuation zones. As constructed, flooding potential is used as the sole criterion to describe zones of homogeneous risk. However, it is conceivable that additional hazards, such as wind, could warrant evacuation of those living in housing vulnerable to severe winds. In this case, housing of similar vulnerability (e.g., mobile homes, or houses built under less stringent building codes) could be merged into a separate set of evacuation zones. Evacuation orders would be made to appropriate zones depending on whether flooding or wind was sufficiently threatening to warrant evacuation.

The methodology could also, possibly, be modified to establish evacuation zones for other hazards, such as wildfires, chemical spills, or nuclear accidents. In the case of wildfires, areas with different vegetation or housing constructed of different materials may have different inherent risk to wildfires and, therefore, be the basis for establishing wildfire evacuation zones. However, the difference in risk is not likely to be very large, bringing into question the establishment of evacuation zones based on risk. In the case of chemical spills or nuclear accidents, scenarios would describe the intensity of the event and to the extent that a hazard could be transported by wind, wind speed, and wind direction. Risk would be described as the vulnerability of residents to the hazard in question.

The process described in this paper formalizes the process of establishing hurricane evacuation zones and introduces a measure of objectivity into the process that had not been present. However, subjectivity should still be applied to ensure that the results being obtained from this relatively mechanical process are reasonable.

ACKNOWLEDGMENTS

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Simulation-Based Emergency Evacuation System for Ocean City, Maryland, During Hurricanes

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Alvin Marquess, and Michael Zezeski

This paper presents a simulation-based system for Ocean City, Maryland, evacuation during hurricanes. The proposed model features integration of optimization and simulation that allows potential users to revise the optimized plan for both planning and real-time operations. Since it is difficult to capture all network operational constraints and driver responses fully with mathematical formulations, six evacuation plans for Ocean City were investigated. Each was optimized initially with the optimization module and then revised on the basis of the results of simulation evaluation. To address potential incidents during the evacuation, the study presents a real-time operation plan with a developed system that allows the responsible operators to concurrently evaluate all candidate responsive strategies and to track the performance over time of the implemented strategy.

Ocean City, a narrow peninsula on Maryland's Eastern Shore, is about 0.3 mi in wide and 9 mi long. The population varies significantly between the summer and the winter seasons. During the summer peak season, the population in Ocean City varies between 150,000 and 300,000 people. Its population may drop to between 7,000 and 25,000 people during the off-peak season (*1*). The summer population in Ocean City is distributed as follows: about 31% between the southern end and 40th Street, about 23% between 40th Street and 94th Street, and about 46% between 94th Street and the state line between Maryland and Delaware.

Figure 1 shows the surrounding area of Ocean City and the highway networks. As described in a previous Ocean City evacuation plan (*2*), the areas about 10 mi away from the ocean are viewed as safe zones during a hurricane evacuation. Thus, such safe areas lie roughly in the region west of US-113. Salisbury, Maryland, the largest city in Maryland's Eastern Shore and about 30 mi from Ocean City, is designated as the major evacuation destination because evacuees will be temporarily relocated to those shelters around Salisbury, if needed. The scope of this evacuation study covers the entire area of about 45 mi by 15 mi, including all major evacuation routes in both Maryland and Delaware.

US-50 westbound, MD-90 westbound, and DE-1 northbound are three primary evacuation routes for Ocean City during emergencies (see Figure 1). US-50 westbound is a divided highway that starts near the south end of Ocean City, goes through Salisbury, Maryland, and then continues to Washington, D.C., and the Baltimore area. MD-90 westbound, having only one lane, begins from the middle of Ocean City (62nd Street), continues to US-113, a north–south highway that connects Maryland's Eastern Shore with Virginia and Delaware, and then merges into US-50 westbound. MD-528 goes through Ocean City, covering its south end, and continues to the state line between Maryland and Delaware. MD-528 is renamed DE-1 after entering Delaware, and it then splits immediately to DE-54 westbound and DE-1 northbound. DE-54 westbound goes to US-113, which carries traffic to different evacuation destinations—Dover, Delaware, and South Salisbury, Maryland. Because of the expected congestion and flooding during hurricanes, coastal highway DE-1 will not be used as an evacuation route during hurricane evacuation for Ocean City. The Salisbury Bypass (US-13), a two-lane highway in suburban Salisbury, is on the city boundaries. Evacuees who arrive at Salisbury Bypass will be regarded as having a safe arrival to the hurricane evacuation destination.

The most recent study for Ocean City evacuation is the revision plan developed by the Maryland State Highway Administration (MDSHA) in the 1980s and updated in 1993 (*2*). This revised traffic control plan for Ocean City during hurricane evacuation was based on the available capacity of critical paths and focused on how to set up traffic control points as well as the clarification of responsibilities among agencies during evacuation operations. Some critical issues, such as how to maximize the network throughput and identify bottlenecks during the evacuation, have not been sufficiently addressed.

The entire evacuation plan should include developments of both candidate strategies and their real-time operational plans during the period of evacuation. These two critical tasks, however, are complicated by the lack of actual demand in Ocean City and the difficulty in predicting travelers' responses during the emergency evacuation. A well-designed plan may have to be changed substantially if some unexpected incidents occur at major evacuation routes. Hence, in response to potentially encountered uncertainties during evacuation operations, it is essential that the responsible agencies have an effective tool with which to efficiently evaluate all candidate evacuation plans and assess the impact of implemented strategies in real-time operations. Such a tool should offer the following functions:

- Assess the potential effectiveness of candidate evacuation plans under various demands and actual roadway geometry constraints;
- Provide the flexibility for planners and operation managers to identify potential bottlenecks during evacuation and to evaluate the effectiveness of various control strategies, such as reverse lane operations and the conversion of shoulders to travel lanes;

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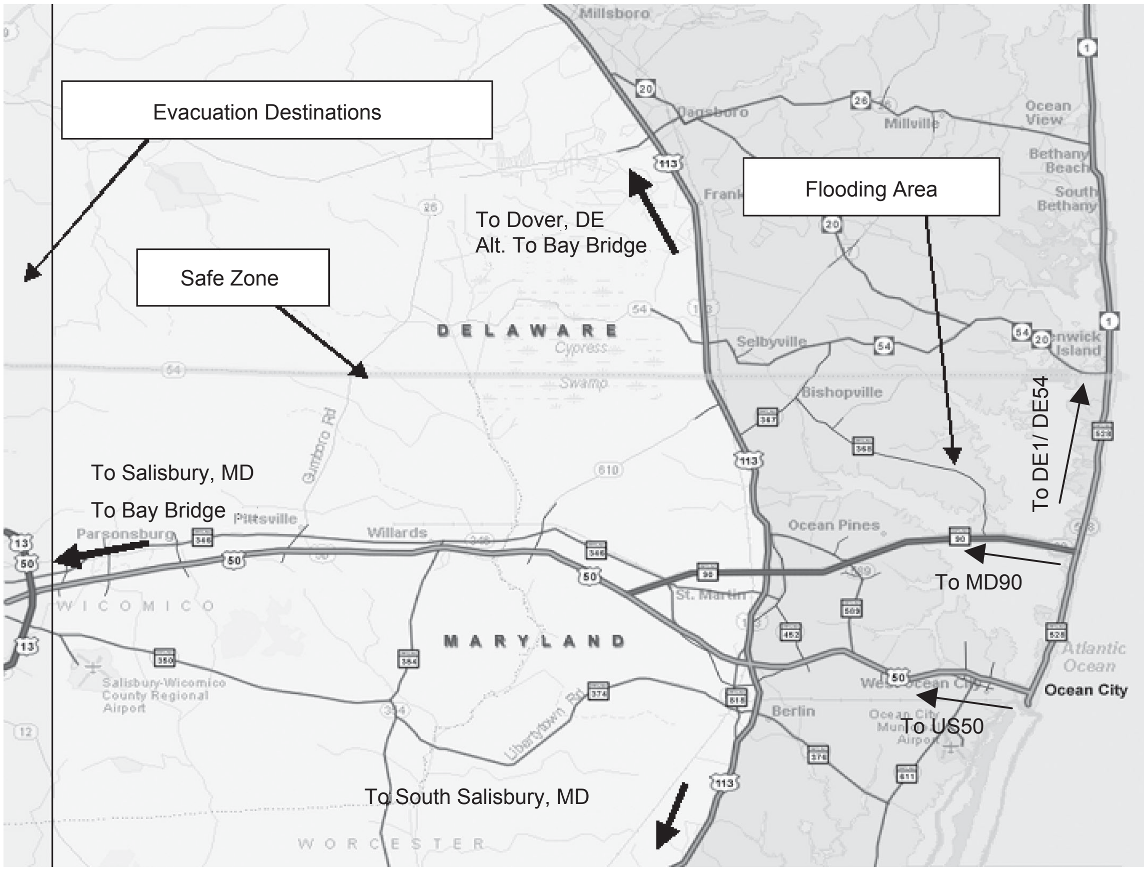


FIGURE 1 Evacuation network and zones.

• Enable the system operators to project traffic conditions on evacuation routes during real-time operations if network traffic sensors have been deployed;
• Efficiently assess and revise any implemented plan during incidents; and
• Offer a real-time evacuation function for system operators to revise an implemented plan when encountering incidents.

From a review of the literature, it is clear that most studies on emergency evacuations can be divided into two categories. Most studies in the first category employ statistical methods or macroscopic or mesoscopic simulation methods to analyze the traffic conditions and generate optimal route choice fractions under the expected demand level (*3–10*). Because such models are mainly for planning applications, they do not take into account the impact of operational constraints on the actual evacuation network, such as insufficient length of acceleration lanes for merging operations and inadequate turning bay length that may cause significant spillback during the evacuation operation.

The other main category of studies on the emergency evacuations is use of microscopic simulations (*11–14*) that estimate the evolution of traffic during the entire evacuation under the expected demand pattern. The extensive simulation output offers its users an effective way to evaluate the performance of candidate plans and to identify potential bottlenecks. The research presented in this paper was developed along the lines of this category, but the proposed simulation-based evacuation model was developed in response to the needs for both planning and real-time operations. It has been integrated with an optimization module (*15*), which may not take into account all operational detail in the network, to produce the preliminary optimal plan. The produced optimal plan can then be refined with the embedded simulation module and be used for real-time applications.

To address potential incidents during the evacuation, this study presents a real-time operation plan with a developed system that allows responsible operators to concurrently evaluate all candidate responsive strategies and track over time the performance of the implemented strategy.

The developed system has a customized interface for both data input and output analyses and has an efficient simulation module for evaluating various operational plans. To facilitate the application, the developed system incorporated six evacuation plans proposed in response to various possible levels of demand in Ocean City. Each control plan includes the target route choice fractions for a given demand level, turning proportions at each control junction, and signal timings at each intersection. Potential users can change these control parameters and develop their own plans if the actual demand varies significantly from those employed in the set of six embedded plans.

DESCRIPTION OF PROPOSED SYSTEM FOR EMERGENCY EVACUATION

Figure 2 presents the principal components of the developed simulation-based emergency evaluation system and their interrelations, which include

• Input module for users to design the evacuation plan, input control parameters, and obtain the detector data;

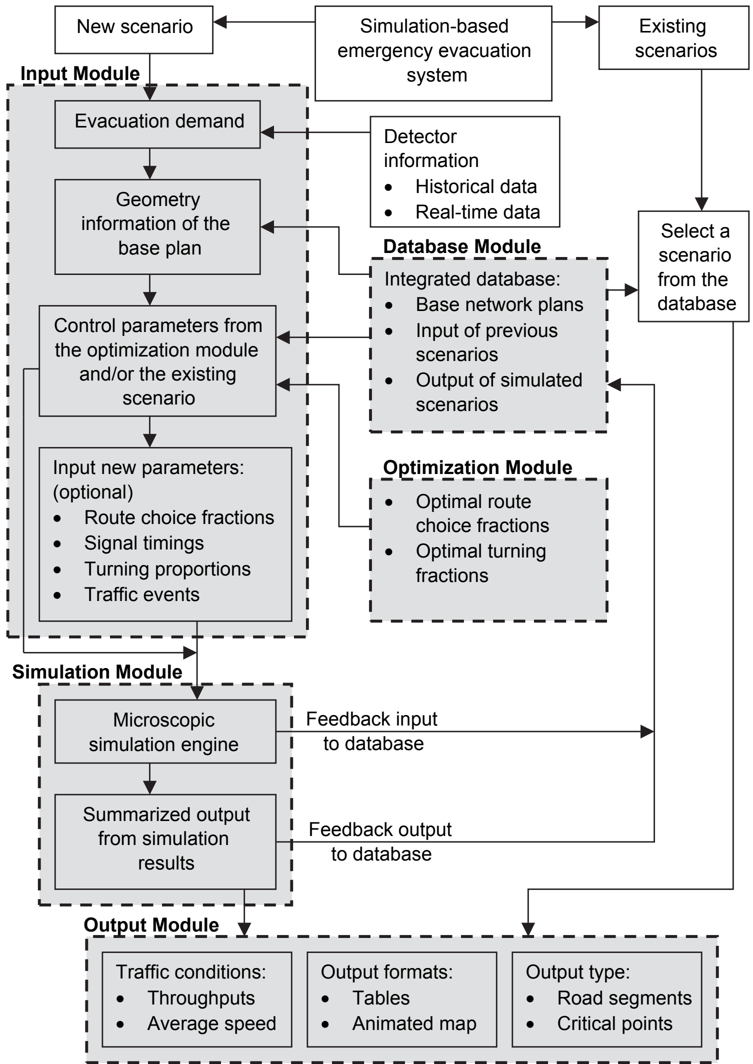


FIGURE 2 Principal modules and their interrelations of proposed simulation-based emergency evacuation system.

- Optimization module (*15*) to generate the optimized route choice and turning fractions for detected demand pattern;
- Simulation module for analysis and projection of traffic conditions during the entire or partial evacuation process under the input scenario;
- Database module for storing newly input scenarios and system outputs and for loading existing scenarios without executing the simulation module; and
- Output module for displaying the customized output from simulation results.

Input Module

The input module is customized for potential users to input the following information during either planning or real-time applications:

- Evacuation duration,
- Distribution of the evacuation demand from both Ocean City and the neighboring regions,
- Selection of the base network from embedded candidate plans,
- Route choice fractions of three primary evacuation routes,
- Turning proportion at each junction,
- Signal timings at each intersection, and
- Location, onset time, and duration of incidents or road closures.

Figure 3 is a snapshot of the input interface, which features its use of a map-based presentation that can guide potential users to operate the system through step-by-step instructions. This design can significantly reduce the learning time and the input error rate of users.

Optimization Module

The optimization module computes the initial optimal demand distribution among available routes and the resulting turning proportions at each intersection under the given network plan (*15*). This module is especially needed during real-time operations, as it can efficiently identify the potentially most effective plan under the detected traffic

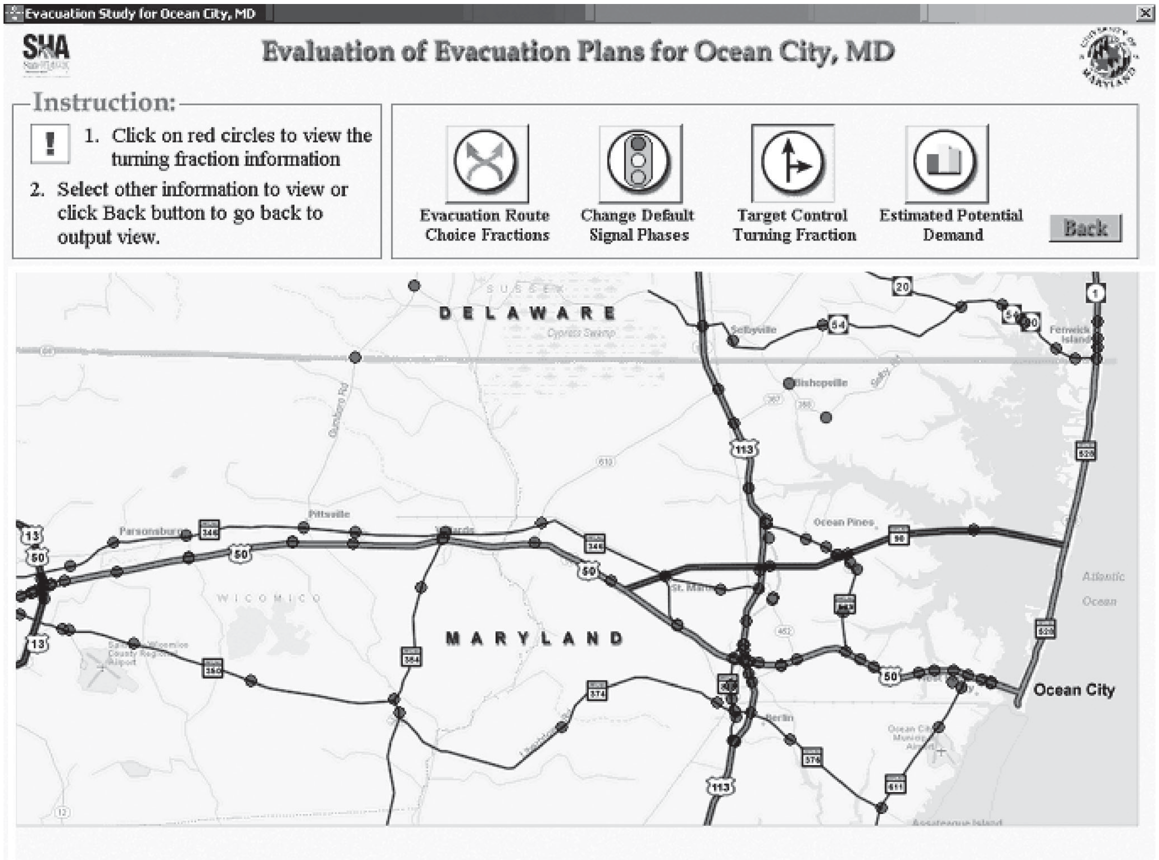


FIGURE 3 Input interface for target control turning fractions.

conditions, which may be different from the planned patterns because of various factors, such as incidents or insufficient guidance.

Simulation Module

The developed simulation-based emergency evaluation system for Ocean City has an embedded microscopic simulation engine for assessing traffic conditions under various demand patterns and the proposed plans. The simulation module developed with CORSIM, a corridor simulation program by FHWA, was customized to fit the evacuation application, which resulted in a substantial reduction of its computing time.

The customized output file is only about 10% of its original size. The computing speed of the developed simulation module under various simulated durations at the target demand level of 6,700 vehicles per hour is shown in Figure 4. It is notable that the simulator needs about 3 min to simulate the entire network traffic condition over a 2-h period of evacuation operations, which is sufficiently fast for real-time operations.

Output Module

The output module is designed to ensure that all simulated traffic conditions from either the networkwide or the individual control perspective can be readily captured by users. It can generate three categories of output data: overall statistics, map-based outputs, and table-based results. With overall statistical results, users can have a clear view of the evacuation state during each hour, including the number of evacuated vehicles, remaining demands in Ocean City, and vehicles that have reached different evacuation destinations. Users can also view the map-based output to see the distributions of throughput and the average speed on each evacuation route.

The primary functions for each type of output are as follows:

- Overall statistical summary shows the numbers of vehicles that have left Ocean City, demands remaining in Ocean City, vehicles arrived at Salisbury, Maryland, and vehicles that have left the study area to Dover, Delaware, or southern Salisbury. The throughputs on those three primary evacuation routes (MD-90, US-50, and DE-54) can also be found in this category of output.
- Map-based output illustrates the distribution of the throughput and the average speed over different evacuation routes with different colors.
- Table-based output highlights detailed traffic conditions, including both the throughput and the average speed over time at critical control points.

Database Module

The database module is designed to store all prior operational experiences, information, and plans, including control strategies on each segment (e.g., reversed MD-90), target volume distribution at key intersections, and potential bottlenecks as well as resulting impacts.

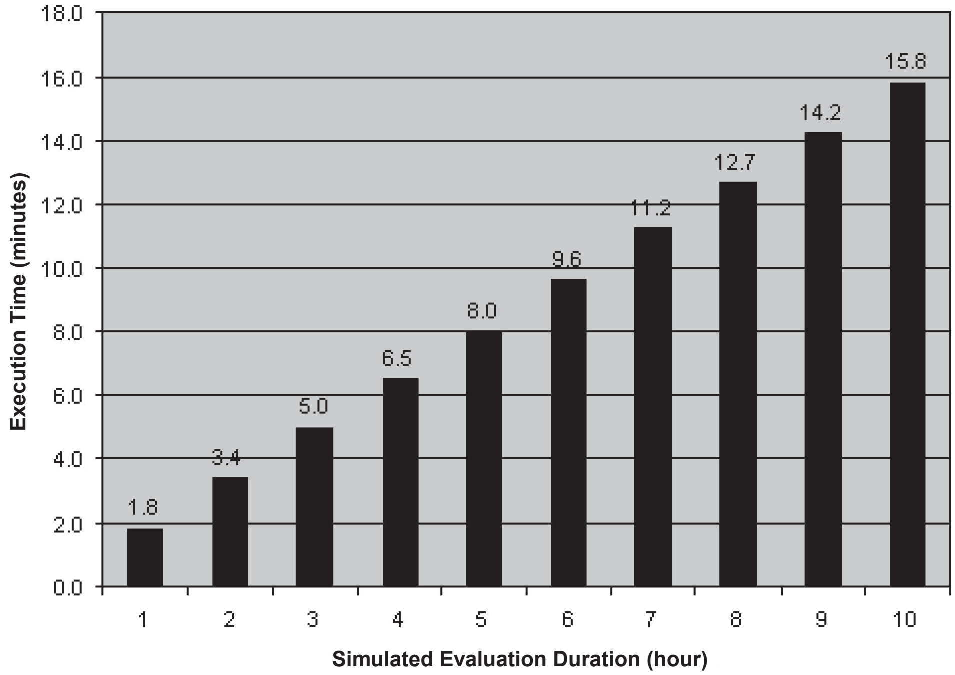


FIGURE 4 Graphical relation between simulation times and required execution times.

All prior experience or plans saved in the database, which has been designed to fit the needs of this study, can also be used in real-time operations. For example, to estimate the impact of one incident occurring on US-50 westbound and the effectiveness of responsive control strategies, responsible staff can load the simulated cases with incidents at nearby locations to approximate the evolution of traffic conditions under the proposed plans. The preliminary estimated results can then be revised after the simulator has completed the execution of the incident scenario with real-time data from detectors.

Operational Flowchart

The flowchart presented in Figure 5 details the operating procedures of the proposed simulation-based evacuation system for Ocean City. The steps are as follows:

Step 1. Input the target evacuation duration and the estimated demand. The system will first ask users to input the expected evacuation duration and the estimated demand distribution. Depending on the available information, users can input the total demand or the distribution of demand over time.

Step 2. Elect a network plan for evacuation operations. This step is designed for system operators to select the candidate network plan for evacuation operations on the basis of the projected demand volume. The current simulator offers six different network plans, each having different levels of reverse lane operations at highway segments and diversion controls at key interchanges and intersections.

Step 3. Optimize control parameters at key control points with the embedded module that includes the target demand distribution between all evacuation routes, turning percentages at each control junction, and signal settings.

Step 4. Execute the proposed plan with the simulation module.

Step 5. Evaluate the effectiveness of the current control plan and identify potential bottlenecks during the evacuation. To ensure the effectiveness of the proposed evacuation plan under the projected demand, one can view the simulated results for time-varying speed, delay, and queue length at key highway segments and control functions through the overall statistics (see Figure 6), map-based outputs (see Figure 7), and table-based displays.

Step 6. Revise the current control plan and resimulate the entire system. Users may choose to investigate traffic conditions at critical control points under various control operations and potential incident scenarios.

Step 7. Repeat Step 5 to identify the optimal control and operational plans under the projected evacuation demand patterns.

Overall, the proposed simulation-based emergency evacuation system for Ocean City offers the capability for potential users to concurrently evaluate the collective impact of various complex demands and operational strategies on the traffic conditions during evacuation. It provides a platform for evacuation planners to develop optimal strategies at both local control points and the entire network level. System operators can also use the system along with deployed sensors to perform real-time evaluation of the evacuation operation.

SYSTEM APPLICATIONS

This section presents the application of the proposed emergency evacuation system for Ocean City during hurricanes, including the development process and the resulting performance under each plan.

The evacuation study for Ocean City starts with two initial network control structures. One is the "do-nothing" plan to the current network, and the other is based on the hurricane evacuation traffic control plan revised in summer 2003 by MDSHA. The microscopic simulation

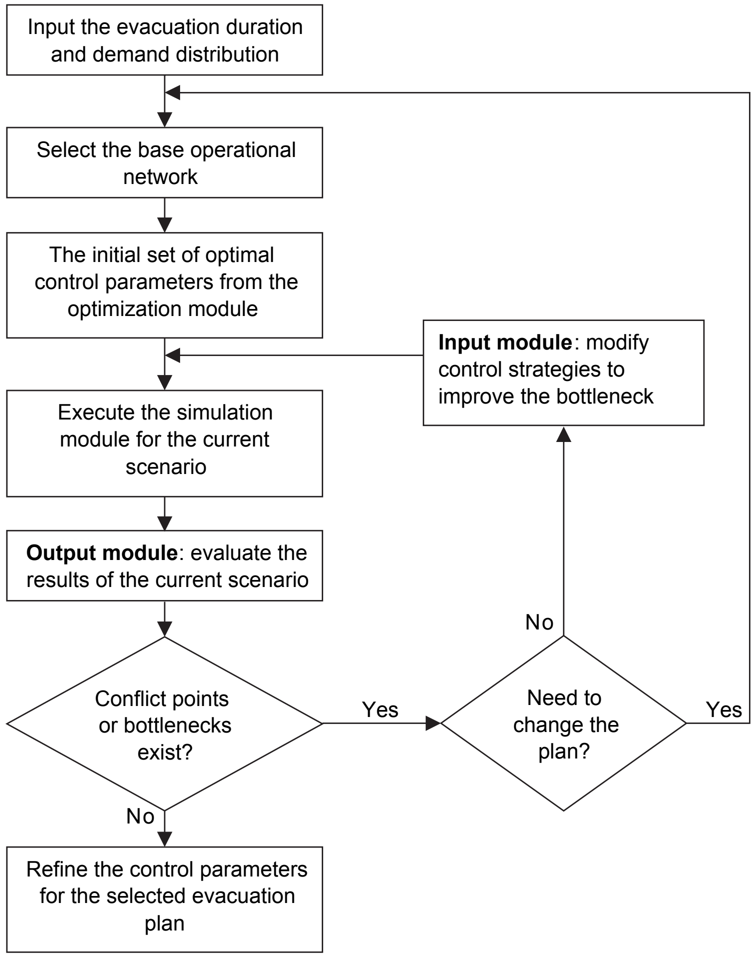


FIGURE 5 Operational flowchart for planning hurricane evacuation.

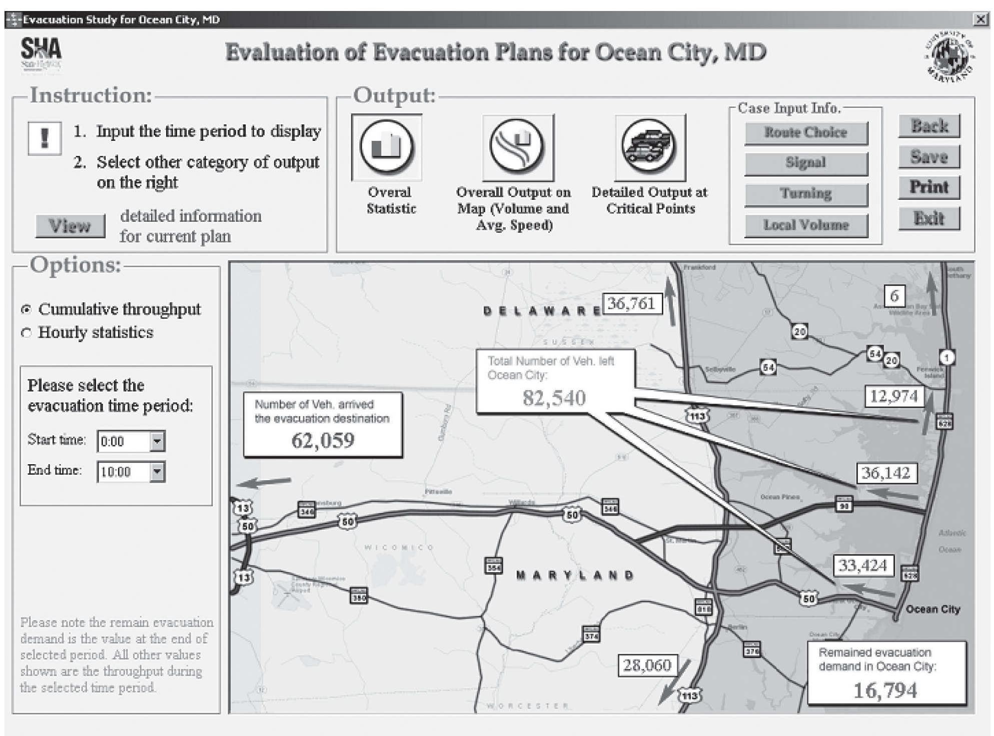


FIGURE 6 Snapshot of overall statistics in output module.

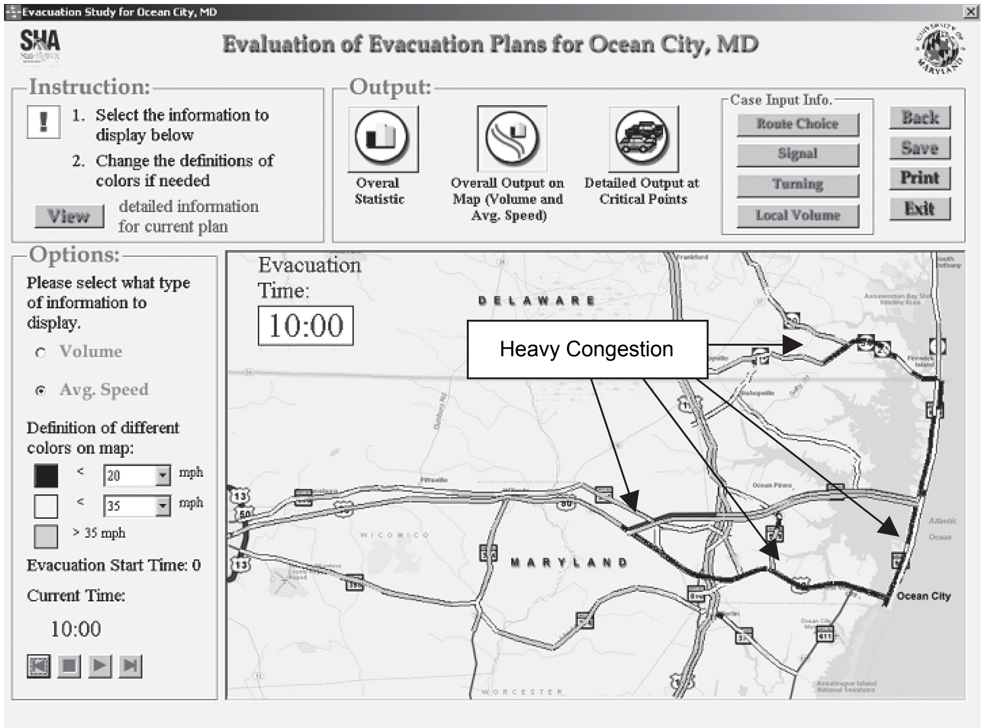


FIGURE 7 Map-based graphical display of heavy congestion on US-50 westbound under Plan 2.

networks for both plans were constructed accordingly. The initial set of control parameters for the do-nothing plan was determined by the optimization module that includes the percentages of route choice at each control point and turning movements at main interchanges or intersections. All control parameters for Plan 2 were based on those provided in the previous research report (*2*).

The simulation results indicate that Plan 2 with control parameters from the 2003 report can successfully evacuate 75,000 vehicles from Ocean City to these safe zones after 10 h of incident-free operations, compared to about 61,000 vehicles under Plan 1. However, the map-based output for Plan 2 showed that heavy congestion may exist on US-50 westbound from the merging point of US-50 and MD-90 back to Ocean City (see Figure 7). A set of new control parameters is then proposed with optimization module for Plan 2, which is to detour some traffic from US-50 westbound to parallel routes and guide some local traffic from West Ocean City to use MD-376. The revised Plan 2 with control parameters from the optimization module can evacuate about 80,000 vehicles within 10 h. However, some bottlenecks can still be identified from the output module. For example, to minimize the interruption caused by the signals on US-50 near Ocean City, all local traffic from West Ocean City shall be detoured to parallel routes so that the throughput on US-50 westbound can be increased more than 14%.

By analyzing the traffic conditions at critical junctions under Plan 2, a potential major bottleneck has been identified at the intersection between MD-528 and MD-90 because the traffic from MD-528 southbound to MD-90 westbound cannot feed MD-90 westbound to saturate traffic condition. That intersection needs to be rechannelized to carry more traffic to MD-90 westbound. Plan 3 is then proposed to convert one through lane to one additional right-turn lane from MD-528 southbound to MD-90 westbound and remove two stop signs on MD-346 and MD-374.

Since all three major detour routes from Ocean City will be saturated under Plan 3 and no other bottleneck exists, reversing one lane on US-50 westbound emerges as the only strategy to improve the evacuation throughput. Plans 4, 5, and 6 are then suggested to reverse different segments of US-50 to increase evacuation throughputs and smooth local traffic conditions. In brief, six network operational plans (including the do-nothing plan and the evacuation plan of summer 2003) for Ocean City hurricane evacuation have been developed and analyzed with the developed simulation-based evaluation system. The geometric features of six hurricane evacuation plans for Ocean City (see Figure 8) are summarized in Table 1.

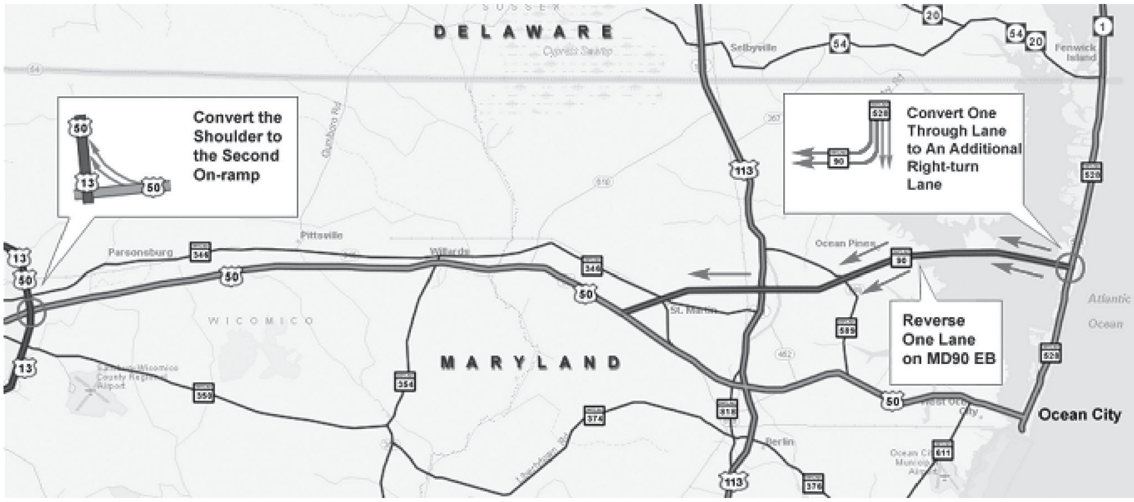
The simulation results of each plan are summarized as follows.

Evacuation Plan 1

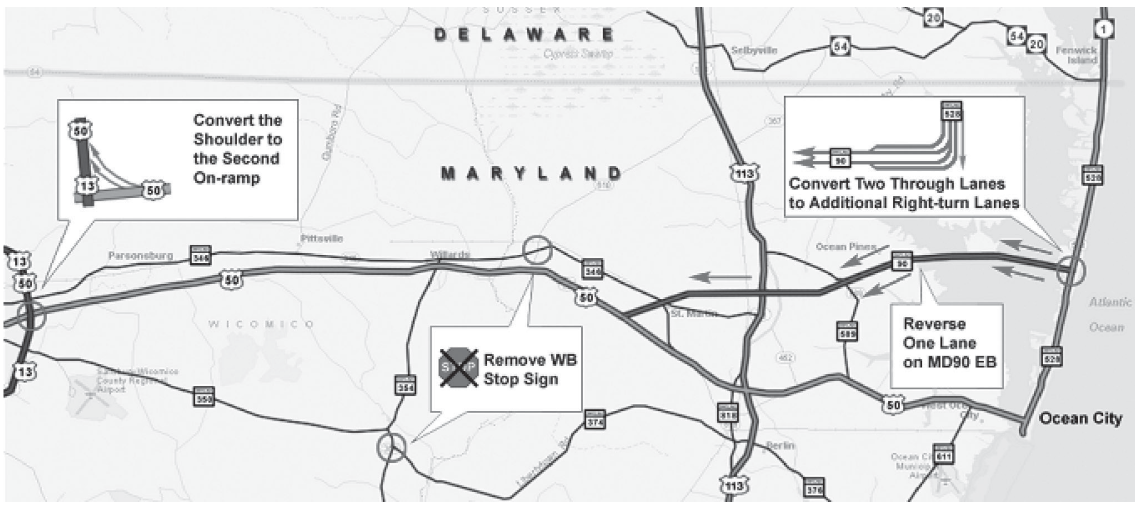
With the current highway network geometry, only a limited capacity is available for evacuees to leave Ocean City. The optimal evacuation plan proposed by the system is to minimize the interruptions on US-50 westbound between MD-528 and US-113. Evacuees from the mainland, including both the flooding area and the safe zone, shall be directed to evacuation destinations via alternative routes other than US-50.

Evacuation Plan 2

Under Evacuation Plan 2, the reverse-lane operation on MD-90 provides an additional outbound capacity from Ocean City. The optimized total throughput on MD-90 under this plan increases up to 1,800 vehicles per hour. The evaluation results from the simulation



(a)



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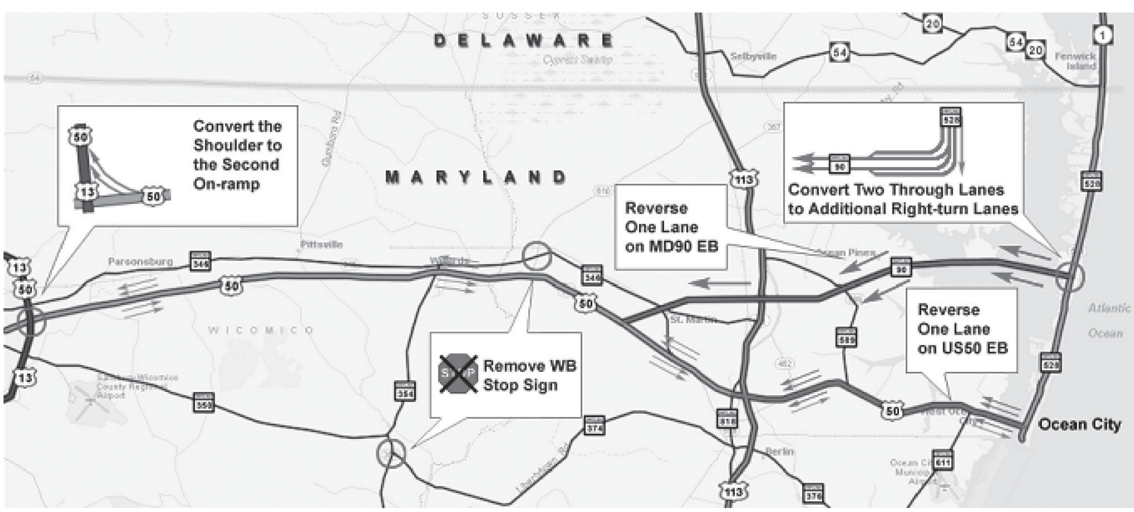

(c)

FIGURE 8 Evacuation plans: (*a*) Plan 2, (*b*) Plan 3, and (*c*) Plan 4.

TABLE 1 Comparison of Operational Features of Proposed Plans

Feature	Plan 1	Plan 2	Plan 3	Plan 4	Plan 5	Plan 6
Reverse MD-90	No	Yes	Yes	Yes	Yes	Yes
Convert the shoulder to one additional on-ramp from US-50 WB to US-13 NB	No	Yes	Yes	Yes	Yes	Yes
Number of through lane(s) converted to right-turn lane from MD-528 SB to MD-90 WB	0	1	2	2	2	2
Remove two stop signs on MD-346 WB and MD-374 WB	No	No	Yes	Yes	Yes	Yes
Reverse one lane on US-50 from Ocean City to MD-818	No	No	No	Yes	Yes	Yes
Reverse US-50 from Walston Switch Rd. to US-13	No	No	No	No	Yes	Yes
Reverse US-50 from Ocean City to Salisbury	No	No	No	No	No	Yes

WB = westbound, NB = northbound, SB = southbound.

suggestion to detour more traffic, especially local demands generated on the mainland, to use alternative routes to avoid the congestion on US-50 westbound.

Evacuation Plan 3

The conversion of one through lane to an additional right-turn lane from MD-528 southbound can increase the throughput on MD-90 westbound from about 3,300 to 3,600 vehicles per hour. Moreover, by removing two stop signs on MD-346 and MD-372 and detouring conflict traffic to use other signalized intersections, the increased capacity on MD-346 and MD-374 westbound provides some additional capacity for assigning more traffic to alternative evacuation routes during traffic events, such as incident and road closure.

Evacuation Plan 4

Evacuation Plan 4 introduces one reversed lane on US-50 westbound between MD-528 and MD-818. With the additional westbound lane, the optimized total throughput on US-50 westbound near Ocean City is about 1,300 more vehicles per hour than in Evacuation Plan 3. However, this plan requires strict turning fraction controls for the local traffic in the suburban area of Salisbury. The variation of turning fractions, for example, from 10% to 15%, on critical points in that area may result in serious backups on US-50 and cause substantial decrease in the total evacuation throughput from Ocean City.

Evacuation Plan 5

To reduce congestion on US-50 caused by local traffic in suburban Salisbury, Evacuation Plan 5 will implement one more segment of one-lane reversed operation than Evacuation Plan 4 on US-50 westbound between Walston Switch Road and Salisbury Bypass. This plan requires less effort for controlling turning fractions in the suburban area of Salisbury. Compared to Evacuation Plan 4, the traffic states on US-50 westbound from the merging point between US-50 and MD-90 to Salisbury Bypass can proceed with fewer potential interruptions.

Evacuation Plan 6

In Evacuation Plan 6, US-50 westbound has one reversed lane from Ocean City all the way to Salisbury Bypass for about 25 mi. This plan does not show any improvement on the total evacuation throughput from Ocean City. However, it is more convenient for evacuees to understand and follow the evacuation directions on US-50 westbound and to reduce the potential of having incidents and other unexpected traffic events.

Comparison

A comparison of overall performance results from these six plans with the optimized control parameters is presented in Table 2. Table 3 presents a comparison of the throughputs on three primary evacuation routes and the number of vehicles that have left Ocean City after 10 h of the evacuation operation. In these two comparisons, a total demand of 100,000 vehicles over the duration of 10 h is assumed. Note that Evacuation Plans 4, 5, and 6 show very similar evacuation throughputs from Ocean City. However, each plan requires a different level of effort and manpower to operate the reverse lane operations.

REAL-TIME OPERATIONS

It should be noted that incidents often incur during emergency evacuation. Thus, a well-planned evacuation plan may need to be revised to accommodate some unexpected events. With properly deployed network sensors, the proposed simulation-based system can serve as an effective tool with which evacuation staff can evaluate responsive

TABLE 2 Comparison Results for Proposed Plans

	Pros	Cons
Plan 1	Easy to implement. Easy for evacuees to follow the evacuation guidance.	Throughputs are the lowest. More efforts on controlling turning percentage are needed to avoid heavy congestion on US-50 WB.
Plan 2	Throughput on MD-90 is higher than Plan 1 due to the reverse-lane operation.	The capacity of MD-90 WB is not fully utilized due to the bottleneck at the intersection between MD-528 and MD-90.
Plan 3	Increases the throughput on MD-90 compared to Plan 2. Less congestion on parallel evacuation routes.	Does not fully utilize all reversible detour routes.
Plan 4	Throughput on US-50 is higher than Plan 3 due to an additional segment of reverse lane operations. Highest level of throughput among all six plans.	Requires turning controls in the suburban area of Salisbury. More efforts are needed for its implementation.
Plan 5	An increase in throughputs over Plan 3. No heavy congestion in the suburban area of Salisbury.	Might cause evacuees to become confused because of two separated segments of reversed lane operation on US-50. More efforts are needed for its implementation.
Plan 6	An extension of the reversed lane operation in Plan 5. Easy for evacuees to follow the guidance.	Much more effort and manpower required by the reverse lane operation on US-50 for about 26 miles. Requires the highest level of efforts to implement.

strategies in real time. Figure 9 illustrates the detailed operational procedures for the real-time evacuation operation. A step-by-step description is as follows:

Step 1. Initiate the system in multiple computers.

Step 2. Acquire historical volume information from the database and obtain up-to-date traffic information from available detector stations.

Step 3. Assign each candidate plan to one computer.

Step 4. Input the information associated with the detected event, including its duration and the reduction in roadway capacity.

Step 5. Execute the simulation module for each candidate plan and then assess its performance from the graphical output.

Step 6. Select the best plan for the current traffic patterns

Step 7. Continue to monitor the evolution of network traffic conditions from detectors, and reexecute the online simulation analysis if needed.

Note that the online simulation function can assist users not only in selecting the best responsive strategies during incidents but also in projecting the network traffic conditions under each implemented plan in real time. For instance, it takes only about 3 min for the simulation-based tool to provide the picture of evacuation traffic conditions during the next 2 h.

Another important function for real-time operations is to compare the detected number of evacuated traffic volume with the target evacuated traffic on each primary route and to revise the evacuation control strategies in real time. Such information can also be provided to those people remaining in Ocean City during the evacuation with guidance from MDSHA and the town of Ocean City.

TABLE 3 Comparison of Throughputs and Number of Vehicles That Left Ocean City in 10 h

Throughput (vehicles)	DE-54 WB	MD-90 WB	US-50 WB	Total
Plan 1	13,000	15,100	33,400	61,500
Plan 2	13,000	33,200	33,400	79,600
Plan 3	13,000	36,100	33,400	82,500
Plan 4	13,000	36,300	46,600	95,900
Plan 5	13,000	36,300	46,600	95,900
Plan 6	13,000	36,300	46,600	95,900

CONCLUSIONS AND FUTURE RESEARCH

This study presented an emergency evacuation system for Ocean City during hurricanes. The proposed system with its customized input–output interfaces and computing module offers an effective tool for responsible staff to perform both planning and real-time simulation of traffic conditions under various control plans. Through proper integration with network traffic sensors, the proposed system can be used to monitor the evolution of traffic conditions during evacuation and efficiently evaluate responsive strategies in contending with unexpected events.

On the basis of preliminary information available in the previous evacuation plan for Ocean City, this study demonstrated the effectiveness of the proposed system with six new plans, each intended for a different level of demand and operational efforts.

The advance in computing technologies and simulation modeling has offered an effective way for traffic professionals to evaluate various emergency evacuation plans at either the planning or the operational phase. The proposed simulation-based system takes advantage of up-to-date computing hardware and software and allows users to assess networkwide traffic conditions at a sufficiently detailed level that can take into account the impact of any local operational or control strategy (such as converting one through lane to a right-turn lane

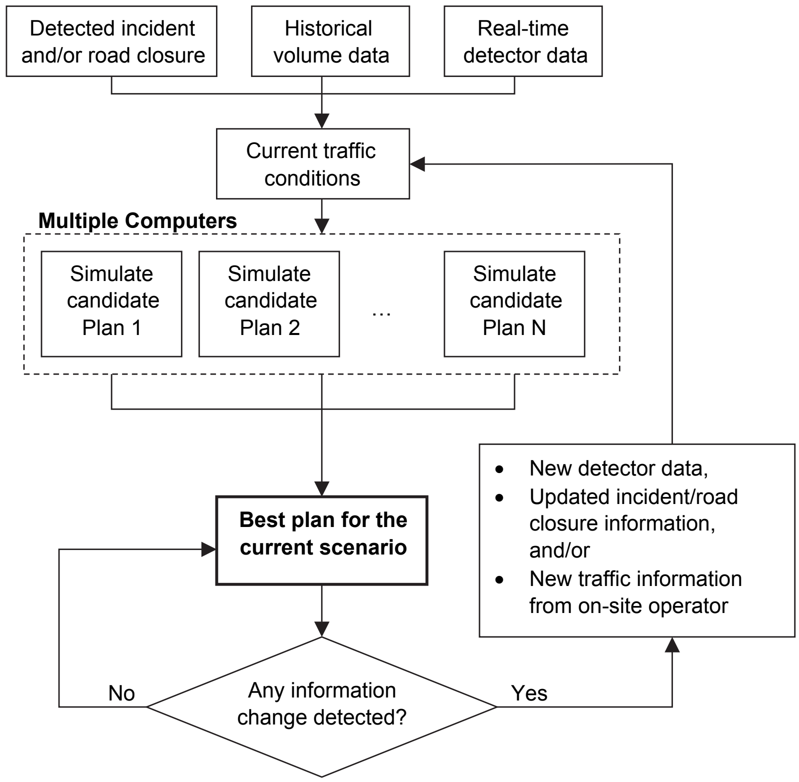

FIGURE 9 Operational procedures for real-time evacuation operation.

at the intersection). The four additional evacuation plans generated from the proposed simulation-based evacuation system demonstrated the potential of using such a system in future emergency evacuation operations.

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Evaluation of Emergency Evacuation Strategies for Downtown Event Traffic Using a Dynamic Network Model

Eil Kwon and Sonia Pitt

This research studied the feasibility of applying a dynamic traffic assignment model, Dynasmart-P, to evaluate the effectiveness of alternative strategies for evacuating the traffic in downtown Minneapolis, Minnesota, under a hypothetical emergency situation that included the evacuation of a sellout crowd in the Metrodome. For this study, the southwest portion of the Twin Cities metro area was selected as the study network, and a set of different network configurations was evaluated for effectiveness in coping with a given emergency situation. The simulation results indicate that managing traffic conditions at the outbound freeway links in the given network during the evacuation period and the access capacity from the downtown area to those outbound freeway links are the critical factors affecting the effectiveness of evacuation operations. For example, the evacuation time under the contraflow operations with the freeways surrounding the downtown area was substantially reduced when the capacities of the key entrance ramps were also increased.

Effective management of traffic operations during and after major emergency events, such as a terrorist attack or high-consequence incidents, is of critical importance in mitigating the impact of a disaster. A key element for an effective emergency operation is the ability to determine the best evacuation strategy that can manage the large-scale movement of vehicles and people for a given situation under time-sensitive and hazardous conditions. Such an evacuation strategy needs to reflect the availability of various resources and the constraints of a given network whose traffic and geometric conditions can change dynamically through time. Although several planning models have been developed to address the needs for evacuation analysis, most of these either adopt approaches that are too simplified for modeling driver behavior or lack the ability to handle a large urban network equipped with different levels of traffic management and information systems. The well-known evacuation traffic models OREMS (*1*) and DYNEV (*2*) adopt the macroscopic simulation approaches developed for the TRAF simulation system. For example, the core simulation module of OREMS is an enhanced version of NETFLOW and FREFLOW, which macroscopically simulate surface street and freeway traffic flows, respectively (*1*). Although OREMS uses an integrated approach of distribution and assignment in determining evacuation routes and estimating traffic performance, the inherent limitations of the macroscopic approach in modeling the route choice behavior of the drivers responding to different levels of traffic information limit the effectiveness of such models. Recently, researchers applied a microscopic traffic simulation model to evaluate the performance of certain route management strategies for hurricane evacuation (*3, 4*). However, the complexity of the conventional car following models and the lack of route choice capabilities under various traffic and information conditions make the application of such a microscopic simulation model for a large-scale urban network extremely difficult.

To address these issues, this research applies a new-generation traffic network planning model, Dynasmart-P, for developing and evaluating emergency evacuation strategies for downtown traffic in a large urban network. This model, developed under FHWA sponsorship, adopts a dynamic traffic assignment approach with the mesoscopic modeling of vehicle behavior and can determine time-variant link traffic conditions by reflecting the effects of various types of travel information and guidance strategies on drivers' route choice behavior in a large network (*5*). A detailed description of the Dynasmart-P model, Version 1.0, used in this study can be found elsewhere (*5*).

STUDY NETWORK, HYPOTHETICAL EMERGENCY SITUATION, AND MODEL CALIBRATION

The emergency scenario hypothesized in this research involves the evacuation of a crowd attending a weekday evening event in the Metrodome, located in downtown Minneapolis, such as a football or baseball game commonly held in fall and spring every year. The evacuation demand from the Metrodome in a sellout event was estimated as 25,000 vehicle trips, which is based on the seating capacity of the Metrodome (*6*). Further, it was assumed that the Metrodome evacuation demand was evenly distributed to the downtown zones, since the parking of those Metrodome attendees is generally distributed over the parking lots in the entire downtown area. In the current version of Dynasmart-P, the vehicles generated from each zone are directly loaded onto the roadways, designated as the origin links for a given zone, following user-specified loading percentages. Therefore, although it is possible to reflect the impact of certain parking lots on the distribution of traffic within a zone, capacity reduction caused by traffic conflict at the exit areas of parking lots cannot be explicitly modeled with the current version of Dynasmart-P.

The hypothetical emergency situation modeled in this study assumes that because of an emergency condition, the evacuation of the entire downtown traffic including the Metrodome crowd starts at 5:00 p.m. on a normal weekday afternoon. Further, to model and evaluate the

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FIGURE 1 Study network coded into Dynasmart-P.

alternative evacuation scenarios involving the downtown area, the southwest portion of the Twin Cities metro area was selected as the study network, whose geometry and trip demand data on a normal weekday were provided by the Metro Council. The traffic demand data include the 15-min trip demand for each origin–destination zone pair from 2:00 until 10:00 p.m. The original network geometry data, for example, coordinates of nodes and link information stored in the ArcView format, were converted for Dynasmart by using special software developed in this study. Figure 1 shows the study network, coded into the Dynasmart format, which consists of 387 zones, 2,488 nodes, and 5,565 links. Because of the limitations of the current version of Dynasmart-P in modeling signal control strategies, the intersections in the study network were assumed to be operated in the actuated mode, and the default timing plans in Dynasmart were used in this study. The detailed description of the input data preparation process for the study network can be found elsewhere (*7*).

By using the data collected for the study network, a qualitative calibration was performed in this study by comparing the simulation results of the current network with a set of the field detector measurements at the selected freeway links. On the basis of the comparison results, a set of the simulation parameters, such as link capacities and the overall scale factor of the demand data, were adjusted to minimize the differences between the estimated flow patterns and the observed traffic flows at the key locations in the network. The execution time of Dynasmart-P for the base network on a personal computer, equipped with a 2-GHz Pentium 4 processor and 1-GB RAM, ranged from 50 to 80 min, depending on the amount of traffic on the network. Figures 2 and 3 show the comparison results after the calibration, which took approximately 20 runs with different combinations of the simulation parameters, at some locations between the Dynasmart output and the field data collected on typical weekdays from three different seasons. Because of limitations on time and resources, an extensive calibration could not be conducted in this study.

As indicated in those figures, the outputs from Dynasmart generally follow the traffic trends observed from those data collection locations, whereas the flow values tend to be underestimated. Note that in this calibration, the demand data provided by the Metro Council for the study network were used without extensive adjustments. In addition, only major arterial streets in the network were modeled with the simplified treatments of the signal control methods for all the intersections. The limited calibration results indicate that the model could be calibrated to the acceptable level by systematically adjusting the origin–destination trip table with more realistic modeling of the signal control strategies.

ALTERNATIVE EVACUATION STRATEGIES

Figure 4 shows the downtown zones, including Zone 130, where the Metrodome is located, that need to be evacuated in this study. As described earlier, the evacuation of the downtown zones is assumed to be started at 5:00 p.m., and the total simulation period for the evacuation analysis is from 4:00 to 7:00 p.m. The sum of the normal outbound trip demand for the 2-h period, that is, from 5:00 to 7:00 p.m., originated from those evacuation zones was estimated as 7,478 vehicle trips as based on the trip demand data provided by the Metro Council. Since this number was considered somewhat lower than the actual value and also because of the lack of the evacuation demand data formally determined for the downtown area, in this simulation analysis two sets of the evacuation demand were used to estimate the evacuation times under two extreme demand conditions:

$$\text{Demand Set 1: } 25{,}000 + 7{,}478 = 32{,}478 \text{ vehicle trips} \quad (\text{Metrodome} + \text{other zones})$$

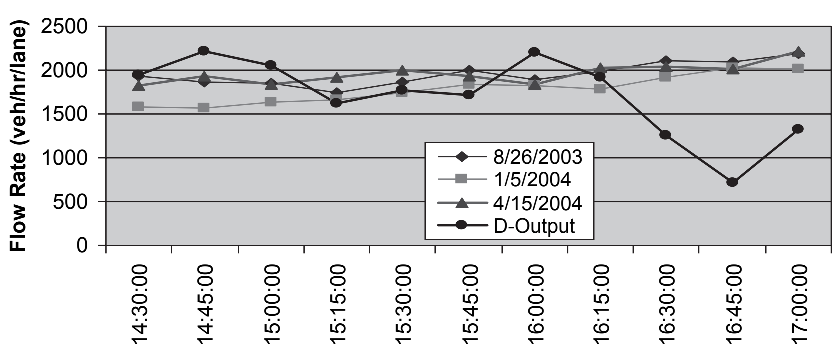

FIGURE 2 Comparison of Dynasmart output and field observations, Station 354.

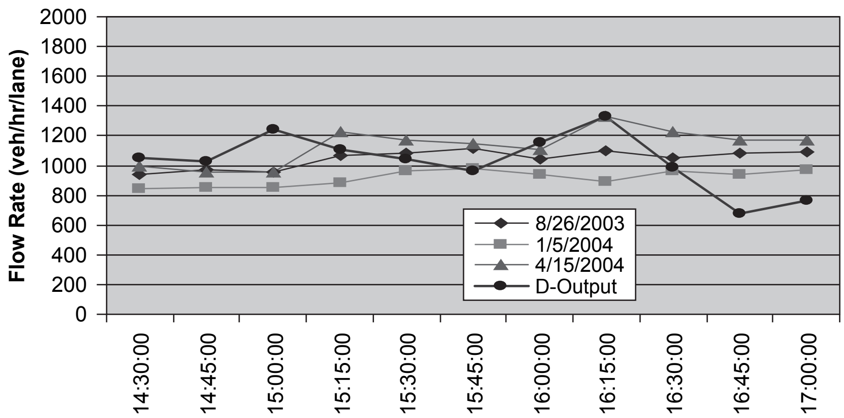

FIGURE 3 Comparison of Dynasmart output and field observations, Station 461.

Demand Set 2: $25{,}000 + 15{,}000 = 40{,}000$ vehicle trips

The destination of this evacuation demand is distributed to different zones in the study network following the 2000 Twin Cities population distribution (*8*) as follows: northwest: 28%; northeast: 17%; southwest 30%; and southeast: 25%

It is further assumed that all the evacuation demand want to evacuate at the same time, at 5:00 p.m. when the evacuation starts. In terms of the network configurations during the evacuation, the following three configurations were modeled and evaluated with Dynasmart:

1. Only the arterial links and freeway exit ramps approaching the downtown area are blocked when evacuation starts (Figure 5).
2. In addition to Configuration 1, the incoming freeway links located inside the network are blocked to prevent vehicles from approaching the evacuation area (Figure 6).
3. In addition to Configuration 1, all the inbound–outbound freeway links in the inside network are converted to one-way outbound links, that is, contraflows, as shown in Figure 7.

Configuration 2 is expected to provide more capacity to the outbound freeway links compared to Configuration 1 by blocking all the inbound freeway links to the downtown area, that is, the outbound freeway links with Configuration 2 would have less traffic coming from other freeways than those with Configuration 1. Note that the changes in the network configuration in the three alternatives become effective when the evacuation starts at 5:00 p.m. during the simulation. Further, since Dynasmart does not provide a built-in function to simulate time-variant contraflows that need to be modeled in Configuration 3, the incident function of the current version was used to emulate the directional changes of the freeway link flows during the simulation. For example, the number of lanes in the outbound links was doubled initially with 50% capacity-reduction incidents until the evacuation started. When the contraflow operation starts, the outbound links start to operate with their full number of lanes, whereas the original inbound links have the incidents that completely block the entire links until the end of the simulation period. The freeway sections used in the contraflow operations have concrete median barriers, and the opposing flows have little impact on the directional capacity on those freeways.

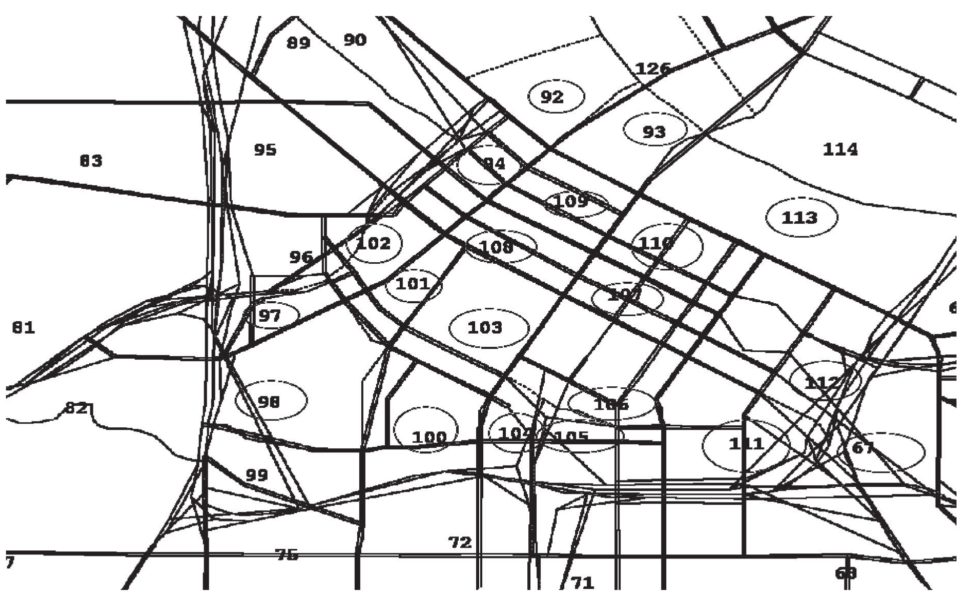

FIGURE 4 Downtown zones (circled) to be evacuated.

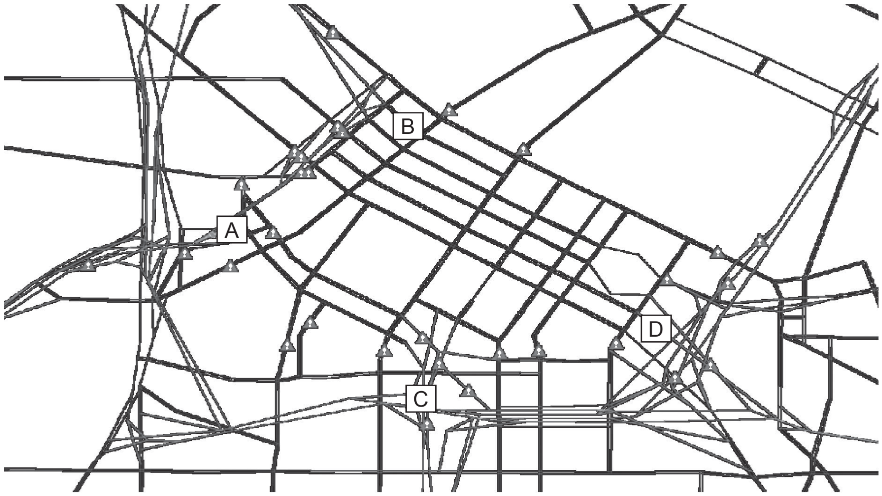

FIGURE 5 Location of arterial links blocked in Configuration 1.

Therefore, in this study, no changes on freeway lane capacity because of the directional changes in traffic flows were assumed.

The preceding treatments of the network changes modeled in this simulation assume that all the network changes can happen in a very short time when the evacuation starts. Further, it is assumed that all drivers in the network are fully aware of the network configuration changes and can adjust their routes accordingly during the simulation. In addition, those downtown zones to be evacuated would not have any incoming demand from the time evacuation starts. Although these assumptions may not be realistic under real emergency situations, the results from this analysis can provide insight into the operational goal for the emergency evacuation.

SIMULATION AND EVALUATION

The alternative evacuation strategies formulated in the previous section were simulated with two different demand sets by using a personal computer that has a 1-GB RAM and a 2-GHz Pentium 4 processor. It took approximately 2 h to complete a run with the cases in Configurations 1 and 2, whereas a Configuration 3 case took 19 h, because to model the contraflow operations on freeways, many more incidents had to be included in Configuration 3 than in Configuration 1 or 2. To estimate the evacuation time from the downtown area, a set of four outbound links located at the boundary of the evacuation area was selected, and the time-variant traffic flow rates from those links were collected from the Dynasmart output files. Therefore, in this study the evacuation time is defined as the flow clearance time at the boundary outbound links. Figure 5 also shows the locations of those four links, A through D, whose traffic performance data were analyzed.

Figures 8 and 9 show the traffic flow rate variations through time at Links A and C with the evacuation demand set Configuration 1, a total of 32,748 vehicle trips. As explained, the duration of the total simulation period is 180 min, and the evacuation starts at the 61st minute in the simulation. The simulation results indicate that the flow clearance time after the evacuation starts ranges from 21 to 27 min at those four links, and it is interesting to see that three alternative network configurations made little difference for evacuating downtown traffic. Although Configurations 2 and 3 are expected to provide more capacity to the outbound freeway links and therefore would produce shorter evacuation time than Configuration 1, because of the

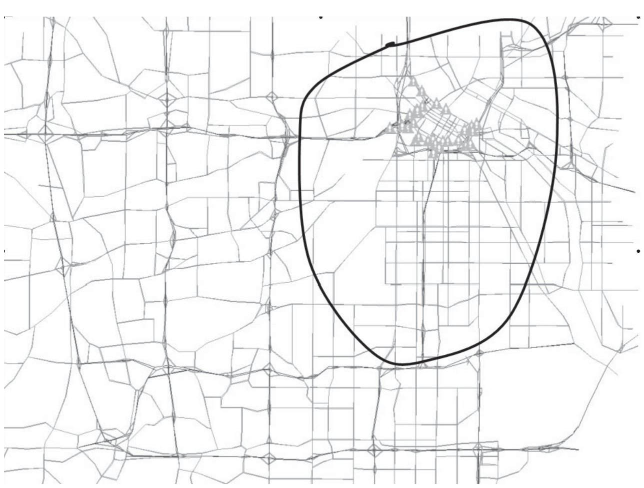

FIGURE 6 Location of freeways with inbound links blocked in Configuration 2.

FIGURE 7 Location of freeways with directions changed during evacuation in Configuration 3.

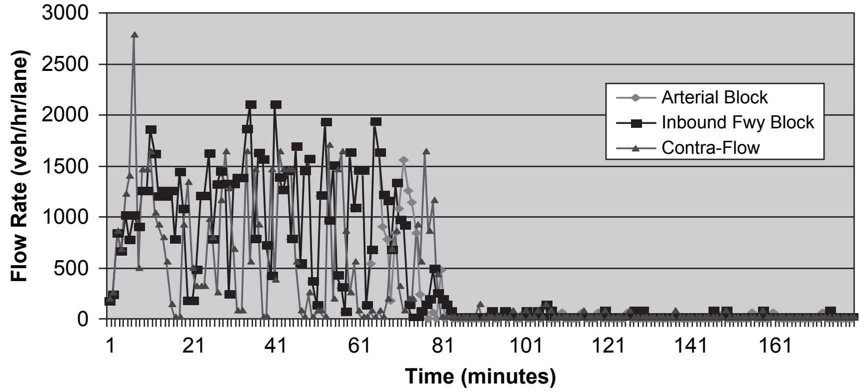

FIGURE 8 Flow variations through time at Link A for different network configurations.

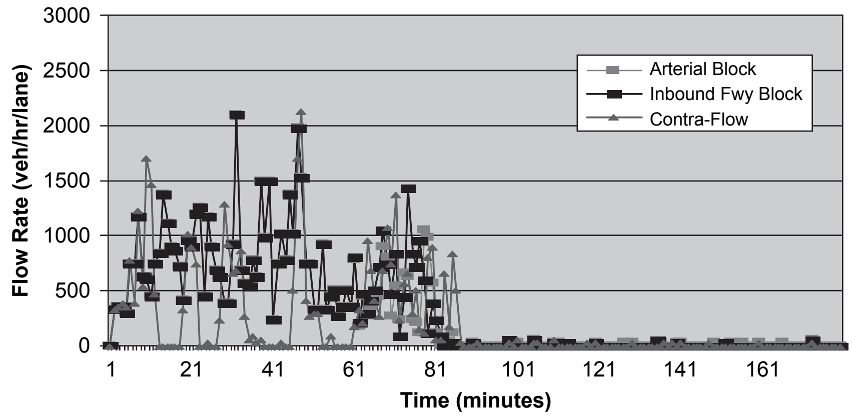

FIGURE 9 Flow variations through time at Link C for different network configurations.

demand data used for this simulation analysis, the traffic at the outbound freeway links when the evacuation started exhibited little congestion. This explains the small difference in evacuation time among those three options. Note that all three alternatives have the same network configurations for the downtown area, for example, the same capacities for all the entrance ramps, and only external freeway configurations were changed.

To evaluate the effects of the ramp capacity increase on the evacuation time, the capacities of two key entrance ramps from the downtown area to outbound freeways were increased. Figure 10 shows the locations of those two entrance ramps, I-394 westbound and I-35W southbound. In the second set of simulations, the number of lanes in these two ramps was increased by one and the demand set Configuration 1 was applied to each network configuration. In this simulation, it was assumed that one regular lane was added at those ramps, that is, not a temporary use of the shoulders that can create traffic conflicts at merge areas. The simulation results with the network Configurations 1 and 2 indicate that although the duration of the evacuation time does not change much, with the increased ramp capacity, the flows at those links during evacuation are significantly lower than the current case without increased ramp capacity. The most significant difference in evacuation time with the ramp capacity increase was observed with Configuration 3, the contraflow operation case for freeways. Figures 11 and 12 include the simulation results for the two ramp links whose capacities were increased. As shown in these figures, the evacuation times were substantially reduced, 5 to 15 min, with the increased ramp capacity when the interring freeways were converted into outbound one-way links.

Finally, the total evacuation demand from the downtown area including the Metrodome was increased from 32,428 to 40,898 vehicle trips, which indicates an almost 100% increase of the non-Metrodome evacuation demand. First, Configuration 1, blocking only the inbound arterial links to downtown, was simulated with the increased evacuation demand. The simulation results indicate that the flow clearance times at those four links are increased from 21 to 27 min to 56 to

FIGURE 10 Location of two entrance ramps with increased number of lanes (A and C).

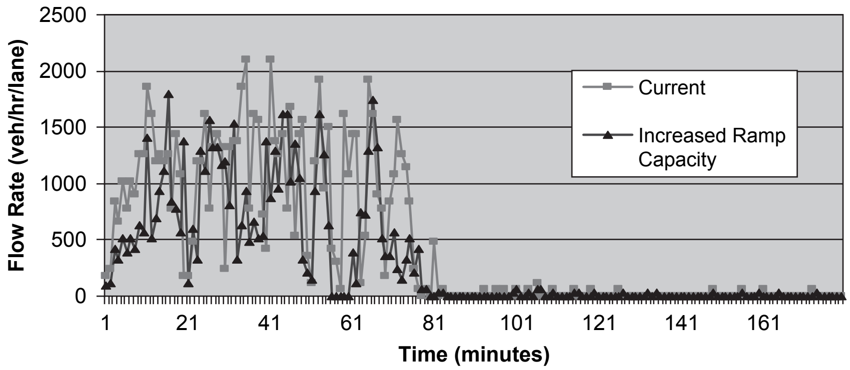

FIGURE 11 Flow variation in Link A with Configuration 1.

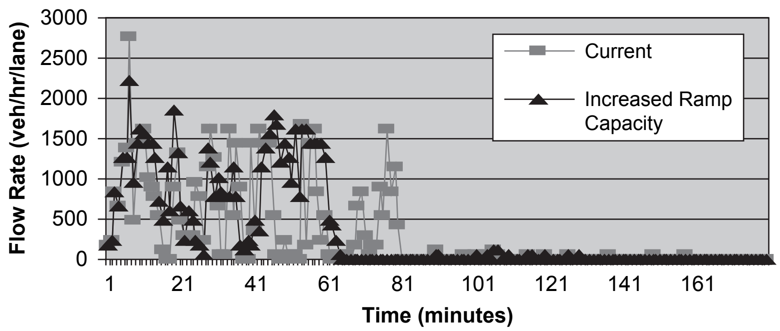

FIGURE 12 Flow variation in Link C with Configuration 3.

75 min. Figure 13 shows the comparison of the flow variations at Link A for two evacuation demand cases. The simulation results with Configuration 2, blocking incoming freeway links in addition to the inbound arterial links, are also similar to the previous case.

The simulation results indicate that when the downtown network configuration is not changed, for example, no entrance ramp capacity increase, the changes in the external freeway configurations do not make significant differences in evacuation time from the downtown area. This suggests that when the traffic at the outbound freeway links is not congested at the time evacuation starts and the incoming demand to the downtown zones can be controlled continuously during the evacuation period, Configuration 1, which blocks only the inbound arterial links to the downtown area, would be as effective as any other alternative tried in this study. However, when the access capacity to the outbound freeway network is increased, it was shown that the contraflow operations with the outbound freeway links would substantially reduce the flow clearance time in the downtown area.

CONCLUSIONS

The application results of the Dynasmart-P model indicate that it is feasible to use a dynamic network assignment model to develop and evaluate evacuation strategies in a large urban network environment.

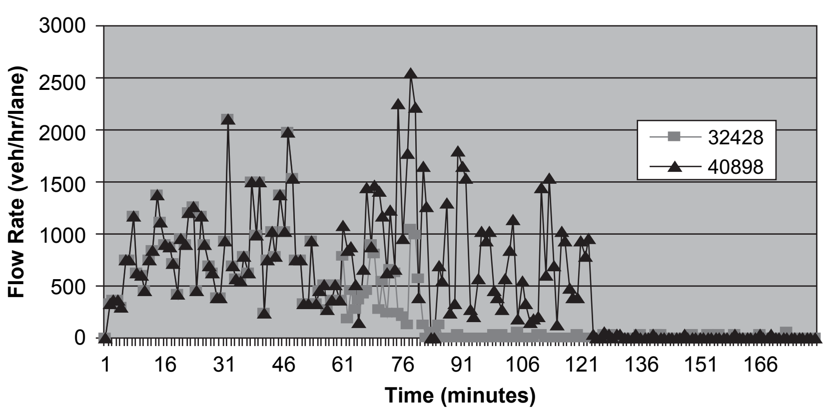

FIGURE 13 Flow variation in Link C with different evacuation demands (Configuration 1).

The qualitative testing results with the study network show the possibility of calibrating the model with the available data for a given network. The example simulations of different network configurations to evacuate the downtown traffic during an emergency situation indicate that the access capacity to the outbound freeway network is the critical issue in reducing the evacuation time in the downtown area. For example, the effectiveness of the contraflow operations with the outbound freeway links showed significant improvements when the capacities of the key entrance ramps in the downtown area were also increased. Because of the limitations in resources and time, a comprehensive analysis was not possible in this study to address various possibilities in terms of network configurations and driver behavior. These include driver familiarity with the network, route choice patterns and traffic behavior under emergency situations with and without real-time information, time delays in configuring network for evacuation, and different signal operational strategies. The estimation of the evacuation demand under dynamically changing environment is another important issue that must be addressed.

ACKNOWLEDGMENTS

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Does Separating Trucks from Other Traffic Improve Overall Safety?

Dominique Lord, Dan Middleton, and Jeffrey Whitacre

Decision makers have long speculated that building separate roads for trucks and passenger cars, or at least separating these into their own lanes, would accomplish two major objectives: (*a*) roadways would be made safer for passenger cars and (*b*) roadways designed specifically for a select class of vehicles rather than for all vehicles might represent overall savings in construction costs. This paper addresses the first objective. Recent studies on the evaluation of safety effects of truck traffic levels on general freeway facilities have not provided a clear understanding of how they affect the number of crashes. In some cases, studies have been contradictory. In addition, no studies have specifically compared passenger car–only with mixed-traffic freeway facilities. The research on which this paper is based aimed to assess whether more homogeneous flows of traffic by vehicle type are safer than the current mixed-flow scenario. An exploratory analysis of crash data was conducted on selected freeway sections of the New Jersey Turnpike for 2002. These sections operate as a dual–dual freeway facility: divided inner and outer lanes. At these locations, the inner lanes have the special characteristic of being for passenger cars only (homogeneous traffic). The selected sections, therefore, offer a very good opportunity to compare the crash experience between passenger car–only and mixed-traffic rural freeway facilities. The results of the study show that outer lanes experience more crashes, both when raw numbers are used and when exposure is included in the analysis. It was shown that truck-related crashes contribute significantly to the total number of crashes on the outer lanes. In fact, trucks are overinvolved in crashes given the exposure on these sections. Although the outcome of this study suggests that separating truck traffic from passenger cars for freeway facilities improves safety, further work is needed to understand the contributing factors leading to truck-related crashes in the outer lanes.

Decision makers have long speculated that building separate roads for trucks and passenger cars, or at least separating these into their own lanes, would accomplish two major objectives: (*a*) roadways would be made safer for passenger cars, and (*b*) roadways designed specifically for a select class of vehicles rather than for all vehicles might represent overall savings in construction costs. This paper addresses the first objective. Given the anticipated growth in truck traffic nationwide because of many factors (e.g., North American Free Trade Agreement), there is an urgent need to evaluate the safety impact of removing trucks from the general flow of traffic. A study of the I-10 corridor published in 2003 and using the Highway Performance Monitoring System and the Freight Analysis Framework found that growth in truck traffic will outpace automobile traffic (*1*). The study concluded that, by 2025, automobile traffic would grow by 62%, whereas truck traffic would grow by 118% along this corridor.

Recent studies that evaluated the safety effects of truck traffic levels on freeway facilities have been sparse (*2–5*). In addition, these studies have not provided clear understanding on how different truck traffic levels affect the number of crashes. Some of the findings have been contradictory. So far, no studies have specifically compared passenger car–only with mixed-traffic freeway facilities. Thus, there is a need to assess whether more homogeneous flows of traffic by vehicle type are safer than the current mixed flow scenario.

To accomplish the objective of the study, an exploratory analysis of crash data was conducted on selected freeway sections of the New Jersey Turnpike for 2002. These sections operate as a dual–dual freeway facility: divided inner and outer lanes. This type of geometry offers more flexibility in closing part of the freeway for maintenance activities or incidents. The turnpike's traffic operations staff can easily shift traffic from one roadway to the other by using changeable message signs. Shifting the traffic need not occur only because of incidents or maintenance; it could be done to balance the flows. Under normal circumstances, the inner lanes have only passenger cars, so the outer lanes serve commercial vehicles (trucks and buses) plus passenger cars. The selected sections, therefore, offer a good opportunity to compare the crash experience between passenger car–only and mixed-traffic rural freeway facilities. Finally, it is important to note that the dual–dual freeway with exclusive passenger car lanes in New Jersey is the only type of facility of its kind in North America.

PREVIOUS WORK

The earliest attempts, in the 1920s and 1930s, to separate cars and trucks were car-only facilities called parkways (*6*). The earliest mention of exclusive truck facilities was in a 1977 ITE paper that described ways to improve efficiency of urban goods movement (*7*). The paper did not document practice because none existed and none were pursued in that era.

In the mid-1980s, the Texas Department of Transportation sponsored research to investigate the feasibility of exclusive truck facilities. In Texas Transportation Institute (TTI) research performed in 1985, Mason et al. described seven types of truck lane configurations (*8*). Construction of all these treatments could occur within an existing right-of-way, especially if sufficient median width remained unused. In 1986, additional research by TTI examined the feasibility of an exclusive truck facility for a 75-mi segment of I-10 between Houston and Beaumont, Texas (*9, 10*). The options considered in the study included the construction of an exclusive truck facility within the

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existing I-10 right-of-way, construction of an exclusive truck facility immediately adjacent to I-10 outside of the existing right-of-way, and construction of an exclusive facility on, or immediately adjacent to, an existing roadway that parallels I-10 (US- 90). The studies concluded that existing and future trends in traffic volumes did not warrant an exclusive facility along the I-10 corridor.

After the passage of the Intermodal Surface Transportation Efficiency Act of 1991, there was more serious consideration of truck-only lanes in the 1990s. Another approach to separating truck and passenger car traffic is the dual–dual roadway, the most notable example of which is the New Jersey Turnpike, the subject of this paper. The initial sections of the turnpike opened to traffic in 1951 (*11*).

A relatively new idea, which TTI is evaluating, is managed lanes. A managed-lane facility is one that increases freeway efficiency by packaging various operational and design actions. The concept promotes adjustment of lane management operations at any time to better match regional goals. Managed lanes also offer peak period free-flow travel to certain user groups. Managed-lane operations for trucks strategies include exclusive-use lanes, separation and bypass lanes, dual-use lanes, and lane restrictions (*12*).

In a current research project (*13*), TTI is again investigating truck roadways. This research is being conducted in conjunction with one of the most revolutionary ideas for transportation in Texas and the largest engineering project ever proposed, the Trans Texas Corridor. This concept will connect Texas and other states with a 4,000-mi network of corridors up to 1,200 ft wide with separate lanes for passenger vehicles (three in each direction) and trucks (two in each direction). The corridor as conceived will include six rail lines (three in each direction—one for high-speed freight and one for conventional commuter and freight trains). There will also be a 200-ft-wide dedicated utility zone. Figure 1 is a conceptual view of the facility.

In a 1990 FHWA study, Janson and Rathi examined the feasibility of designating exclusive lanes for vehicles by type (*14*). This study, which ultimately resulted in a computer program known as Exclusive Vehicle Facilities (EVFS), evaluated exclusive lane-use feasibility by using the following lane-use possibilities: (*a*) mixed vehicle lanes, (*b*) light vehicle lanes (vehicles weighing less than 10,000 pounds), and (*c*) heavy vehicle lanes. Some 10 years later, Battelle Memorial Institute updated the values used in the model and evaluated the program code, determining that its continued use was appropriate (*15*). The program can evaluate the economic feasibility of exclusive lanes for specific sites on high-volume, limited-access highways in both urban and rural areas. The Battelle study resulted in some criteria for providing truck facilities based on annual average daily traffic (AADT), annual average daily truck traffic, level of service, truck-involved crash rates in million vehicle miles traveled, daily traffic delays, and proximity to freight origin–destination points.

The mechanism for financing truck facilities in upcoming years will be an important topic. A recent study for the Reason Policy Institute by Samuel et al. proposed that self-financing toll truckways consisting of one or two lanes in each direction be built in the existing right-of-way (*16*). These truckways would be barrier separated from existing lanes and have their own ramps. The lanes would be designed specifically for trucks, and trucks would have exclusive use of the lanes. Financing for the truckways would be from tolls collected from trucks using the facilities. Trucks using the truckways would be rebated federal and state fuel taxes for the mileage traveled on the truckways. Federal truck size and weight regulations would also be eased for truckway users.

Harwood et al. presented summary results from a survey of states indicating the states that have considered or have implemented measures to separate trucks from passenger vehicles (*17*). Harwood et al. stated that exclusive lanes for trucks and buses have been considered by 17% of highway agencies, exclusive lanes for buses only by 20% of highway agencies, and exclusive roadways for heavy vehicles only by 3% of highway agencies. The authors note that many highway agencies face decisions about whether to reduce traffic congestion by building exclusive truck roadways or exclusive truck lanes. They then make the following statement: "Research is needed to provide safety performance measures to assist highway agencies in such

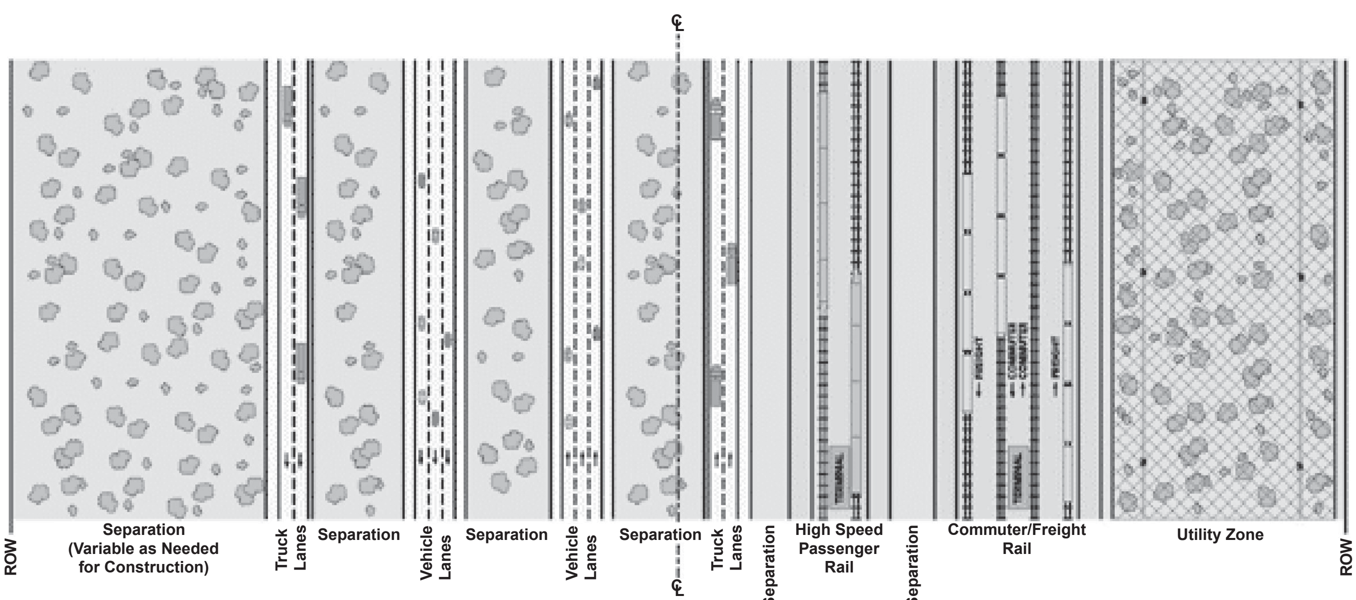

FIGURE 1 Concept plan view of Trans Texas Corridor (ROW = right-of-way). SOURCE: Trans Texas Corridor, 2002, www.dot.state.tx.us/ttc/view1.htm.

decisions" (*17*). Therefore, this paper addresses a critical need in a timely fashion.

As indicated, few studies have examined how the level of truck traffic affects safety on freeway facilities. There exist studies that looked at the safety effects of different truck traffic control strategies (e.g., lane restrictions, exclusive truck lanes) (*18–20*), but few addressed regular mixed-traffic facilities. For instance, Jovanis and Chang studied the safety effects of traffic exposure by vehicle and collision types on Indiana highways (*2*). They found that increased truck traffic is usually associated with an increase in the number of crashes, although the relationship increases at a decreasing rate for all truck-related crashes. On the other hand, Miaou reported that an increase in truck percentages on urban and rural freeways in Utah was associated with a decrease in the number of crashes (*3*). He hypothesized that for a constant vehicle density, as the percentage of trucks increases, the frequency of lane change lessens, hence reducing the number of truck-car collisions. Winslott Hiselius also found that increases in the number of trucks resulted in a decrease in the number of crashes on 83 rural highway sections in Sweden (*4*). She attributed this effect to the lower average vehicle speed in the traffic stream when the proportion of trucks increases. Nonetheless, she indicated that the low sample size may have affected the conclusions of the study. In summary, there is no clear understanding about the effects of how homogeneous and nonhomogeneous traffic flows affect truck-related crashes on freeway segments.

DATA COLLECTION

Two study sections were used in this analysis. These are located on the northern part of the New Jersey Turnpike, near the Garden State Parkway (see Figure 2). The first study section is situated between Interchanges 10 (Milepost 88.1) and 11 (Milepost 90.6) for a total length of 2.5 mi. On this section, both inner and outer segments have three lanes in each direction. The second section is located between Interchanges 11 and 12 (Milepost 95.9) for a total of 5.3 mi. The inner segment contains three lanes per direction, and the outer segment has four lanes per direction. The left lane on the outer segment is used as a high-occupancy-vehicle lane during the morning peak period and no trucks are allowed to use it. Trucks are restricted to the right two lanes in both the four-lane outer roadway and the inner roadway if they happen to be diverted. All sections have 12-ft lanes with a 12-ft paved shoulder on the right side of the traveled way. The posted speed limit is 65 mph for both study sections, but turnpike personnel can reduce the speeds as needed via dynamic speed limit signs.

The study period covered crashes that occurred in 2002. Crash data contained detailed information about the severity, location, crash type, type of vehicle, day of the week, direction of travel, and time of day. The data were initially obtained as a printed computer output and were eventually coded into an electronic database. In 2002, there were 298 crashes, of which 78 involved trucks. The seven crashes that occurred on exit or entrance ramps were removed from the analysis to minimize the influence of these ramps on crashes. Thus, all crashes used in this work occurred on the main traveled ways.

Traffic flows in AADT were obtained from the New Jersey Turnpike Authority. The data were available for each section and were separated by vehicle class and by direction. The data were collected for nine vehicle classes (passenger cars, two-axle trucks, tractor-trailers, two- and three-axle buses, etc.). Only passenger cars (Class 1) are allowed in the inner lanes (again, except for incidents, maintenance, and lane balancing). The split for passenger car traffic between the inner and outer lanes is about 65% and 35%, respectively. Table 1 summarizes the AADT traffic by vehicle class (1 = passenger cars; 2 through 9 = trucks and buses). The table shows that about 30% of the vehicular traffic on outer lanes is heavy vehicles.

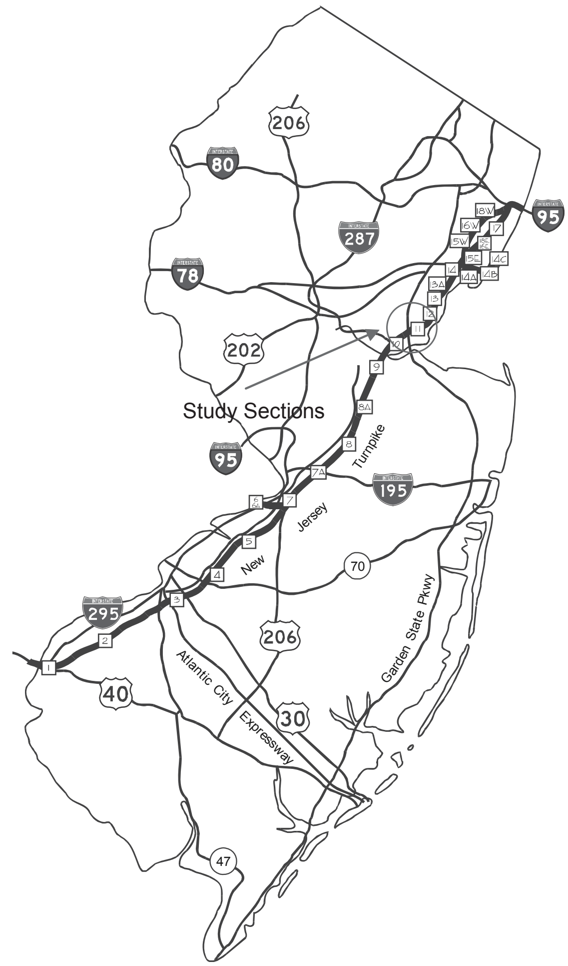

FIGURE 2 Location of study sections on New Jersey Turnpike.

CRASH DATA ANALYSIS

This section describes the characteristics of crashes that occurred between Mileposts 88.1 and 95.9 on the New Jersey Turnpike in 2002.

General Characteristics

A total of 298 crashes occurred on the New Jersey Turnpike in 2002. Table 2 depicts the number of crashes by collision type and whether the crash occurred in the outer or inner lanes. The table shows that sideswipe collisions occurred more frequently than any other type of crash in both the inner and the outer lanes. Table 2 also illustrates that more crashes per mile occurred in the outer lanes than in the inner lanes. Also, total rear-end collisions occurred more frequently in the outer lanes than the inner lanes, which may suggest that traffic flo is subjected to more unstable traffic conditions (or nonhomogeneous

TABLE 1 AADT Traffic by Direction and Type

Interchanges	Inner Lanes Total	Outer Lanes Passenger Cars	Outer Lanes Trucks & Buses	Outer Lanes Total
Southbound				
10 to 11	54,280	29,228	12,953[a]	42,181
11 to 12	64,424	34,690	15,498[b]	50,188
Northbound				
10 to 11	57,587	31,008	14,431[c]	45,439
11 to 12	66,260	35,679	16,810[d]	52,489

Buses = [a]4.7%, [b]7.9%, [c]4.4%, [d]7.5%

TABLE 2 Crashes by Accident Type and Lane Designation

Accident Type	Outer Lanes	Inner Lanes	Total
Run-off-road	11	2	13
Collision with an object	22	31	53
Collision with a guardrail	29	24	53
Rear-end	43	31	74
Sideswipe	50	30	80
Others	20	5	25
Total	175	123	298
Miles	7.8	7.8	7.8
Crashes/mile	22.44	15.77	38.21

flow). Similarly, sideswipe collisions occurred more frequently in outer lanes. Interestingly, collisions with an object happened more frequently in the inner lanes. This finding may suggest that the lower undercarriage clearance of cars is a contributing factor in object collisions. The data show that very few heavy vehicles hit an object on the road.

Figure 3 shows the number of crashes by severity and excludes the cross-median collisions (8) and the uncategorized or unknown crashes (3); these crashes could not be assigned by using the criteria defined in the figure. This figure shows the data by direction of traffic, northbound and southbound, as well as by lane designation, that is, inner and outer lanes. Figure 3 shows that property-damage-only (PDO) crashes account for about 75% of all crashes; there were no fatal crashes in 2002 on these two sections. There were proportionally more

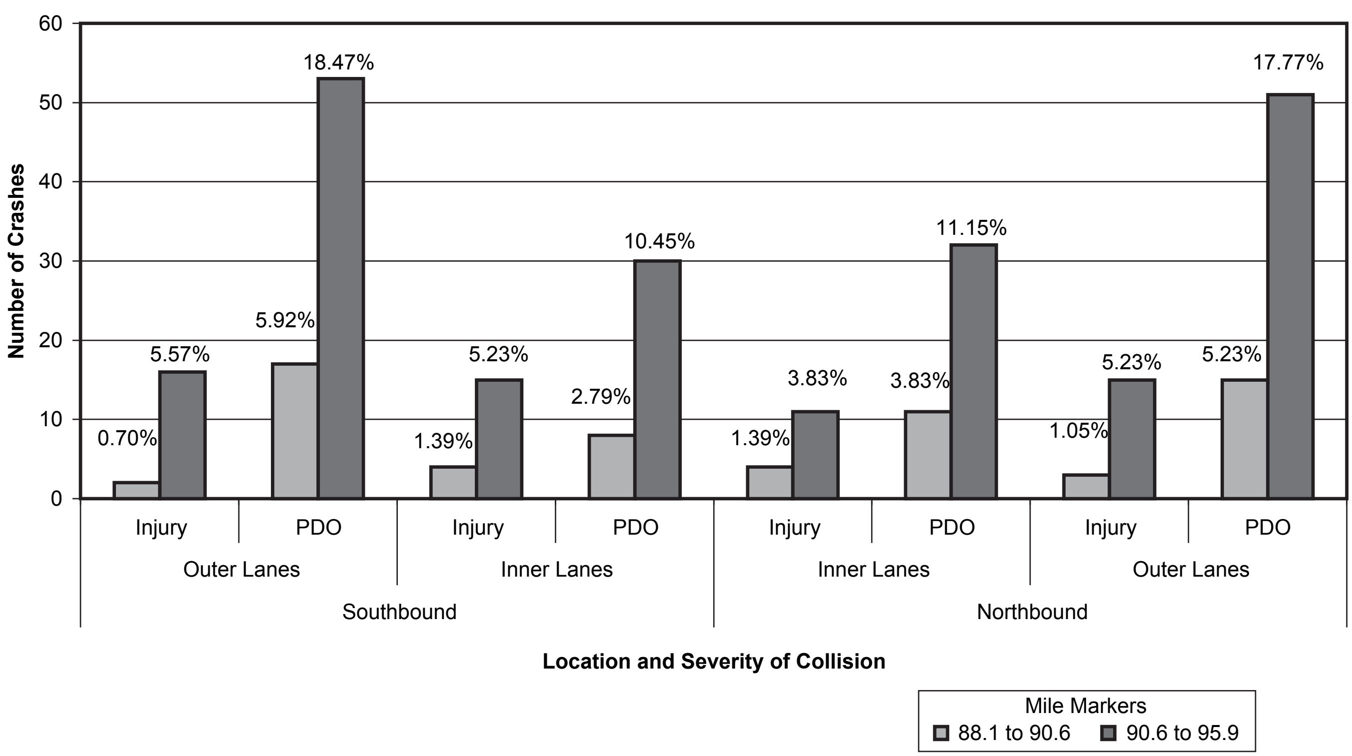

FIGURE 3 Number of crashes by direction, lane designation, and severity.

PDO crashes on outer lanes than on inner lanes. This indicates that the speed of traffic is probably lower on outer lanes than inner lanes. Higher vehicle speed is associated with higher occupant severity (*21*). Finally, the northbound and southbound lanes experienced similar numbers of crashes in both outer lanes.

Figure 4 shows the number of vehicles involved in a crash. The figure reveals that more single-vehicle crashes occurred on inner lanes than outer lanes, with about 30% and 50% of all crashes, respectively.

Figure 5 illustrates the number of crashes by weather conditions. This figure shows that more than 80% of crashes occurred during clear conditions. The outer lanes experienced more crashes than the inner lanes for all types of weather conditions.

Figure 6 shows the number of crashes by day of the week. This figure shows that outer lanes had a higher percentage of crashes occurring during a weekday than inner lanes. On weekends, the inner lanes experienced more crashes than on weekdays. As discussed in the next section, the higher percentage of crashes in the outer lanes on weekdays may be attributed to truck traffic.

Truck-Related Crashes

Figure 7 illustrates the types of crashes for passenger cars and trucks. As this figure indicates, about 45% of all truck-related crashes are categorized as sideswipe collisions. This finding is similar to previous work on this subject (*5*). However, trucks are not overinvolved in rear-end collisions and run-off-the-road crashes, as reported by Golob and Regan (*5*). As indicated earlier, passenger cars collide more frequently with an object on the pavement than do trucks. Finally, passenger cars hit guardrails more frequently than trucks do.

Figure 8 illustrates the severity of the crashes and the lanes where they occurred. A few truck crashes occurred on the inner lanes when the outer lanes were closed. The severity pattern for passenger cars is very similar between inner and outer lanes.

Figure 9 shows the number of crashes by day of the week for trucks and cars respectively, as well as inner and outer lanes. As illustrated in Figure 8 and initially shown in Figure 6, the outer lanes experienced a large number of truck-related crashes. If truck-related crashes were removed from the inner lanes, the outer lanes would experience roughly the same number of crashes during weekdays. Very few truck crashes occurred during the weekend because trucks travel less frequently during this period.

Figure 10 illustrates the severity of the crash by type of vehicles involved in the collision. This figure shows that the proportion of severe-injury car–truck collisions is about the same as that of severe-injury collisions involving two cars. Very few single-truck or truck–truck crashes caused an injury.

Table 3 summarizes the crash rate (in 10^6 vehicle miles) by direction of travel and mile markers. This table shows the rates (all crashes) as a function of the combined passenger car, bus, and truck exposure (all vehicles). It is important to note that the relationship between crashes and exposure usually has been found to be nonlinear (*22, 23*). There were not enough observations, in this study, to properly test this assumption. Thus, a simplification (i.e., use of crash rates) had to be made for this part of the analysis. Table 3 suggests that the crash rate in the outer lanes is almost double that in the inner lanes, given the same exposure. This outcome may indicate that truck traffic had an influence on crashes. Finally, the crash rates for the northbound and southbound traffic provide similar values, as reported earlier.

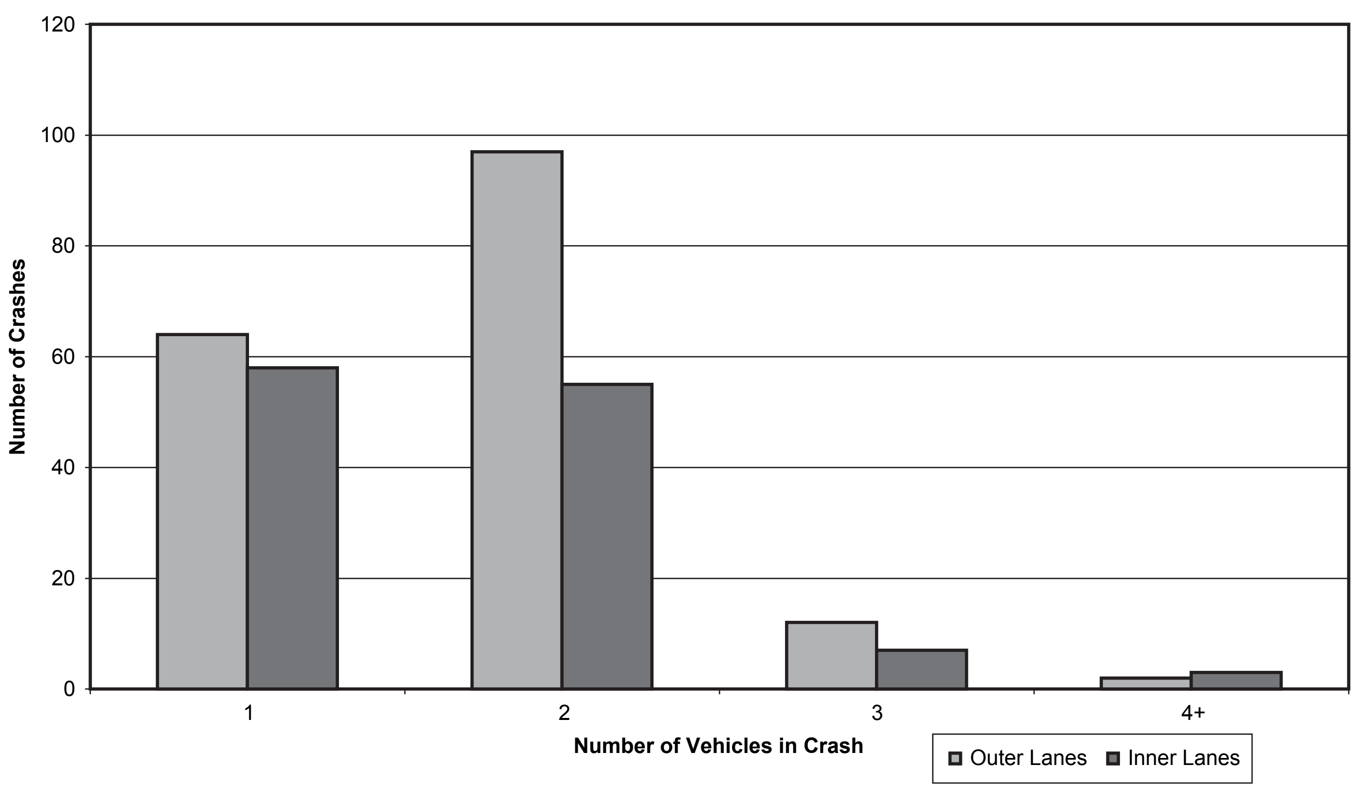

FIGURE 4 Number of vehicles involved in crash.

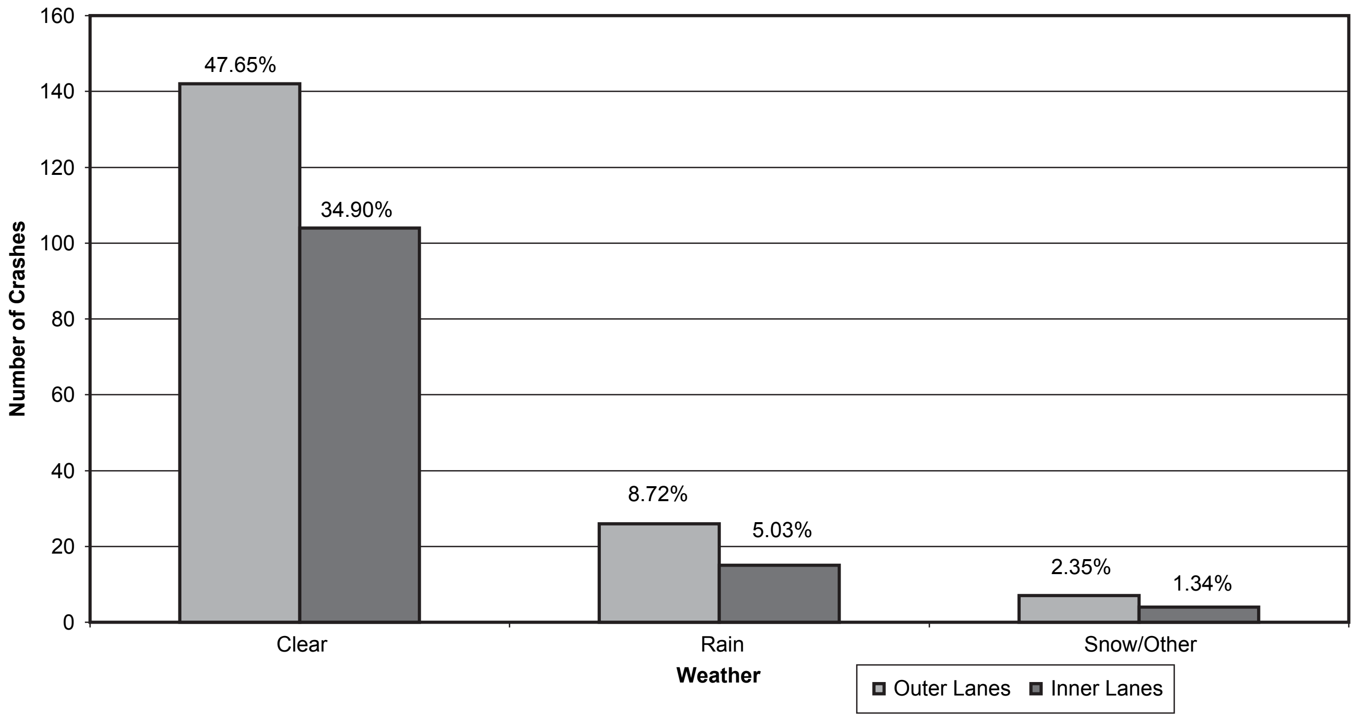

FIGURE 5 Number of crashes by weather conditions.

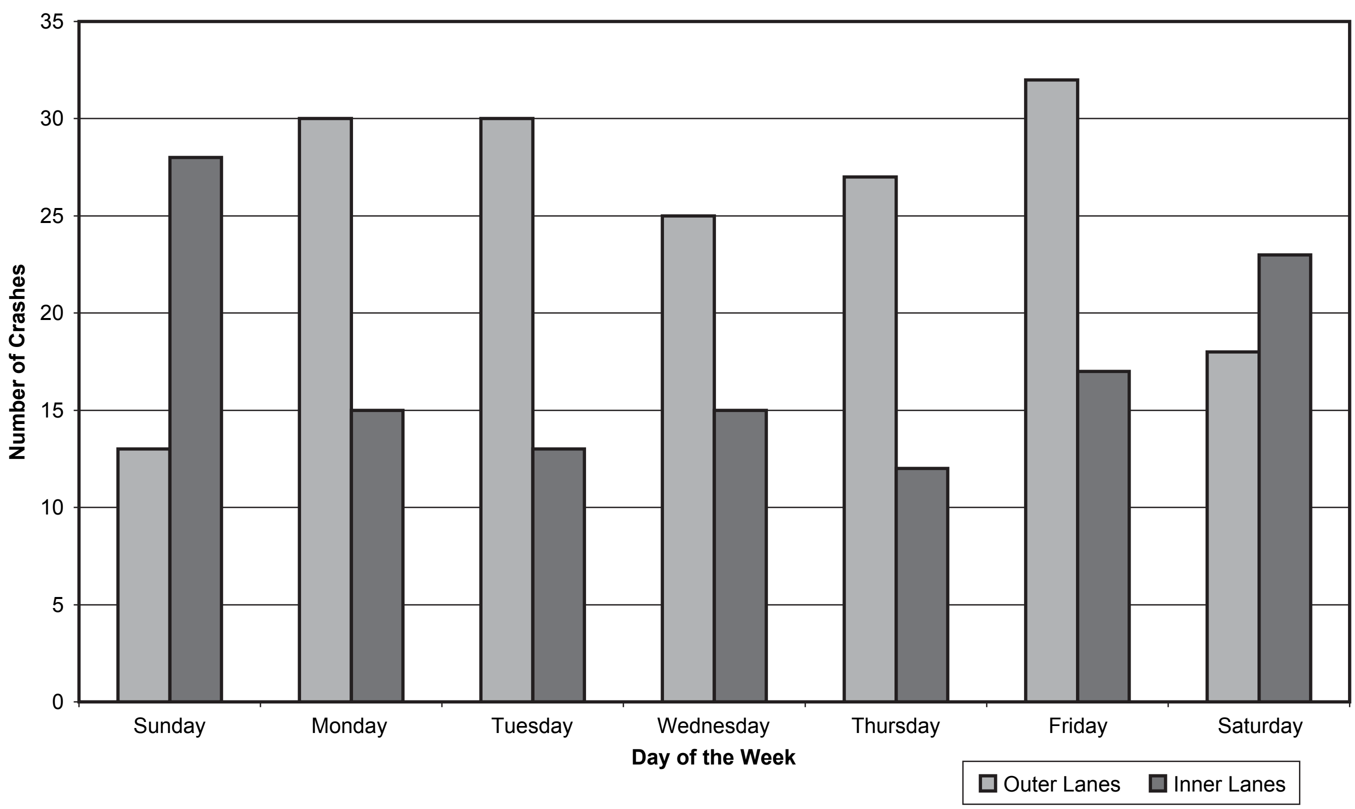

FIGURE 6 Number of crashes by day of week.

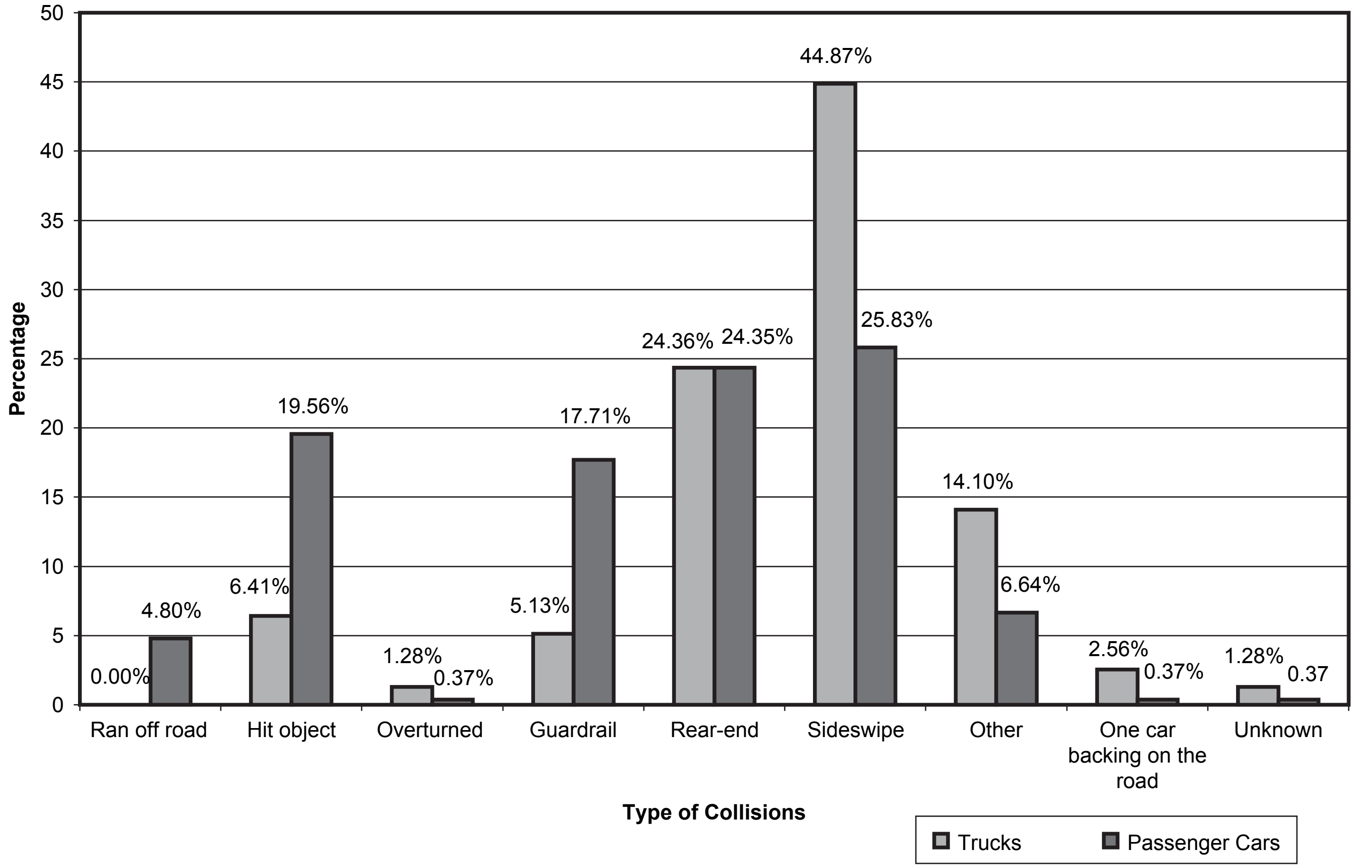

FIGURE 7 Percentage of crashes by collision type for trucks and passenger cars.

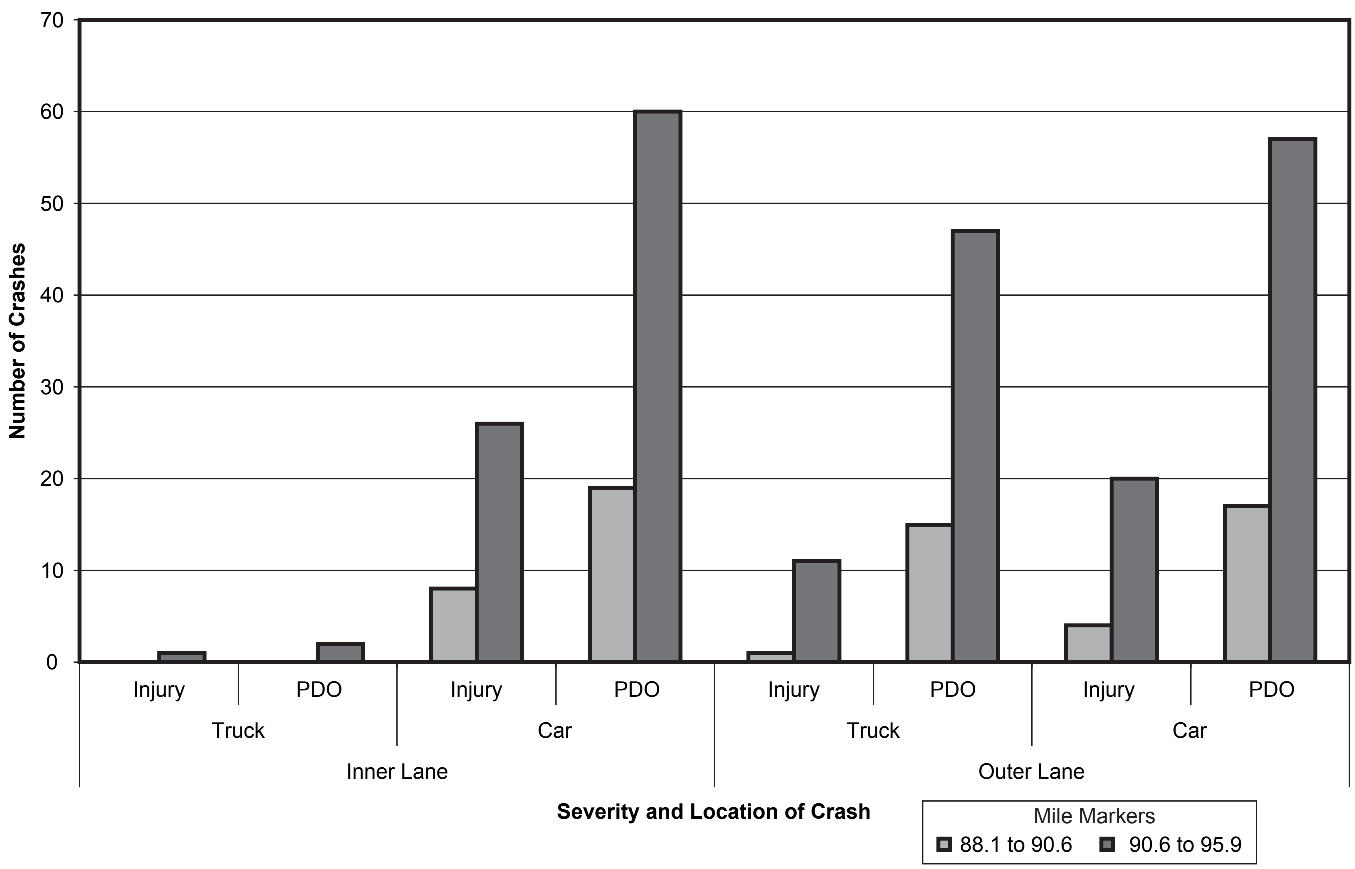

FIGURE 8 Number of crashes by type of vehicle, lane designation, and severity.

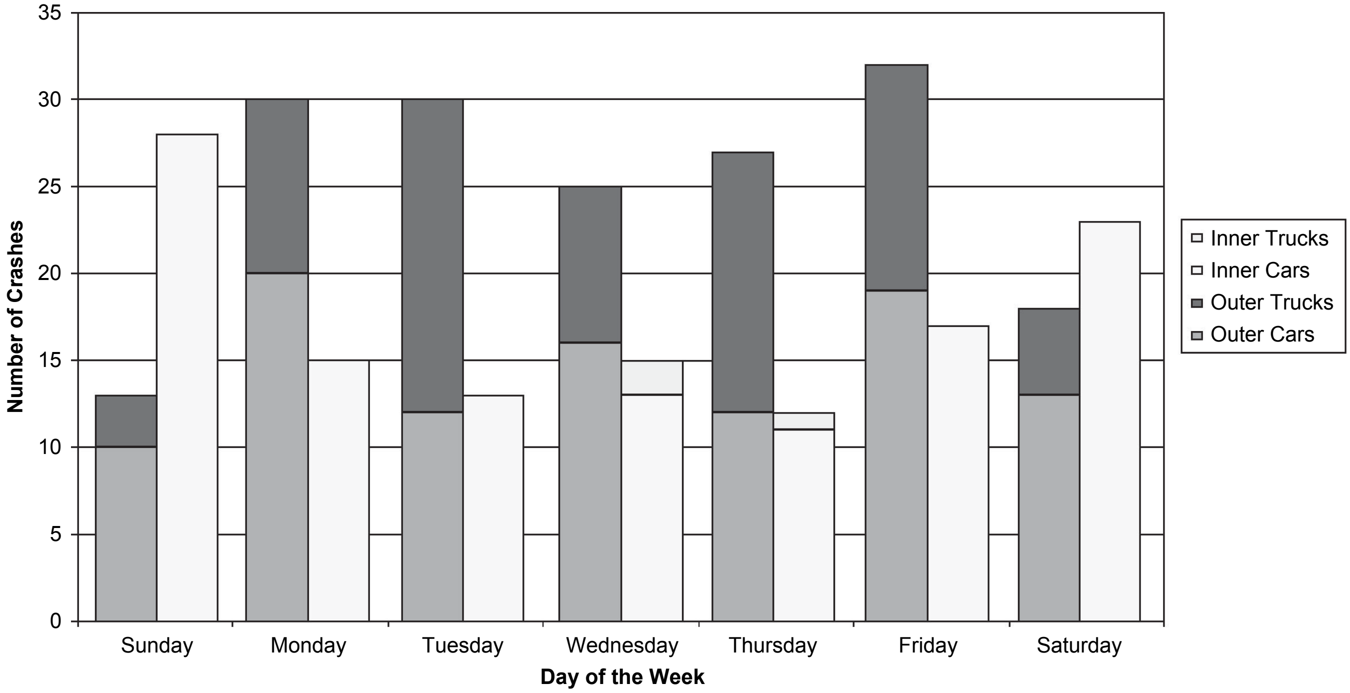

FIGURE 9 Number of crashes by day of week, location, and vehicle type.

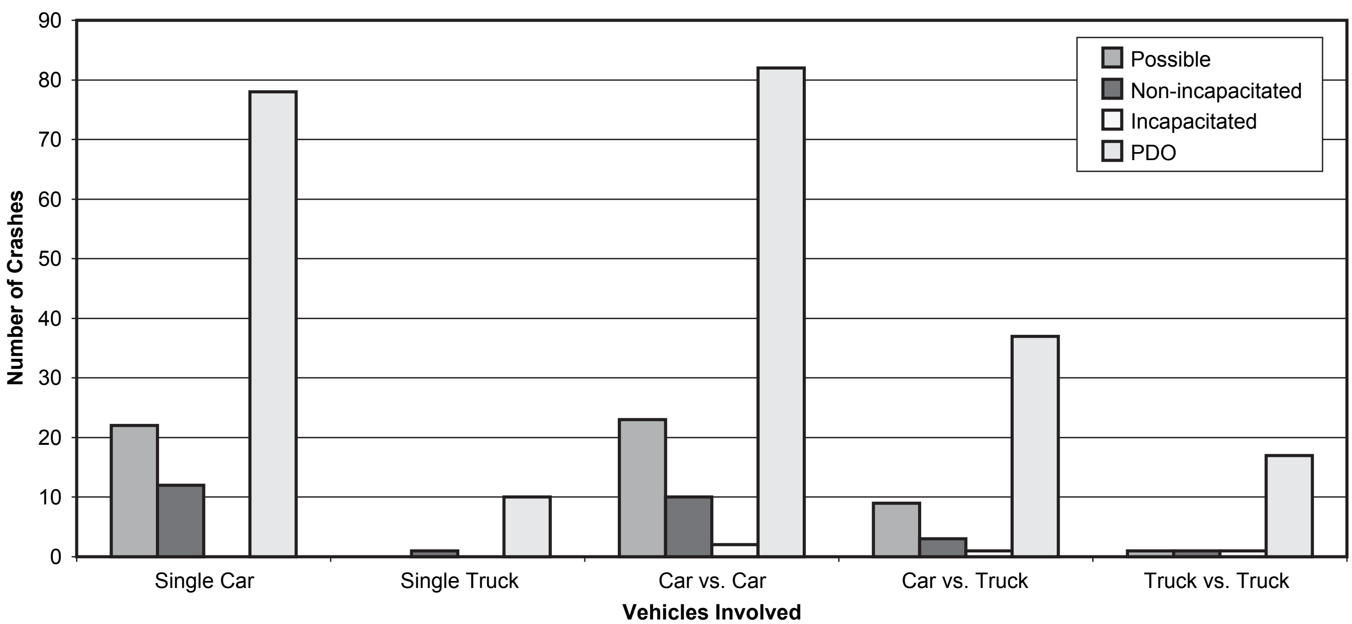

FIGURE 10 Number of crashes by vehicle and severity type.

TABLE 3 Crash Rates for Full Data with Trucks and Passenger Cars

	Southbound						Northbound					
	Outer Lanes			Inner Lanes			Inner Lanes			Outer Lanes		
Mile Marker	Injury	PDO	All	Injury	PDO	All	Injury	PDO	All	Injury	PDO	All
88.1 to 90.6	0.052	0.442	0.494	0.081	0.162	0.242	0.076	0.209	0.285	0.072	0.362	0.434
90.9 to 95.9	0.165	0.546	0.711	0.120	0.241	0.361	0.086	0.250	0.335	0.144	0.491	0.635

Table 4 shows the crash rates by isolating the passenger car and truck traffic exposure (no bus exposure). In this table, for three of four sections, truck-related crashes occur more frequently than passenger car–only crashes given the same exposure. In other words, the number of truck crashes per truck is higher than the number of passenger car crashes per passenger vehicle ceteris paribus. Similar numbers were reported by Miaou (*3*).

Figure 11 shows the crash rates separated by passenger car and truck crashes. This figure offers a clearer picture of the magnitude of truck-related crashes to the overall crash rate. On two of the four sections, truck-involved crashes are the majority, and on the other two, trucks are significant contributors to the overall crash rate. Most of the truck-related rates involve a truck and a passenger car.

DISCUSSION OF RESULTS

The results of the exploratory analysis show that the outer lanes experienced more crashes than the inner lanes, both when raw numbers are used and when exposure is incorporated into the analysis. Given the outcome of the analysis, there is a need to determine the factors that could explain this difference. Possible hypotheses follow.

The analysis performed in this work appears to indicate that trucks have a strong influence on the safety of outer lanes. Truck-related crashes account for more than 40% of all crashes occurring on the outer lanes, yet trucks account for only 30% of the vehicles traveling on the outer lanes. This means that truck-related crashes are overrepresented in outer lanes. Garber and Joshua noted the same outcome in their study of large-truck crashes in Virginia (*24*). It is unclear whether truck traffic levels, highway geometrics, traffic flow states, or a combination of all these factors play a role in truck-related crashes.

As indicated earlier, the safety effects of truck levels, defined as homogeneous and nonhomogeneous traffic flows, are not well understood (*2–4*). The two seminal studies arrive at opposite conclusions. Jovanis and Chang found that an increase in truck traffic increases truck-related crashes (*2*), whereas Winslott Hiselius established no such relationship (*4*). Thus, the jury is still out on this effect. If one makes an abstraction of vehicle performance and its effects on traffic flow states, the exploratory analysis shows that trucks are often involved in sideswipe collisions. It is known that trucks have significant blind spots. Thus, it may be reasonable to assume that increased truck traffic may lead to more sideswipe collisions compared to a similar facility with passenger cars only (though other types of crashes are expected to increase as well, such as run-off-the-road crashes).

Another hypothesis could be related to differences in highway geometrics. For instance, controlling criteria governing relevant highway design elements, such as grades, lane widths, lateral sight distances, or horizontal curves, could affect the vehicle performance of trucks, thus negatively influencing the safety of the facility. At the study locations, however, the roadway geometry between inner and outer lanes is very similar. For instance, the typical cross section, including the lane width, is essentially the same between both sets of roadways. Similarly, the selected study sections do not have any steep grades that would affect the performance of trucks. Perhaps the location of ramps could explain the difference, especially since a large proportion of trucks are involved in sideswipe collisions (e.g., trucks that change lanes near entrance ramps). However, with the current database, it is not possible to investigate whether crashes occurred near an exit or an entrance ramp (i.e., the data do not indicate the lane in which the crash occurred).

It is possible that a portion of the difference for the crash rates between inner and outer lanes could be attributed to the fact that PDO crashes involving a heavy vehicle are more likely to be reported than PDOs involving a single or two passenger vehicles. Although it is possible, one would not expect the driver of a heavy vehicle to flee the scene of a crash. Unfortunately, with the data at hand, the research team was unable to examine the issue of reportable crashes as a function of the vehicle type. Further work is needed on this topic.

The last hypothesis is related to the traffic flow states. A significan amount of research has been conducted in the last 2 or 3 years on the safety effects of traffic flow states on urban and rural freeways (*25–27*). The recent work showed that vehicle density and volume-to-capacity (V/C) ratios have a great impact on freeway safety, although the effects are more significant for urban freeways. Some have argued that a greater variance in the speed distribution of vehicles on a freeway segment increases the risk of collisions (*28, 29*); however, not everyone agrees with this argument (*30*). It is well known that increased truck traffic can have a significant impact on freeway operations (*31*). It is impossible with the current data to evaluate this hypothesis.

Some of these hypotheses could be answered through more sophisticated statistical analyses, combined with the use of disaggregated data (e.g., hourly flows, crashes per lane). For instance, incorporating V/C ratio or vehicle density would help determine the safety effects of traffic flow states as a function of truck traffic levels (*27*). Thus, additional work that uses disaggregated data is needed to understand the characteristics of the differences in safety between outer and inner lanes.

The results of the exploratory analysis appear to suggest that truck-free freeway facilities would have a better safety record than mixed-traffic facilities. This outcome is consistent with other work on this subject. By using simulation tools, others have suggested that removing trucks from mixed-traffic lanes and building exclusive truck facilities would significantly improve operations, which should result in important safety gains (*14, 15*). Additional work should be conducted on this subject before passenger car–only freeway facilities are implemented.

TABLE 4 Crash Rates for Trucks and Passenger Cars Disaggregated by Exposure

		Southbound						Northbound					
		Outer Lanes			Inner Lanes			Inner Lanes			Outer Lanes		
	Mile Marker	Injury	PDO	All	Injury	PDO	All	Injury	PDO	All	Injury	PDO	All
Trucks	88.1 to 90.6	0.000	0.444	0.444	—	—	—	—	—	—	0.079	0.795	0.874
	90.6 to 95.9	0.181	0.724	0.905	—	—	—	—	—	—	0.200	0.898	1.131
Cars	88.1 to 90.6	0.075	0.450	0.525	0.081	0.162	0.242	0.076	0.209	0.285	0.071	0.133	0.133
	90.6 to 95.9	0.164	0.492	0.656	0.112	0.223	0.345	0.078	0.250	0.328	0.130	0.348	0.478

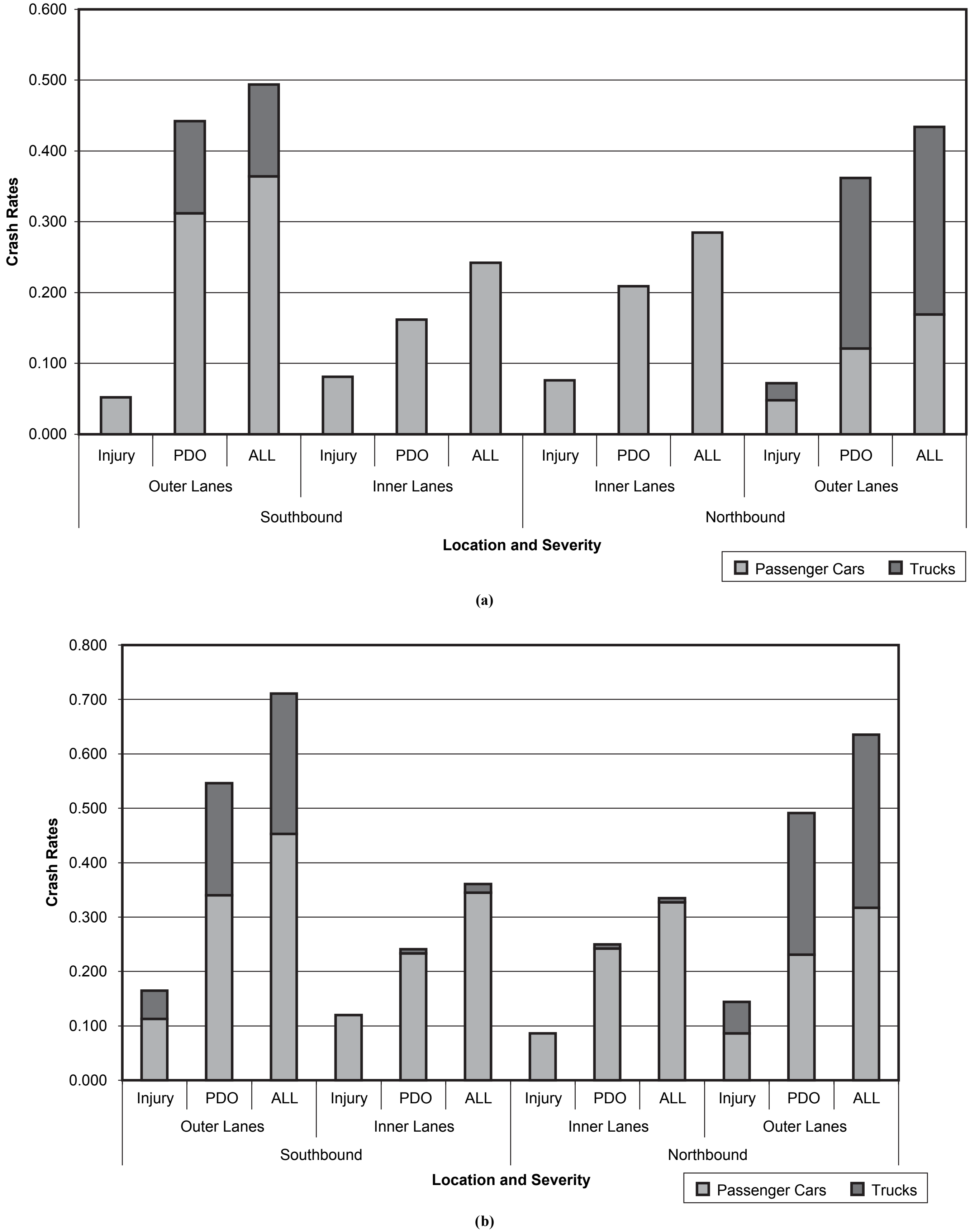

FIGURE 11 Crash rates for trucks and passenger cars by lane designation: (*a*) Mileposts 88.1 to 90.6 and (*b*) Mileposts 90.6 to 95.9.

SUMMARY AND CONCLUSIONS

The objective for this study was to assess whether more homogeneous flows of traffic by vehicle type are safer than the current mixed flo scenario. An exploratory analysis of crash data was conducted on freeway sections of the New Jersey Turnpike for 2002. These sections operate as a dual–dual freeway facility: divided inner and outer lanes. At these locations, the inner lanes are for passenger cars only (homogeneous traffic) with some exceptions. The selected sections, therefore, offered a good opportunity to compare the crash experience between passenger car–only and mixed-traffic rural freeway facilities.

The results of the study showed that the outer roadway experiences more crashes, both when raw numbers are used and when exposure is included in the analysis. The results also show that truck-related crashes contribute significantly to the total number of crashes on the outer lanes. Trucks are overinvolved in crashes given the exposure on these sections. Although the outcome of this study suggests that separating truck traffic from passenger cars for freeway facilities improves safety, further work is needed to understand the contributing factors leading to truck-related crashes in the outer lanes.

There is also a need to compare the cost of truck roadways versus no-truck roadways, given the difference in crashes. The sponsored research will investigate costs, but until then, a computer program (*15*) can evaluate the economic feasibility of exclusive lanes for specifi sites on high-volume, limited-access highways in both urban and rural areas.

ACKNOWLEDGMENTS

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Safety Implications of Multiday Driving Schedules for Truck Drivers

A Comparison of Field Experiments and Crash Data Analysis

Sang-Woo Park, Aviroop Mukherjee, Frank Gross, and Paul P. Jovanis

The detailed analysis of preexisting crash and noncrash data representing an estimated 16 million vehicle miles of travel has revealed strong consistency between crash analysis using data from the 1980s and field experiments conducted in the 1990s. Time of day of driving is associated with crash risk: night and early morning driving has elevated risk in the range of 20% to 70% compared with daytime driving. Overall, 16 of 27 night and early morning driving schedules had elevated risk. Irregular schedules with primarily night and early morning driving had relative risk increases of 30% to 80%. In addition, there remains a persistent finding of increased crash risk associated with hours driving, with risk increases of 30% to more than 80% compared with the first hour of driving. These increases are less than previously reported and are of similar magnitude to the risk increases caused by multiday schedules. Finally, there is some evidence, although it is far from persuasive, that risk increases may be associated with significant off-duty time, in some cases in the range of 24 to 48 h. The implication is that "restart" programs should be approached with caution. Areas for additional research include further studies of crash risk associated with extended off-duty time, closer examination of irregular schedules that better reflect truckload operations, and analysis of irregular schedules with primarily daytime driving (largely nonexistent in this data set) to further explore the effect of variability.

Management of driver hours of service (HOS) for commercial vehicle operators has been a continual safety challenge. After more than 50 years with the same regulations, the U.S. Department of Transportation (USDOT) implemented changes in the HOS in January 2004, only to have them overturned in a lawsuit. The European Union has also been active in the area and is considering changes in its regulations (*1*).

There are many reasons why managing service hours is a challenging task, but one of the most perplexing aspects is the inconsistency in research findings concerning the effect of driving schedules on driver performance and safety. A recent major study sponsored by USDOT (*2*), using instrumented vehicles, off-line tests of driver performance, video recording of driver faces on the road, physiological monitoring, and a series of driver ratings through surveys, found that the principal factor associated with a decline in driving performance was time of day. Number of hours driving (time on task) and cumulative number of days driving were not strong or consistent predictors. The study was one of the most extensive field studies of its kind, involving 80 drivers measured for performance during revenue-producing runs for a carrier operating in both the United States and Canada (to allow legal driving for up to 12 h consecutively).

These findings stand in contrast to work with carrier crash data conducted over several years (*3–6*), in which driving time was most strongly associated with increases in crash risk, and night driving, although significantly associated with crash risk in some situations, was consistently of neither the same magnitude nor the same significance. Others have noted that the use of different performance measures often yields different findings; the search for convergent validity is important (*2*).

The objective for this paper is to examine the effect of multiday driving and continuous driving (time on task) on crash risk. This is an exploratory study that uses preexisting data to seek convergent results. It is recognized that the crash data used in this analysis are from the 1980s and that the measurements from a Driver Fatigue and Alertness Study (DFAS) were conducted in the mid-1990s. Nevertheless, the nature of the driving task is similar, and physiological capabilities of humans are similar. The research explores whether a more detailed examination of time of day of driving, particularly over multiple days, indicates associations with crash risk.

DATA DESCRIPTION

All crash data were obtained from a national less-than-truckload firm. At the time of data collection, the company conducted operations from coast to coast, with no sleeper berths. The findings thus are not intended to typify the trucking industry as a whole. The carrier undertook scheduled service between its own terminals, with significant knowledge of the time to be taken to complete trips and safety supervisors in the field to verify driver behavior. This reduces the incentive for drivers to misstate driving hours on their logs. Although an independent assessment of driving hour data was not feasible, given the type of service provided and the steps taken by the company to adhere to (USDOT) service hour regulations, the researchers believe the driving hour data reflect operations that adhere to HOS regulations in existence at the time.

The analyses presented in this paper use data from 1984 through 1985. The data set consists of accident-involved drivers and non-accident-involved drivers. For accident-involved drivers, the day of the crash serves as the starting point for additional data collection (this day can be called the day of interest). For each crash-involved driver, driving logs are assembled and coded and represent the duty status of the driver on the accident-involved day as well as the previous 7 days. These data are used to develop a detailed characterization of the

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driving status of the accident-involved drivers for each 15 min of each day for an 8-day period (the accident day and 7 prior days). This data structure corresponds to the hours of service policy in effect at the time of operations (maximum multiday driving or on-duty time of 70 h in 8 days).

In addition to the crash data, a data set of two non-accident-involved drivers was assembled by having a person from the trucking firm randomly select two sets of driver logs from the same terminal as each accident-involved driver. In this way, two non-accident drivers are selected as controls for each crash-involved driver, where the selection from the common terminal serves as a mechanism to help control for driving environment. A day and a trip within the day were selected at random for each noncrash driver so there would be a starting point of a randomly selected trip that would be comparable to the accident trip for the crash-involved drivers. (Again, this day that begins the measurement of the driving schedule is referred to as the day of interest.)

The data set includes 954 accident-involved drivers and 1,506 non-accident drivers in 1984 and 887 accident drivers and 1,604 non-accident drivers in 1985 for a total sample size of 5,050 drivers. This is a large data set for a truck safety study, estimated to encompass approximately 16 million vehicle miles of travel (assuming an average of 8 h driving over the 8-day period and an average speed of 50 mph). There are many possible schedules for drivers over an 8-day period; it is only through the use of a large data set that common schedules can be identified for enough drivers to allow statistical estimation of crash risk.

METHODOLOGY

Under the HOS regulation enforced during the time of data collection, drivers could drive for a maximum of 10 h followed by a mandatory minimum 8-h off-duty period. Driving time was divided into 10 1-h periods, starting with the first hour.

Identification of Multiday Driving Schedule

Previous research used cluster analysis on a small subset of the study data (either 6 months or 1 year of crash and noncrash data) (*3–6*). As a result, only 10 sets of driving schedules were identified.

For this analysis, data for a full 2 years were used. As a starting point, the DFAS was carefully reviewed and an attempt was made to extract driving schedules from the data set that included those from the DFAS (*2,* pp. 3-6–3-7). In particular, an attempt was made to identify drivers with regular and irregular schedules over multiple days. It was not possible to identify drivers with 12-h driving times, as used in the DFAS, but the pattern of day and night driving and the irregularity of schedules were specifically sought within the crash data.

Drivers were manually grouped on the basis of their multiday schedule by a search through the record of the 5,050 drivers to find those who started driving at approximately the same time every day—for example, those starting to drive at 10:00 a.m. the day before the crash (the day of interest). An accuracy of plus or minus 2 h was used to group drivers with similar driving schedules. The set of drivers starting at 10:00 a.m. on the day before the day of interest was then searched further for those drivers who started driving at 10:00 a.m. the previous day (2 days before the crash). This process was continued for 3 and 4 days before the crash. In searching back in time from the day of interest, the sample size of drivers was successively reduced, so in some cases it was possible to go back only 1 or 2 days. (See Table 1 for a summary of manually extracted driving schedules and their sample sizes.) The occurrence of a crash on the day of interest was irrelevant to this search; the driving schedules on the days before the crash (or randomly selected matching trip) were the only information used to form the schedules.

Figure 1, an example of one manually derived driving schedule, illustrates the method and the outcome of the driver grouping. The figure illustrates the percentage of drivers within the group that are on duty or driving for every 15 min for 7 days before the day of interest (here, time zero is midnight). Each day is represented by 24 h, so Hour 168 is midnight on the beginning of the eighth day (the day of interest). In this particular driving schedule, more than 90% of the drivers are on duty or driving starting at around 10:00 a.m. for the 2 days before the day of interest. Before these 2 days, the drivers have a less well-structured pattern of driving, with at most 40% of the drivers on duty at any one time. Note also that there is virtually no early morning driving for the 2 days before the day of interest. This grouping then represents drivers with a regular, largely daytime driving schedule for two consecutive days. The figure contains two very similar lines, one representing data from 1984 and one of data from 1985, although the procedure would not necessarily lead to this outcome.

Table 1 summarizes all 23 manually developed driving schedules. Schedules C21 through C28 are regularly scheduled drivers who started driving at either 10:00 a.m. or midnight for 1 to 4 days before the crash. (Figure 1 represents Schedule C22.) These particular times were selected because they match the time of day used in the regularly scheduled driver in the DFAS. In addition to time of day, there is an interest in the off-duty status of drivers. Drivers who had one or more days off duty and regular driving on previous days are represented in Schedules C29 to C35.

Irregularly scheduled drivers were first extracted on the basis of the irregular schedules driven in DFAS. Schedules C36 and C37 were derived from the description of the irregular schedule in the DFAS report (*2*). Additional irregular schedules included

- Drivers who alternated the start of driving between 7:00 and 10:00 p.m. (Schedules C38 through C41);
- Drivers who started driving at 10:00 p.m. the day before the crash and shifting ahead in time by 3 h (to 7:00 p.m. and then 4:00 p.m., respectively); and
- Drivers with no driving the 2 days before the crash and very infrequent driving previous to those days (these drivers were grouped in Schedule C43).

An example of an irregularly scheduled driver group, C39, is shown in Figure 2. As in Figure 1, the two lines representing separate years of data are very similar.

After allocation to the 23 manually derived groups, there remained approximately 2,500 unallocated drivers. Cluster analysis was used to allocate these remaining drivers to one of 20 clusters by using the same procedure as in the previous research (*4*). The schedules derived from cluster analysis were assigned schedule numbers C1 through C20.

Figure 3 is a schedule identified through cluster analysis:

- The drivers in this schedule (109 in all) drive during the night and early morning hours starting 4 days before the day of interest and continuing during the third day before the day of interest.
- Two days before the day of interest (around Hour 144) there is little driving, and most drivers appear to be off-duty during this time.

TABLE 1 Manually Identified Driving Schedules

Schedule Number (Sample Size)	4 Days Before Crash	3 Days Before Crash	2 Days Before Crash	1 Day Before Crash
C1-C20	Created using cluster analysis software			
C21 (481)	—	—	—	10 a.m.
C22 (125)	—	—	10 a.m.	10 a.m.
C23 (28)	—	10 a.m.	10 a.m.	10 a.m.
C24 (19)	10 a.m.	10 a.m.	10 a.m.	10 a.m.
C25 (517)	—	—	—	12 a.m.
C26 (134)	—	—	12 a.m.	12 a.m.
C27 (52)	—	12 a.m.	12 a.m.	12 a.m.
C28 (17)	12 a.m.	12 a.m.	12 a.m.	12 a.m.
C29 (32)	—	—	10 a.m.	Off-duty
C30 (20)	—	10 a.m.	10 a.m.	Off-duty
C31 (41)	—	—	12 a.m.	Off-duty
C32 (19)	—	12 a.m.	12 a.m.	Off-duty
C33 (11)	12 a.m.	12 a.m.	12 a.m.	Off-duty
C34 (29)	—	10 a.m.	Off-duty	Off-duty
C35 (25)	10 a.m.	10 a.m.	Off-duty	Off-duty
C36 (83)	—	—	3:30 p.m.	11 a.m.
C37 (20)	—	7 p.m.	3:30 p.m.	11 a.m.
C38 (657)	—	—	—	10 p.m.
C39 (113)	—	—	7 p.m.	10 p.m.
C40 (67)	—	10 p.m.	7 p.m.	10 p.m.
C41 (24)	7 p.m.	10 p.m.	7 p.m.	10 p.m.
C42 (20)	—	4 p.m.	7 p.m.	10 p.m.
C43 (362)	—	—	Off-duty	Off-duty

- On the day before the day of interest, drivers again start to drive, a bit earlier than before, starting at late afternoon and early evening times.
- Around the night of the fourth day before the crash day, 90% or more of the drivers are on duty; there are also very few times when no drivers are on duty. This is in contrast to the manually derived schedules, which are more precise in that they have more well-defined peaks and troughs.

Table 2 contains a qualitative description of the nature of the driving schedule for each of the 20 clusters, including the sample size.

Characterization of Driving Time

One of the most important aspects of early studies of safety and HOS (*7, 8*) is the need to characterize continuous driving by using the notion

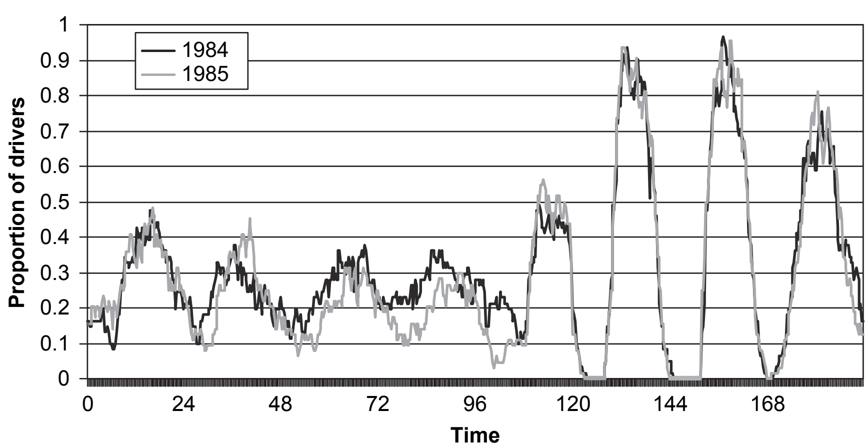

FIGURE 1 Regular multiday driving schedule: start driving at 10:00 a.m. for 2 days before day of interest (Cluster 22).

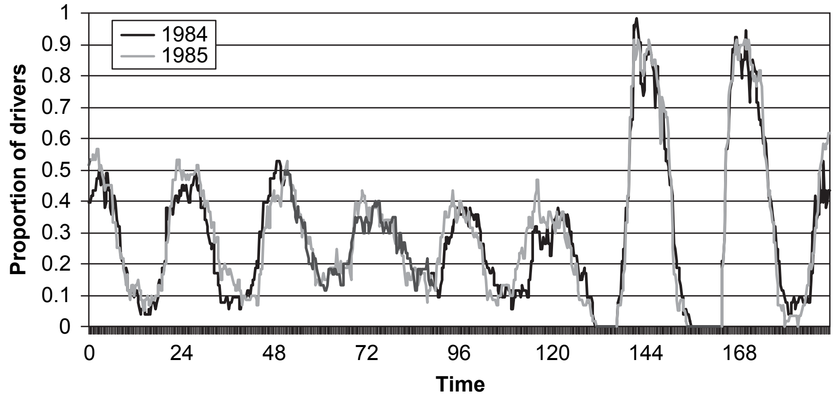

FIGURE 2 Irregular schedule: initiate driving at 10:00 p.m. 1 day before and at 7:00 p.m. 2 days before day of interest (Cluster 39).

of survival. A driver who has a crash after driving 9 h, for example, has successfully survived the first 8 h. Any model that attempts to characterize the probability of a crash as a discrete outcome must recognize in its statistical formulation that most drivers can thus be considered to have multiple outcomes during a crash-involved trip: the survival for hours before the crash and a crash outcome (i.e., a failure) for the period of the crash. Early statistical models of driving-time crash risk proposed the use of survival theory in recognition of this phenomenon (*9, 10*).

Subsequent research used a data replication scheme and logistic regression to capture the survival effect (*5*). In the described case of a driver having a crash in the ninth hour of driving, each prior hour must be coded individually with an outcome of a nonaccident; this would occur for each of the 8 h before the crash event. In addition, the ninth hour would be coded as the alternate outcome, a crash. Similarly, a 9-h trip without a crash would have to be replicated for all 9 h with a nonaccident outcome. It is only through this replication that the logistic regression can account for the survival phenomenon. There is evidence in the statistical literature to support the use of this type of model (*11, 12*). The model is thus

$$P_{it} = P\left(Y_{ti} = 1 \middle| Y_{t'i} = 0 \text{ for } t' < t, X_i\right) = \frac{\exp\left[g\left(X_i, t, \beta\right)\right]}{1 + \exp\left[g\left(X_i, t, \beta\right)\right]} \qquad (1)$$

The model is interpreted as the probability that driver i has an accident (outcome $Y = 1$) at time t, given survival until that time (i.e., an outcome $Y = 0$ for all periods t' before period t) is given by the familiar logistic function with time t, predictor variables X, and estimated parameters β. A linear addition function is assumed for $g(X, t, \beta)$.

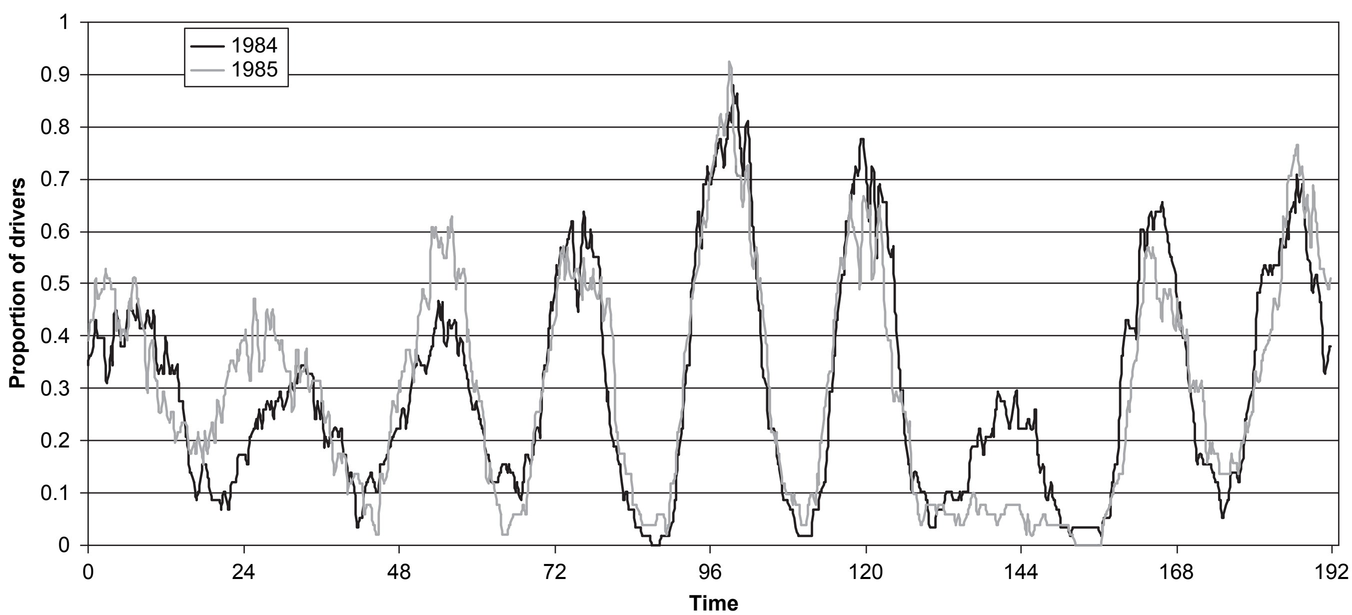

FIGURE 3 Schedule derived from cluster analysis (Cluster 20-7).

TABLE 2 Driving Schedules Derived from Cluster Analysis

Cluster #	Description	Sample Size
C1	Regular night and early morning driving for 3 days prior to day of interest	86
C2	Consistent regular daytime driving for all 7 prior days	200
C3	Afternoon and night driving 1 day prior; day off 2 days prior, and afternoon and night driving previous days	146
C4	Little driving 2 days prior; consistent daytime driving prior days	114
C5	Little driving 1 day prior; daytime driving 2–3 days prior	63
C6	Early morning to noon driving consistently for all days	152
C7	Little driving 1 day prior; night and early morning driving consistently for 4–5 days prior to day off	109
C8	Night and early morning driving; almost all drivers on duty 5 days prior to day of interest; decreasing numbers on duty to 1 day prior	121
C9	3–4 days off duty just prior to day of interest, but night and early morning prior to that	97
C10	Consistent daytime driving for 4 days prior; little driving 6–7 days prior	123
C11	Night and early morning driving 2–3 days prior; largely a day off 4 days prior; consistent night and early morning driving before that	101
C12	Consistent afternoon and late night driving for 2 days prior; little driving for 3–5 days prior	182
C13	Evening and night driving day before, with some drivers on duty 2 days before; little driving 3–5 days prior	146
C14	Late night driving night before; some are off duty 2 days before, but very consistent night and early morning driving 3–7 days before	113
C15	Intermittent daytime driving 1–3 days before; very consistent daytime driving 4–7 days before	86
C16	Consistent afternoon and night driving for 4 days prior	156
C17	Night and early morning driving for 3 days prior; almost no driving 5–7 days prior	148
C18	Very consistent daytime driving 1–2 days prior; day driving before that but not with high % of drivers	97
C19	Daytime driving 2 days before; minimal driving 3–7 days prior	138
C20	Night and early morning driving 3–4 days prior; 5 days prior is off duty; 6–7 days prior night and early morning	104

EMPIRICAL RESULTS AND INTERPRETATION

Two sets of models were estimated with the data. Model 1 was developed to assess the effect of driving time in which the time is divided into 10 1-h periods and the first hour (designated T0) serves as the baseline. The second model retains driving time and adds as covariates the 43 driving schedules manually derived and developed by cluster analysis. The interpretation of the model is that a parameter statistically different from zero implies that a driver with that characteristic has a significantly higher crash risk compared with the first hour. In this way, the model estimates the relative risk of a crash compared to the baseline. The baseline for multiday driving schedules is Schedule C22, daytime driving for 2 days before the day of interest; changes in risk for driving schedules are relative to this baseline risk.

Interpretation of Model 1

As seen in Table 3, the positive parameter in each covariate represents an increase in the log of the odds ratio or, more simply, an increase in the probability of accidents among the drivers in the specific driving time category compared with the drivers in the baseline category (i.e., the first hour). All driving hour variables are significant at $\alpha = 0.05$, which leads to the rejection of the hypothesis of constant hazard over time. The crash risk increases slightly, but significantly, as driving time increases through the fourth hour of driving. Statistical tests of equality of Parameters 1 through 4 fail to reject the null hypothesis that the parameters are equal. Parameters for Hours 5 through 10, however, are all significantly higher than the baseline first hour and significantly higher than Hours 2 through 4 but are unable to be differentiated from each other in additional statistical tests. One may thus infer that crash risk appears to increase only slightly between the first and fourth hours of driving, increases significantly in the fifth hour, and is sustained at a higher level through Hour 10. Importantly, the risk trend with driving time differs in comparison to earlier findings (*5*): the risk increase after Hour 4 (variable T5) is not nearly as steep, particularly in the last hour of driving. Although the crash risk cannot be statistically differentiated, the trend in risk is a general increase from Hours 5 through 10.

TABLE 3 Model 1 Estimates: Effect of Driving Time

	β	S.E.	Sig.	Exp(β)
Constant	−1.238	.707	.080	.290
D.H. < 1*				1.000
1 ≤ D.H. < 2	.229	.113	.043	1.257
2 ≤ D.H. < 3	.348	.111	.002	1.417
3 ≤ D.H. < 4	.287	.114	.011	1.333
4 ≤ D.H. < 5	.623	.107	.000	1.865
5 ≤ D.H. < 6	.601	.109	.000	1.825
6 ≤ D.H. < 7	.608	.111	.000	1.837
7 ≤ D.H. < 8	.678	.112	.000	1.969
8 ≤ D.H. < 9	.555	.122	.000	1.741
9 ≤ D.H.	.746	.135	.000	2.108

D.H. = driving hours
*Baseline category

Interpretation of Model 2

Estimation results for Model 2 are summarized in Table 4. All the driving time variables are significant and have relative magnitudes and interpretations similar to those in Model 1.

The pattern of significance for the multiday driving schedules is of particular interest. In keeping with the recommendations of others in the safety field (*13*), the discussion of each parameter is conducted without a strict use of null hypothesis tests of significance; a liberal level of significance (any with alpha less than 0.25) will be used to screen driving schedules and identify those of possible interest, a procedure consistent with the exploratory nature of the research.

By using the screening criteria of alpha less than 0.25, 21 schedules were identified for further exploration (C1, C2, C7–C9, C12, C13, C16, C17, and C20 derived from the cluster analysis and C25, C27, C31, C32, C34, C35, C38–C40, C42, and C43 derived manually). Importantly, 16 of the 21 involve increased crash risk associated with night and early morning driving, irregular schedules, or both. Closer examination indicates that 16 of the 27 night and early morning driving schedules (C1, C7–C9, C12, C13, C16, C17, C20, C25, C27, C 32, C38–C40, C42) have elevated crash risk compared to the baseline.

The exceptions to the general trends are also of interest. There is a reduction in risk for Schedule C2, consistent regular daytime driving for all 7 days, further evidence of the safety benefits of regular driving schedules. Schedule C31, drivers who started driving at midnight 2 days before the day of interest but who had a day off in between, is one of the few driving schedules that showed a decrease in crash risk associated with a day or more off duty. However, Schedules C34 and C35 indicate increased risk for daytime driving, immediately after 2 full days off duty, as does Schedule C32 for drivers starting at midnight after 2 days off duty.

Lastly, Schedule C43 consists of drivers who are off duty for the 2 days before the day of interest and before that are infrequently driving. This schedule may reflect drivers from the "extra board," who may differ in some other fundamental ways from other drivers at the firm; for that reason they are separated from the other schedules.

Taken as a whole, Model 2 shows rather conclusively that night and early morning driving results in increased crash risk relative to daytime driving and that irregular schedules during the night also have elevated risk. The benefits of extended off-duty time are unclear: in some cases there are risk decreases, but there are also several cases of risk increases.

Comparison of parameter scale for driving schedule and driving time indicate that many schedules have a relative risk increase comparable to driving time. For example, Clusters C32, C34, C38, C40, and C42 all have parameters in the range of 0.5 to 0.7, indicating a relative risk increase of 60% to 90% compared to the baseline (see the last column of Table 4); previous modeling did not indicate risk increases of nearly these magnitudes.

TABLE 4 Driving Time and Multiday Schedule Model Estimates

	β	S.E.	Sig.	Exp(β)
Constant	−3.688	.186	.000	.025
T1	.230	.113	.041	1.259
T2	.351	.111	.002	1.421
T3	.292	.114	.010	1.339
T4	.632	.107	.000	1.882
T5	.612	.109	.000	1.844
T6	.625	.111	.000	1.867
T7	.700	.112	.000	2.014
T8	.581	.122	.000	1.788
T9	.786	.135	.000	2.194
C1	.284	.247	.251	1.328
C2	−.271	.223	.225	.763
C3	.197	.222	.373	1.218
C4	.238	.233	.307	1.269
C5	.172	.275	.532	1.188
C6	.129	.222	.560	1.138
C7	.404	.230	.079	1.498
C8	.363	.226	.109	1.437
C9	.301	.235	.200	1.352
C10	.080	.236	.735	1.083
C11	.166	.243	.495	1.180
C12	.240	.210	.252	1.272
C13	.262	.219	.232	1.299
C14	.086	.238	.716	1.090
C15	−.229	.283	.418	.796
C16	.286	.215	.182	1.332
C17	.254	.217	.243	1.289
C18	.150	.245	.540	1.162
C19	−.039	.233	.866	.962
C20	.289	.235	.219	1.334
C21	.059	.192	.759	1.061
C23	.251	.451	.579	1.285
C24	−.480	.534	.368	.619
C25	.311	.183	.090	1.365
C26	.115	.226	.609	1.122
C27	.370	.295	.210	1.447
C28	−.283	.535	.597	.754
C29	−.150	.420	.721	.861
C30	−.457	.534	.392	.633
C31	−.567	.418	.175	.567
C32	.608	.384	.114	1.837
C33	−.472	.736	.522	.624
C34	.513	.332	.123	1.670
C35	.451	.367	.219	1.570
C36	−.099	.274	.717	.905
C37	−.294	.484	.544	.745
C38	.647	.182	.000	1.909
C39	.482	.231	.037	1.619
C40	.605	.248	.015	1.831
C41	.337	.366	.357	1.401
C42	.764	.370	.039	2.146
C43	.479	.197	.015	1.614

DISCUSSION OF RESULTS

Among the schedules that involved night driving and no days off immediately before the day of interest, nine (C1, C12, C13, C16, C17, C20, C25, C27, C38) of 12 schedules have elevated risk (see Figure 4 for a summary). Drivers with 1 or 2 days off immediately before the day of interest have elevated risk in three (C7, C8, C32) of seven cases, and drivers with irregular schedules have elevated risk in four (C9,

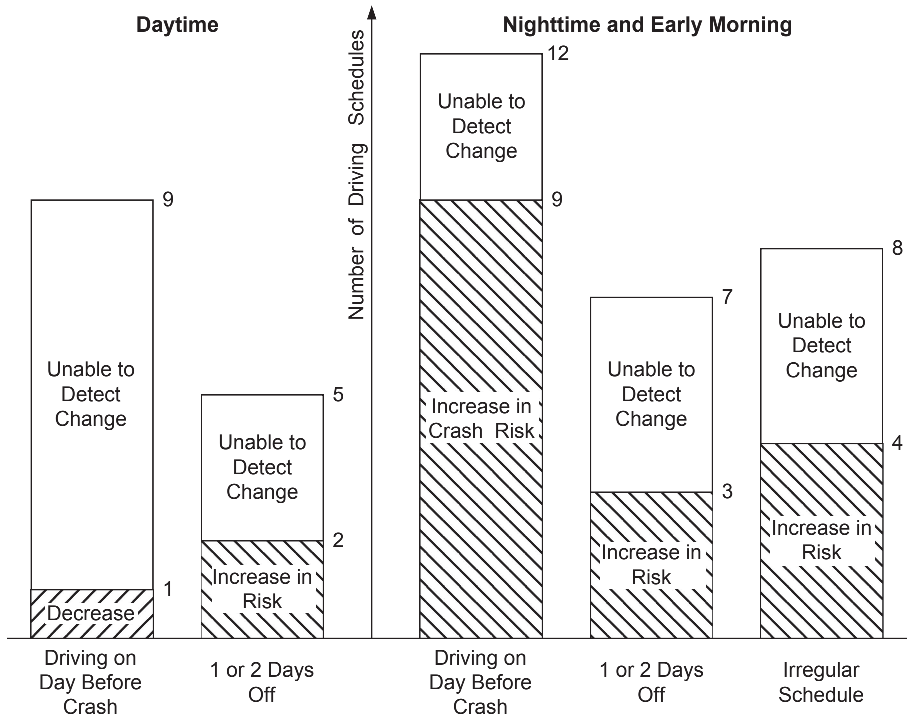

FIGURE 4 Detailed analysis of crash risk during daytime and night–early morning driving schedules.

C39, C40, C42) of eight cases. These detailed comparisons further highlight the elevated risk posed by night driving compared to the baseline regular daytime driving.

There is also evidence that even as much as a 24-h off-duty period may not be sufficient to alleviate the elevated risk of night and early morning driving. Driving Schedules C7 to C9 (averaging about 100 drivers in each group) involve drivers with night and early morning driving and include large amounts of off-duty time 1 or 2 days before the day of interest; all show elevated crash risk. A similar result appears for Schedule C32, although the sample size is only 19 drivers. These findings raise questions about the efficacy of a "restart" period (*14*); there appears to be evidence from this analysis that 24 h and perhaps 48 h may be insufficient, particularly for night and early morning driving. Further, the elevated risk associated with Schedules C34 and C35 indicates that 2 days off duty before driving may actually elevate risk, compared to more regular schedules even for daytime driving. This may be because of the relative unfamiliarity of driving a heavy vehicle again, or other personal factors, but the evidence exists for those driving at night as well as during the day.

The inconsistency with the DFAS study is the continued importance of driving time (time on task) as a significant correlate of crash risk. This result has been consistently obtained by one of the coauthors, by using partially overlapping data sets. It is interesting to recall, however, that increases in crash risk beyond the fourth hour were also observed in the 1970s (*7*). This remains an area of additional study.

By examining the findings in the context of the HOS implemented in 2004 in the United States, there appears to be support for the changes in regulations that sought more regular schedules. Several of the driving schedules with the highest relative crash risk (e.g., C38, C39, and C40) involved irregular schedules. Although the sample size in each group was small, the increase in relative risk was large and significant. Previous studies that used smaller crash data sets were unable to identify this important effect.

CONCLUSIONS

The detailed analysis of drivers representing an estimated 16 million vehicle miles of travel has revealed a stronger consistency between field experiments, such as DFAS, and crash data analysis than has previously been reported. In particular, the time of day of driving is significantly associated with increased crash risk: drivers with predominately night and early morning schedules have crash risk that is 20% to 70% higher than drivers in the baseline regular daytime driving schedule. Drivers with irregular schedules also have elevated risk, again in the 30% to 80% range. These findings of convergent validity are an important independent verification of some of the DFAS findings.

In addition, there remains a persistent finding of increased crash risk associated with hours driving, with risk increases of 30% to more than 80% compared to the first hour of driving. These increases are less than previously reported and are of magnitude similar to the risk increases caused by multiday schedules. Finally, there is some evidence, although it is far from persuasive, that there may be risk increases associated with significant off-duty time, in some cases in the range of 24 to 48 h. The implication is that restart programs should be approached with caution.

Areas for additional research are many, including further studies of crash risk associated with extended off-duty time, closer examination of irregular schedules that better reflect truckload operations, and analysis of irregular schedules with primarily daytime driving (largely nonexistent in this data set) to further explore the effect of variability. These findings, taken as a whole, support the case of continued research and evaluation of HOS along with other truck safety regulatory actions.

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Pilot Test of Fatigue Management Technologies

David F. Dinges, Greg Maislin, Rebecca M. Brewster, Gerald P. Krueger, and Robert J. Carroll

This study involved over-the-road testing of four fatigue management technologies (FMTs) in trucking operations in Canada and the United States. Technologies bundled into a single intervention came from four fatigue management domains: one providing objective information on driver sleep need, one providing objective information on driver drowsiness, one providing objective information on lane tracking performance, and one reducing the work involved in controlling vehicle stability while driving. The objective was to determine driver reactions to such technologies and whether FMT feedback would improve alertness, especially during night driving, or increase sleep time on workdays or nonworkdays. A within-subjects crossover design was used to compare the effects of FMT feedback to no feedback. Each driver underwent the conditions in the same order: 2 weeks of no feedback (control) followed by 2 weeks of FMT feedback (intervention). Data from the devices and other driving performance variables were recorded every second of truck operation for 28 days for each driver, with a resulting 8.7 million data records among the 38 drivers. Support was found for FMT effects. During night driving, FMT feedback significantly reduced driver drowsiness ($p = 0.004$) and lane tracking variability ($p = 0.007$). However, there was evidence from probed psychomotor vigilance task testing that these improvements may have had cost because of the effort (in attention and compensatory behaviors) required to respond to information from the devices. In general, participants agreed that commercial drivers would benefit from FMT and were more positive about those involving vehicle monitoring than those involving driver monitoring.

There are a growing number of technologies that purport to help drivers manage fatigue and drowsy driving (*1–3*). In addition to establishing their validity to detect fatigue, there is a critical need to determine whether feedback from such technologies during driving could affect the behavior or alertness of commercial motor vehicle operators. Building on previous work by the U.S. Department of Transportation (USDOT), a study was carried out on the effects of feedback from a group of fatigue management technologies (FMT) bundled as a single intervention. Sponsored by the Federal Motor Carrier Safety Administration (FMCSA) and Transport Canada, in cooperation with the American Transportation Research Institute (ATRI), the study was tasked to develop an experimental design and instrumentation plan and to conduct a pilot test of commercial truck drivers' reactions to a combination of FMT, under federally mandated hours of service in both Canada and the United States. Since it was neither cost-effective nor practical to conduct a separate study of each individual technology, the selected technologies were combined and tested as a set within a single field trial that had two phases: one in Canada and one in the United States. The project involved an extensive over-the-road test of the combined FMT. The objective was to determine how drivers, engaged in over-the-road trucking operations, reacted to FMT and whether the technologies would improve the alertness and fatigue awareness of commercial truck drivers by providing information feedback about changes in sleep need, in drowsiness, and in driving performance during routine driving schedules. Specifically the research sought to determine whether feedback from combined FMT would enhance drivers' alertness and performance at work and increase sleep times on workdays or nonwork days. A secondary specific aim was to obtain driver reaction to FMT. It was hypothesized that deployment of FMT would result in improved driver alertness and performance while driving (Hypothesis I) and in increased sleep time (Hypothesis II) and under both current U.S. hours of service and Canadian hours of service.

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METHODS

Criteria for FMT Selection

Technologies selected were bundled into a single intervention from four fatigue management domains: one providing objective information on driver sleep need, one providing objective information on driver drowsiness, one providing objective information on lane tracking performance, and a technology that reduced the work involved in controlling vehicle stability while driving. Although each technology is described separately, the effects of feedback from them was investigated as a single intervention encompassing all four. This was deliberate—the project was not designed or resourced to compare the impact of individual FMT to each other or to compare the effects of FMT in Canadian versus U.S. drivers. The selection of specific technologies was not an endorsement of their validity or reliability. Technologies were selected for use in the pilot study because (*a*) each was representative of one of the four fatigue management domains, (*b*) each was available for study through the cooperation of their respective developers, and (*c*) each could be implemented by using participating company trucks.

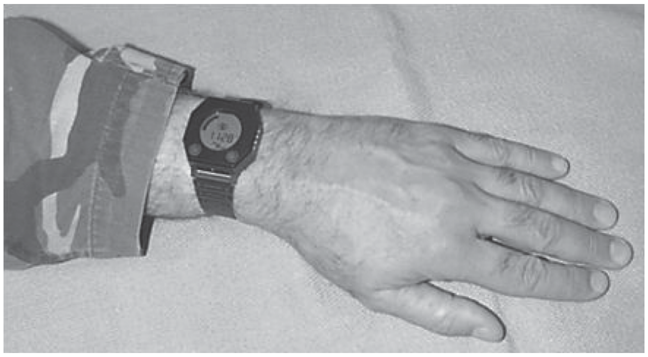

FIGURE 1 WRAIR SleepWatch.

SleepWatch

The technology selected for providing feedback to drivers on sleep need was the actigraphically based, wrist-worn SleepWatch (Precision Control Design, Inc., Ft. Walton Beach, Florida) shown in Figure 1, combined with an internal algorithm called the sleep management model from Walter Reed Army Institute of Research (WRAIR). Investigators at WRAIR developed the wrist-worn actigraph device used and the algorithm to detect sleep in actigraphy data (*4,* p. 149; 5–*8*). Wrist-worn actigraphic monitoring of drivers' rest–activity patterns, with feedback regarding estimated sleep need, was judged to be a potentially useful objective way to inform drivers of the development of cumulative sleep debt (*9–11*) and the need to obtain more sleep or take additional alertness-promoting countermeasures. SleepWatch displayed a clock and an analogue "performance fuel gauge" based on sleep need. When a button was pressed, an estimated numeric value of performance readiness was displayed as a percentage of from 0% to 100% performance (see Figure 1). The feedback aspects of the SleepWatch (i.e., the performance fuel gauge and the numeric value of performance readiness) were suppressed in the control (no-feedback) condition although objective data on sleep time were still collected by using the sleep management model.

CoPilot

The technology selected for providing drowsiness feedback to drivers was the CoPilot system (Attention Technologies, Pittsburgh, Pennsylvania) for monitoring percent eyelid closure (PERCLOS). USDOT-funded research in the laboratories of Wierwille et al. (*12–14*), Dinges et al. (*1*), Mallis et al. (*15*), and Dinges et al. (*16*) led to the discovery that slow eyelid closures were a highly reliable measure of lapses of attention caused by sleepiness or drowsiness, which led to the development of CoPilot, an infrared-based retinal reflectance monitor for eye closure detection by R. Grace of Carnegie Mellon University. CoPilot used a structured illumination approach and identified a driver's eyes by using two identical images with different sources of infrared illumination. The image of the face was passed through a beam splitter that reflected the image onto the lenses of a camera with an 850-nm filter and a camera with a 950-nm filter. The 850-nm filte yielded a bright-eye camera image (i.e., distinct glowing of the driver's pupils), as seen in Figure 2*a*. The 950-nm filter yielded a dark-eye image, as seen in Figure 2*b.* A third image enhanced the bright eyes by calculating the difference of the two images (Figure 2*c*). A driver's eyes were identified in this third image by applying a threshold determined adaptively by examining the average brightness in each video frame. The CoPilot infrared retinal reflectance device requires it to be operated at low ambient light levels. It was mounted on the truck dashboard, typically just to the right of the steering wheel (Figure 3). Feedback from the system was provided on a separate digital display box and consisted of a CoPilot proprietary algorithm score from 0 to 99, in which 0 indicated maximum eyelid closure and 99 indicated least eyelid closure. Eyelid closure feedback information was active during the 2 weeks drivers operated in the feedback condition. The numeric feedback from the PERCLOS system was disabled during the no-feedback condition, but PERCLOS information was still being recorded for analyses.

SafeTRAC

The technology selected for providing lane tracking feedback to drivers was SafeTRAC (Applied Perception and AssistWare Technology, Wexford, Pennsylvania). Lane tracking, which refers to monitoring the position of the vehicle in the driving lane and detection of lane drifting, weaving, or variability in tracking the lane, is a well-established measure of driving performance with a long history of use. In addition to having excellent face validity in driving safety, many studies of fatigue-related driving deficits have found variability in lane tracking to be one of the more sensitive measures of drowsiness and fatigue. SafeTRAC consisted of a video camera mounted on the windshield (Figure 4) and coupled to a small computer that continuously analyzed the image of the road, lane markings, and other roadway features. Lane departures, erratic movements, and other possible errors were detected. Intentional lane shifts indicated by the turn signal were designed to be ignored by the system. The SafeTRAC feedback monitor was mounted on the dashboard just to the left of the

(a) (b) (c)

FIGURE 2 Eye images taken by CoPilot: (*a*) bright-eye, (*b*) dark-eye, and (*c*) difference images.

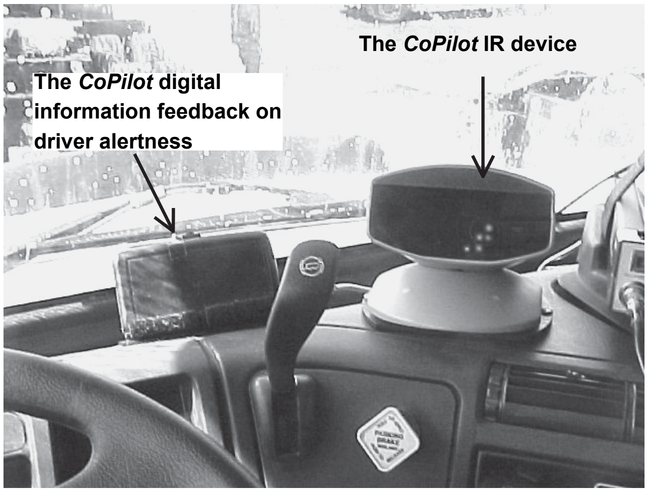

FIGURE 3 CoPilot infrared retinal reflectance monitor.

steering wheel. Feedback from the system consisted of a 0-to-99 scale, in which 0 indicated most erratic lane tracking and 99 indicated least erratic lane tracking, according to a proprietary algorithm. If a driver made an abrupt deviation from the lane without signaling, SafeTRAC provided an auditory warning signal. As with other FMT technologies, feedback information from the SafeTRAC device was active during the 2 weeks drivers operated their trucks in the feedback condition. The numeric feedback from the system was disabled during the 2-week no-feedback period, although it still collected objective data on lane tracking.

Howard Power Center Steering System

The technology selected for reducing the physical work of controlling vehicle stability while driving was the Howard Power Center Steering (HPCS) system (River City Products, San Antonio, Texas). Unlike the other FMT technologies that were designed to provide feedback to drivers on behavioral alertness relative to fatigue based in sleep and circadian biology, the HPCS system was designed to lessen physical fatigue associated with drivers fighting the steering wheel in cross winds. Heavy-vehicle stability and control problems

FIGURE 4 SafeTRAC lane-tracking monitor.

contribute to the work of driving a truck, inducing fatigue because of the often continuous amount of driver steering corrections needed to counteract the unstable behavior of the castered truck wheels. The physical workload associated with fighting the steering wheel in cross winds is particularly fatiguing to neck and shoulder muscles. There was a need to determine whether a technology that lessened this physical workload on drivers would result in less fatigue. The technology that best fulfilled this requirement and that was tested in the pilot study was the HPCS system. HPCS involved a hydraulic device attached to a truck's tie rod and steering system to reduce the physical demands of driving. The system consisted of two principal components: the hydraulic power centering cylinder and the air-activated hydraulic pressure accumulator. The normal operation of the system was automatic and required little attention from the driver. The driver controlled the desirable hydraulic pressure on a panel by adjusting air pressure, which increased or decreased effectiveness of the system. The system was turned on and off by the driver via a switch the driver pressed to release air pressure in the accumulator. Unlike the SleepWatch, the CoPilot drowsiness monitor, and the SafeTRAC lane tracker, HPCS did not provide numeric feedback. Rather, this system was turned on in the feedback condition, and it was off in the no-feedback condition. When the system was turned on, drivers could feel the steering wheel stability relative to when the system was turned off. As with the measurements made by other FMT technologies, steering wheel variability was recorded electronically in both the feedback (HPCS turned on) and no-feedback (HPSC turned off) conditions. Figure 5 displays HPCS as used in the project trucks.

Other Non-FMT Data Recording Technologies

Volunteer drivers' trucks were instrumented with the Accident Prevention Plus (AP+) onboard recording device (black box) to continuously record a range of truck motion variables (speed, lateral acceleration, etc.) as well as information from three of the FMT devices (CoPilot, SafeTRAC, HPCS). Volunteer drivers completed a daily diary on work–rest activities and performed the 10-min psychomotor vigilance task (PVT) (*17*) twice daily—midway in each trip and at the end of each trip—as an independent validation of behavioral alertness levels.

Education on Alertness and Fatigue Management

In addition to training in the use of all these technologies, drivers received education on alertness and fatigue management before driving the instrumented trucks at the beginning of the 2-week FMT no-feedback portion of the study and at the beginning of the 2-week FMT feedback portion of the study. Drivers were provided a 3-h course entitled "Mastering Alertness and Managing Driver Fatigue" (sponsored by FMCSA and ATRI), which was prepared for this study (*18*). The course was taught to four drivers at a time, 2 to 3 days before they were issued an instrumented truck. The education module encouraged drivers to be responsible for alertness levels at all times throughout the study. Since all drivers in the study received it as part of risk mitigation, it was not varied between feedback and no-feedback conditions. It likely increased drivers' acceptance of the FMT.

Human Factors Structured Interview Questionnaire

Following completion of the study, drivers were debriefed and completed the human factors structured interview questionnaire, in which

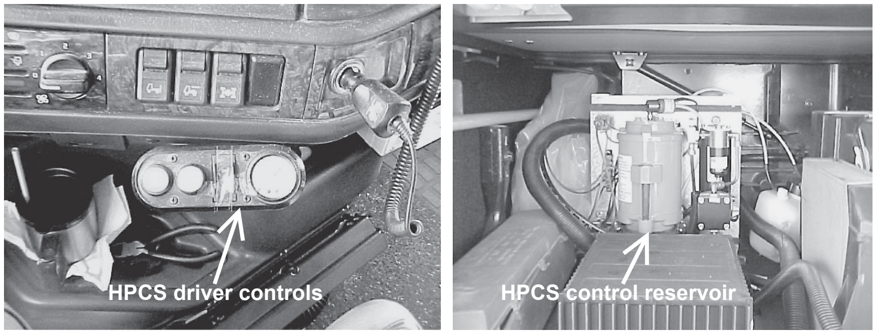

FIGURE 5 HPCS.

they reported reactions to all interventions, measures, and technologies used in the study.

Experimental Design

A within-subjects crossover design was used in both phases (countries) of the study to compare the effects of feedback from combined FMT with no feedback from FMT. The design did not require manipulating or controlling what the participating companies and drivers did, what schedules the drivers adhered to, or what operating practices they followed. Rather, the FMT intervention and data collection were applied to existing routine trucking operations. Thus, for comparisons of the effects of FMT feedback versus no feedback, volunteer drivers served as their own controls—undergoing both conditions under nearly identical circumstances (i.e., a given truck driver drove comparable trucks and schedules during both feedback and no-feedback conditions). A crossover design is efficient and has a number of advantages over an independent-groups design. It ensures roughly the same intersubject variability across both conditions, it provides an opportunity for subjects to explicitly compare and contrast conditions, and it requires fewer subjects than an independent-groups design, which makes it more feasible from both cost and time line perspectives. On the downside, a crossover design necessarily burdens a smaller group of subjects with more recording time than would be the case in an independent-groups design. If too burdensome, subjects may fail to complete all conditions. This occurred to some extent in both phases of the present study but was not a major problem.

The focus of the study was not on comparing Canadian and U.S. operations but rather on comparing drivers during the FMT feedback and no-feedback conditions. Each driver underwent the conditions in the same order: 2 weeks of no feedback (control condition) occurred first, followed by 2 weeks of feedback (intervention condition). Condition order was not counterbalancing because providing the no-feedback condition after the feedback condition would have involved a change in driver behavior carried over from the feedback condition. In contrast, by providing the no-feedback condition first, drivers engaged in normal driving practices for 2 weeks, although driving performance, drowsiness, and sleep need were still recorded by the relevant FMT technologies (i.e., FMT devices were recording but not providing feedback). The no-feedback condition therefore served as a baseline against which the FMT feedback intervention was compared.

Volunteer Drivers

A total of $n = 39$ drivers volunteered for the study ($n = 27$ from Canada, $n = 12$ from the United States). One driver dropped out after being empanelled, which reduced the Canadian sample to $n = 26$ (20 males, six females) and the total sample to $n = 38$. Demographic characteristics of the volunteers as they pertain to truck driving experience are shown in Table 1. More drivers were empanelled than the target sample size of $n = 24$ because of the need to compensate for the loss of data caused by equipment failure. Equipment failure during the 4-week data acquisition study reduced specific comparisons between feedback and no-feedback conditions on some variables to sample sizes ranging between $n = 15$ and $n = 25$ drivers in the Canadian study phase and between $n = 7$ and $n = 12$ drivers in the U.S. study phase. Therefore, when study phases are combined, the hypothesis-testing sample size ranged between $n = 22$ and $n = 38$, depending on the variable being analyzed. As shown in Table 1, most participating drivers were middle-aged males with many years of long-haul driving experience. Drivers were solicited for participation after the protocol, procedures, and informed consents were reviewed and approved by the Canadian Research Ethics Board and by the WRAIR institutional review board.

TABLE 1 Characteristics of Participating Truck Drivers

Country	*n*	Sex	Age Mean (yr)	Age Range (yr)	Years at Company (mean)	Years at Company (range)	Years Driving Large Trucks (mean)	Years Driving Long Haul (mean)	Miles Driven Past Year (mean)
Canada	20	M	45.4	22–58	4.6	< 0.5–17	16.6	11.3	> 109K*
Canada	6	F	35.3	22–50	4.0	< 0.5–15	2.1	1.6	> 76K
U.S.	12	M	46.9	32–57	11.5	6.5–18	23.7	18.0	> 99K
All drivers	38	84% male	44.2	22–58	6.7	< 0.5–18	16.6	11.9	> 100K

*Based on $n = 18$ (data missing from 2 male drivers)

Data Quality Control

Given the extraordinarily large volume of data gathered in the study, it was necessary to determine data management and variable extraction procedures that would ensure quality control of the data. Of particular concern was the need to use procedures that avoided including erroneous data values, especially data corrupted by equipment failure in the field. [Although all the equipment accompanied drivers during 4 weeks of work, no investigator or study technicians were present while drivers were on the road, and hence no one was present to prevent data loss or corruption from equipment damage due to environmental conditions (vibration, heat, cold, rain, snow, ice) in which it was deployed.] Data were carefully segregated into three broad categories: (*a*) all AP+ data with no records excluded, (*b*) AP+ data records in which speed was at least 30 mph, and (*c*) AP+ data for speed ≥30 mph, artifacts eliminated, and records within measurement range. Thus, final cleaned analysis samples from Canada and the United States were defined on the basis of the subset of drivers with sufficient data under both conditions (feedback and no feedback), restricting attention to records recorded at speeds of at least 30 mph, after excluding additional data found to be invalid, following careful examination of driver-specific distributions.

Study Phase 1 took place under Canadian HOS and involved a Canadian trucking company. Volunteer drivers operated single tractor-trailer units with sleeper berths, and approximately 26% of their driving was conducted during nighttime hours (74% in daylight hours). Study Phase 2 took place under U.S. HOS and involved a U.S. trucking company. Volunteer drivers operated tandem tractor-trailer units without sleeper berths, and approximately 93% of their driving was conducted during nighttime hours (7% in daylight hours). The differences between the Canadian and U.S. trucking companies were in part a function of which companies agreed to be part of the study as well as the study goal to expressly study companies for which night driving was both a minority (Study Phase 1) and a majority (Study Phase 2) of trucking operations. For these reasons, the Canada study phase and the U.S. study phase were analyzed separately for the effects of FMT feedback on driving and alertness outcomes before being combined.

Statistical Methods

For each outcome variable recorded by the AP+ system, four analyses were performed to assess if there was a significant change from the no-feedback condition to the feedback condition within Study Phase 1 in Canada and again within Study Phase 2 in the United States. The first of the statistical methods was unweighted analysis for means and standard deviations values across all records for a specific driver under a specific condition (no feedback and feedback). Mean values were compared for the following outcome variables: CoPilot measures of PERCLOS during night hours and SafeTRAC alertness score. Standard deviations were compared for lateral distance, steering wheel movements, and front wheel movements. Then within-driver change scores were computed between no-feedback and feedback conditions. Paired *t*-tests were performed to assess the statistical significance of the changes in means or standard deviations as appropriate.

The second statistical method introduced two weighting factors. First, when the within driver and condition mean, median, standard deviation, and interquartile range values were computed, records were replicated if they corresponded to more than 1 s in duration. In this way, records with durations that were 3 s contributed a weight three times greater than records with durations of 1 s. Even accounting for record duration, drivers varied greatly for total duration of data in the cleaned analysis sample. Drivers with greater total durations under both conditions contributed more information about intervention effects. In contrast, a driver with a short duration under one of the conditions contributes less information about within-driver changes. To account for this, and to optimize the ability to consider both within-subjects and between-subjects sources of variance, mixed model analyses of variance were used to compare mean (duration weighted) values between the no-feedback and feedback conditions, weighting by the total number of available records (separately by condition). All mixed model analyses were implemented by using the Proc. Mixed procedure available in SAS.

The analyses were repeated to summarize the no-feedback and feedback distributions of CoPilot PERCLOS during night hours and SafeTRAC alertness score by median values rather than mean values, to provide summaries of the center of these distributions that are less sensitive to outliers and skewness. Similarly, AP+ lateral distance, AP+ steering wheel movements, and AP+ front wheel movements were summarized by using interquartile ranges (IQR) instead of standard deviations. The IQR is defined as the difference between the 75th percentile value and the 25th percentile value and is less influenced by extreme values than the standard deviation. Both the paired *t*-test and the mixed model weighted analyses were performed on the median and the interquartile range for each variable (which are the nonparametric alternatives to the mean and standard deviation).

Mixed model analysis of variance was used to assess the significance of the intervention effect (no feedback versus feedback), controlling for time-of-day category (day, evening, night). The initial model included fixed effects for time of day (morning, evening, night), presence versus absence of feedback, and time of day by feedback interaction. It also included a random effect for driver to account for correlations within driver. The interaction model (i.e., feedback condition, time of day, time of day by feedback condition) was used to compute an adjusted intraclass correlation (ICC). The ICC is the proportion of total variance explained by systematic differences among drivers after accounting for time-of-day and feedback condition effects. The model used to determine the ICCs was used to examine whether differences between responses obtained during the no-feedback and feedback conditions varied by time of day. A p-value of 0.10 was used because of the low power inherent in tests for interaction. If $p \geq 0.10$, then the interaction terms were removed from the model and the feedback effects and time-of-day effects were tested as main effects in the ANOVA model. If $p < 0.10$, it was concluded that differences between the no-feedback and feedback conditions significantly varied by time of day. Therefore, separate mixed models were used to test for feedback effects at each time-of-day interval (day, evening, night). Daily mean values were analyzed for variables derived from Sleep-Watch. Mixed model analyses of variance were used to assess the significance of the fixed intervention effect. Random effects included between- and within-driver variance, which were used to compute ICCs. Descriptive statistics were used to analyze the drivers' daily diary and postexperimental responses to the human factors structured interview questionnaire.

RESULTS

Data from the FMT devices and other driving performance variables gathered on the AP+ black box recorder every second the trucks were operating for the 28 days each driver was in the study resulted in 8,737,705 total records among the 38 drivers in the combined study phases, which reduced to 6,683,855 data records among 29 drivers

(Canada, $n = 20$; United States, $n = 9$), when data analyses were confined to artifact-free records in which speed was at least 30 mph (i.e., highway driving). Equipment failure resulted in a loss of approximately 25% of the data. Even with this attrition, the data set and remaining sample sizes were adequate for hypothesis testing. Although rough road conditions in the operating trucks caused some data loss, the final data set was among the most extensive on truck driver alertness and truck performance ever recorded. In addition, data acquired from the drivers' daily diaries, their 933 PVT performance tests, their 1.2 million minutes of SleepWatch actigraphic data, and their extensive responses and comments to the human factors structured interview questionnaire resulted in millions of additional data records. Many of the latter variables could be analyzed by using all 38 drivers who completed the study. Key findings are summarized here relative to the primary hypotheses and to other key findings and recommendations relevant to fatigue management in long-haul trucking.

Hypothesis I: FMT Feedback Will Improve Driver Alertness or Reduce Driver Drowsiness or Both

Phase I: Canadian Drivers

There was marginal evidence to support the hypothesis that FMT feedback will improve driver alertness or reduce driver drowsiness. Drowsiness as measured by the CoPilot index of PERCLOS during night hours was modestly lower under the feedback condition compared to the no-feedback condition ($p = 0.094$). Drivers' subjective sleepiness ratings taken before and after PVT performance tests at night also indicated they were less sleepy ($p = 0.009$), although Canadian drivers spent only a minority of time in night driving. However, the SafeTRAC index of driver alertness and drivers' PVT performance lapses during daytime trials showed effects opposite those found for nighttime driving. There was a slight reduction in SafeTRAC alertness during the daytime in the feedback condition relative to the no-feedback condition among Canadian drivers ($p = 0.013$) and an elevation of PVT lapses ($p = 0.0004$). Hence there was no consistent finding in support of Hypothesis I in the Phase 1 data.

Phase 2: U.S. Drivers

There was evidence in support of Hypothesis I in the Phase 2 data. This phase focused more extensively on drivers who primarily drove at night (73% of the time), when sleepiness would be expected to be more of a problem. There was clear evidence of greater alertness in the feedback condition during night driving than in the no-feedback condition at night from both the SafeTRAC index of driver alertness ($t = 2.67$, $df = 8$, $p = 0.028$) and the CoPilot index of PERCLOS ($t = 2.70$, $df = 8$, $p = 0.027$). Although only a statistical trend, lane tracking variability also improved with feedback during night driving in the U.S. study phase ($p = 0.083$).

Combined Canadian and U.S. Data

Composite results from pooling data from the two study phases yielded strong support for Hypothesis I. During night driving, feedback from fatigue management technologies significantly reduced slow eyelid closures (PERCLOS) as measured by CoPilot ($t = -3.24$, $n = 25$, $p = 0.004$), increased the SafeTRAC estimate of driver alertness ($t = 3.49$, $n = 24$, $p = 0.002$), and decreased lane tracking variability ($t = -2.96$, $n = 24$, $p = 0.007$).

Hypothesis II: FMT Feedback Will Increase Driver Sleep Time

Phase 1: Canadian Drivers

Within the Canada study phase, none of the SleepWatch actigraphy outcomes demonstrated systematic differences between the no-feedback and feedback conditions. There was also no evidence from drivers' daily diaries to support the hypothesis that FMT feedback resulted in increased sleep time relative to no feedback.

Phase 2: U.S. Drivers

Within the U.S. study phase, there was a significant increase in the number of SleepWatch actigraphically identified sleep episodes but not sleep duration in the feedback condition relative to the no feedback. There was also no evidence from drivers' daily diaries of increased sleep time.

Combined Canadian and U.S. Data

There was no support for Hypothesis II when SleepWatch data were combined between study phases.

Sleep on Workdays Versus Nonworkdays

Not surprisingly, drivers in both countries slept significantly more on nonworkdays than on workdays. During the no-feedback 2-week period of the Canadian study phase, drivers averaged 7 h 17 min of sleep per 24-h period on nonworkdays compared to 6 h 15 min on workdays, a mean difference of 1 h 2 min ($p = 0.023$). Similarly, during the feedback 2-week period of the Canadian phase, drivers averaged 7 h 31 min of sleep per 24 h on nonworkdays compared to 6 h 14 min on workdays, a mean difference of 1 h and 17 min ($p = 0.0005$). Comparable results were obtained in the U.S. study phase. During the no-feedback 2-week period, U.S. drivers averaged 6 h 32 min of sleep per 24 h on nonworkdays compared to 5 h 14 min on workdays, a mean difference of 1 h 18 min ($p = 0.018$). Similarly, during the feedback period, U.S. drivers averaged 7 h 32 min sleep compared to 5 h 1 min on workdays, a mean difference of 2 h 31 min ($p = 0.0004$). These are relatively large differences in 24-h sleep durations, suggesting that drivers developed sleep debts across the work week.

Effect of FMT Feedback on Nonworkdays Sleep

Although mean sleep duration was significantly less for U.S. drivers compared to Canadian drivers ($F_{1,28} = 7.50$, $p = 0.011$), when SleepWatch actigraphically identified sleep duration per 24 h was analyzed for both study phases, separating workdays and nonworkdays, there was clear evidence in support of Hypothesis II. In contrast to workdays, for which FMT feedback had no effect on sleep time, there was a significant increase in mean sleep duration during nonworkdays in the feedback condition relative to no feedback in both the Canadian drivers ($t = -2.55$, $df = 15$, $p = 0.023$) and the U.S. drivers ($t = -2.88$, $df = 10$, $p = 0.018$). Drivers in both study phases increased their nonworkday sleep durations by an average of 45 min per day over sleep duration on nonworkdays in the no-feedback condition ($F_{1,25} = 4.39$, $p = 0.046$).

Other Key Findings

Cost for Being More Alert with FMT Feedback?

As summarized, during FMT feedback, alertness improved significantly during driving in the U.S. study phase, which involved driving at night 93% of the time. However, there was also consistent evidence that PVT performance worsened and subjective sleepiness ratings increased during the feedback period of the U.S. study relative to the no-feedback period. U.S. drivers' nighttime PVT performance lapses per trial during the no-feedback and feedback conditions averaged 3.12 and 4.59, respectively ($t = 2.83$, df = 11, $p = 0.016$). Similar findings were obtained during daytime driving periods in the Canada study phase, when 74% of driving occurred. During daytime PVT test trials, the mean number of lapses per trial during the no-feedback and feedback conditions was 1.95 and 3.89, respectively ($t = 4.49$, df = 16, $p = 0.0004$). The feedback condition was also associated with slower median PVT reaction times during night driving in the U.S. phase ($t = 5.14$, df = 11, $p < 0.0001$) and during day driving in the Canada phase ($t = 3.54$, df = 16, $p = 0.003$). Drivers' ratings of their sleepiness on a post-PVT visual analogue scale also revealed greater sleepiness in the feedback condition than in the no-feedback condition during nighttime PVT tests of the U.S. study phase (3.29 versus 5.33; $t = 6.63$, df = 11, $p < 0.0001$). These findings suggest that FMT feedback in drivers who operate primarily at night may have alertness-promoting benefits during driving, but such feedback may also create a modest cost for the added effort (in attention and compensatory behaviors) required to respond to the information from the devices, and cost may manifest as slightly worse performance and greater subjective sleepiness when drivers perform a demanding vigilance-based reaction time task such as the PVT (while not driving).

Do Drivers Prefer Vehicle-Based Measures of Alertness?

In general, drivers agreed that commercial drivers would benefit from fatigue management aids (Canada, 88%; United States, 100%). Descriptive analyses of driver responses to the human factors structured interview questionnaire at the end of the 2-week no-feedback period, and again at the end of the 2-week feedback condition period, revealed clear preferences of both Canadian and U.S. drivers for fatigue management training and certain fatigue management technologies. Drivers were uniformly positive about the education on alertness and fatigue management course given at the beginning of each study phase. Among technologies designed to detect alertness or drowsiness, drivers gave higher ratings to SafeTRAC, medium ratings to the SleepWatch, and low ratings to the CoPilot. Among all FMT technologies deployed, however, drivers were significantly more enthusiastic about the benefits of the HPCS system and SafeTRAC than they were about SleepWatch and CoPilot. It is noteworthy that HPCS and SafeTRAC both interface with the vehicle, whereas SleepWatch and CoPilot interface with the driver. It may be that truck drivers prefer fatigue management to be through vehicle monitoring rather than through driver monitoring. More research is needed to understand what influences commercial drivers' attitudes toward feedback by technology (*19*).

Future for FMT Technologies

Overall, participant drivers were positive toward the FMT approach in general and thought that if such technologies could be further improved, they would help manage fatigue and alertness.

RECOMMENDATIONS FOR FUTURE WORK OUTSIDE SCOPE OF PROJECT

Continue Development of FMT Technologies

There is enough evidence to support the case for continued development of FMT technologies. However, these should not be used only for driver monitors. Vehicle-based monitoring should also get increased attention, as truck drivers appear to have some preference for this mode of fatigue management.

Drivers Want Alertness and Fatigue Management Courses

Despite differences in country of operation, hours of service, type of trucks, and many other factors, U.S. and Canadian drivers had surprisingly similar views toward the FMT project. They were positive toward the alertness and fatigue management training course provided in the study. Postexperimentally, drivers rated the course content and knowledge gained as "good" to "very helpful" (highest rating); 83% to 96% indicated the course lessons were used by them during the FMT study and that they intended to continue to use them. Qualitative comments from drivers indicated they perceived benefit from the course and would like to have more of this type of didactic to help teach them how to manage fatigue. This is impressive given that these were largely seasoned long-haul drivers who appeared not to be inhibited about reporting that they can still learn about fatigue and ways to manage it. These positive views toward fatigue management training suggest that some segments of the trucking industry are likely to welcome fatigue management programs.

PVT Should Be Developed as a Fitness-for-Duty Test

Although PVT was not discussed with drivers as either an FMT technology or a fitness-for-duty test, a majority of drivers in both countries indicated when asked that the PVT could be used as a personal checking system on a driver fitness-for-duty system, if it could be reduced in duration. Drivers' generally positive view of the PVT as a potential fitness for-duty device suggests that efforts should be made to attempt to validate the sensitivity of and positive and negative predictability of a shorter-duration PVT test (e.g., 3 to 5 min) relative to truck driver fatigue.

Barriers to Drivers Obtaining Adequate Sleep During Workdays Must Be Identified

One of the more striking outcomes was the finding that drivers in both countries were routinely averaging between 5 h and 6.25 h of sleep per day during workdays, despite very different work schedules. Recent scientific work, some of it by USDOT on volunteer truck drivers, shows that severe sleep debt and deficits in behavioral alertness can develop within a few days at these sleep durations. That project participants markedly increased sleep durations on nonworkdays also supports the view that they were suffering sleep debts. Much more must be understood about the factors that determine when and where drivers obtain sleep on workdays and nonworkdays, the barriers to obtaining adequate sleep on workdays, and the factors that convince drivers to get more recovery sleep on nonworkdays.

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Motorcycle Helmet Use and Trends Before and After Florida's Helmet Law Change in 2000

Patricia A. Turner and Christopher Hagelin

The Center for Urban Transportation Research at the University of South Florida conducted this study for the Florida Department of Transportation to analyze motorcycling trends in Florida before and after the July 2000 change to the motorcycle helmet law. The change permits motorcyclists 21 years of age and older to ride without a helmet if they carry at least $10,000 in insurance to cover medical costs incurred as a result of a crash. This paper discusses study findings on motorcycle trends before and after the Florida change related to observed and reported motorcycle helmet use, number and severity of motorcycle crashes, and number and severity of injuries sustained in motorcycle crashes. Additionally, national and Florida data related to vehicle miles of travel (VMT), registrations, crashes, injuries, fatalities, and helmet use are presented, and recommendations for future motorcycle research are made. Findings show that Florida's observed helmet use rate declined from 99.5% in 1998 to 52.7% in 2002. Sport bike riders were among those most likely to be helmeted, whereas lack of helmet use typically was associated with riders on cruiser-style motorcycles. Declines in observed helmet use rates in Florida are comparable to declines in other states with recently amended universal helmet laws. Helmet use among crash-involved motorcycle operators continues to decline even among younger riders required by law to wear helmets. Crash rates and injury rates per registered motorcycle and per motorcycle VMT declined following the helmet law change, with the exception of fatal crash rates.

Florida is above the national averages in the proportion of motorcyclists killed in traffic crashes compared to all traffic fatalities and in the fatality rate per 10,000 registered motorcycles. Florida trends show an overall decline in the proportion of motorcycle fatalities compared to all traffic fatalities from 1993 through 1999 until trends began to reverse (see Figure 1). In 2001, the proportion of motorcycle fatalities compared to all traffic fatalities reached an all-time high in Florida, 9.2% compared to 7.6% nationally. Similarly, the fatality rate per 10,000 registered motorcycles declined in Florida from 10.6 in 1993 to 7.0 in 1999 and then increased to 9.6 in 2000, followed by a decrease to 9.0 in 2001.

A significant change that occurred during this time was an amendment to the Florida motorcycle helmet law. In 2000, Florida became one of six states (along with Arkansas, Kentucky, Louisiana, Texas, and Pennsylvania) that repealed or amended motorcycle helmet use laws since 1997. On July 1, 2000, Florida motorcyclists 21 and older could ride without a helmet if they carried at least $10,000 in medical insurance to cover injury costs as a result of a crash.

Increased exposure, measured in motorcycle registrations and vehicle miles traveled (VMT), may in part explain the increase in motorcycle fatalities since 1997. NHTSA cites newer bikes with larger engines, increasing numbers of older motorcyclists, speeding, impaired riding, improper licensing, lack of training, and a decline in helmet use as other contributing factors to rising deaths among motorcyclists (*1*).

In 2001, the Florida Department of Transportation (FDOT) safety office contracted with the Center for Urban Transportation Research (CUTR) at the University of South Florida to conduct a third statewide motorcycle helmet use observational survey to determine motorcycle helmet use rates on Florida roadways. Because motorcycle fatalities have been on the rise in recent years in Florida, the research scope was expanded to include an examination of motorcycle crash, injury, and fatality trends and factors that may contribute to increasing motorcycle fatalities. This study builds on previous research conducted by CUTR for FDOT on observational helmet use (*2, 3*), motorcycle alcohol-related crashes (*4*), and general motorcycle safety (*5*).

RESEARCH OBJECTIVES

The study objectives were as follows:

- Conduct a statewide motorcycle helmet use observational survey in Florida and compare findings with previous helmet use survey results.
- Compile national and Florida data related to motorcycle VMT, registrations, crashes, injuries, fatalities, and helmet use to examine emerging trends.
- Compile motorcycle crash and injury data and examine trends 18 months before and 18 months after the Florida motorcycle helmet law change.

The research was not specifically designed to determine the effect of the helmet law change on motorcyclist injuries and fatalities, because to do so would require statistically controlling for the influences of other risk factors that contribute to motorcycle crashes and deaths, such as speed, alcohol, training, licensing, weather, and enforcement. Nonetheless, the study findings provide a comprehensive examination of motorcycling trends in Florida and should be of interest to state departments of transportation, state motorcycle education programs, motorcycle safety advocates, public health organizations, law enforcement agencies, the motorcycling community, the general motoring public, and other groups interested in motorcycling.

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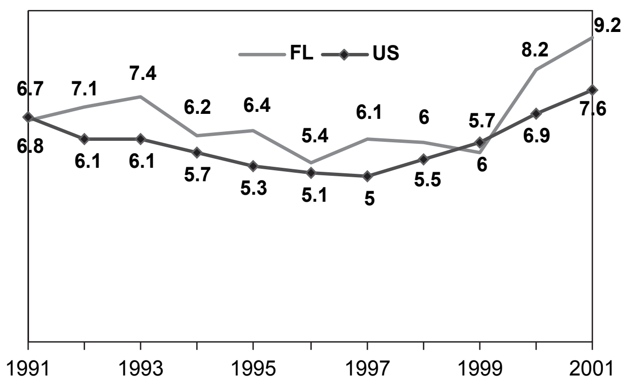

FIGURE 1 Motorcyclist fatalities as proportion of all motor vehicle fatalities, United States versus Florida, 1991–2001 (SOURCE: DHSMV, NHTSA).

DATA SOURCES

The following data sources were used in the study:

- Fatality Analysis Reporting System (FARS), for fatal and injury crashes;
- FHWA, for motorcycle registrations and VMT;
- Florida Department of Highway Safety and Motor Vehicles (DHSMV), for motorcycle crashes, injuries, and fatalities; and
- Florida helmet use observational survey data, for observed helmet use trends.

RESEARCH METHODOLOGY

Observational Helmet Survey

NHTSA guidelines for state observational surveys of safety belt and motorcycle helmet use were used to develop a sampling plan to ensure a statistically valid sample and to ensure that collected data complied with the 5% relative error precision requirement (*6*). The sampling plan was developed to determine which counties to survey, the number of observational sites in each county, the specific location of these sites, and the days and times for data collection. The final multistage stratified sampling design, approved by NHTSA, used stratificatio and clustering. Stratification was used to increase the precision of sample estimates for a given sample size according to population, number of registered motorcycles, daily VMT (DVMT), and functional classification of roadways, and clustering was done to achieve cost-effectiveness and efficiency by grouping together sites within designated timeframes. [DVMT is the product of the length of a road (centerline miles) and the annual average daily traffic. For example, if a road is 10 mi long and 2,500 vehicles travel it each (average) day, the DVMT is 25,000 (*7*).]

In May and June 2002, data collectors observed motorcyclists at 486 observation sites in 13 Florida counties and recorded at least one observation during the 1-h observation period at 91% of the sampled sites. Five of the 13 counties surveyed were double sampled. In the 2-month period, a total of 3,491 motorcyclists were observed. Among those observed, 3,002 were motorcycle operators and 489 were passengers. The majority of bike types observed were cruiser-style motorcycles (50.4%), followed by an equal percentage of sport and touring bikes (18.5% and 18.4%, respectively).

A combination of descriptive and inferential statistics was used to analyze the survey data and interpret relationships between categorical variables of interest. Statistical relationships between helmet use and motorcycle type, gender, and occupant type were explored, and the Pearson chi-squared test (χ^2) for independence was used to determine whether the differences between observed and expected frequencies were statistically significant. These results were compared with previous helmet use survey findings conducted in 1993 and 1998.

Motorcycle Trend Analysis

Researchers compiled data from FARS, FHWA, and the Florida DHSMV to determine U.S. and Florida trends related to motorcycle registrations, VMT, crashes, injuries, fatalities, and helmet use.

Researchers also obtained data on all police-reported motorcycle crashes that occurred between January 1, 1999 (18 months before the motorcycle helmet law change), and December 31, 2001 (18 months after implementation of the motorcycle helmet law change), as well as monthly frequency distributions for all crashes, injury crashes, and fatal crashes. The data were grouped into two categories: before the helmet law change, consisting of crashes that occurred between January 1, 1999, and June 30, 2000, and after the helmet law change, consisting of crashes that occurred between July 1, 2000 (the date that the amendment change took effect), and December 31, 2001.

Researchers also conducted the following trend analyses:

- Comparison of U.S. and Florida data on motorcycle registrations from 1991 to 2001;
- Comparison of U.S. and Florida data on motorcycle VMT from 1991 to 2001;
- Comparison of U.S. and Florida crash, fatality, and injury trends and rates per registered motorcycles and per motorcycle VMT from 1991 to 2001;
- Comparison of U.S. and Florida operator helmet use trends in fatal crashes from 1991 to 2001; and
- Florida helmet use and age data from 1992 to 2001.

Data from FHWA's highway statistics series, which provides total VMT for each state by functional road class and an estimation of the percent of vehicle types by functional roadway classification, were used to determine Florida's motorcycle VMT. The VMT was estimated by multiplying the percent of motorcycles by the total VMT for each functional class and summing to determine statewide motorcycle VMT estimates. Observations by vehicle type are not collected for all functional classes in Florida, in particular, collectors and local roads. Therefore, the total motorcycle VMT figures used represent a sample of actual motorcycle VMT per year.

Florida registration data were obtained from an FHWA highway statistics report. Registration and motorcycle VMT data were used to calculate crash and injury rates per registered motorcycles and per VMT to look at changes before and after the motorcycle helmet law change. Because monthly breakdowns of motorcycle registration and VMT data were not available, researchers assumed that the data were equally divided between the first and second halves of the 2000 calendar year.

RESULTS

Observed Helmet Use

Significant reductions in observed helmet use occurred since the 1998 observational survey. In 2002, helmet use was observed at 52.7%, down from the 1998 observed helmet use rate of 99.5%. Corresponding with the drop in observed helmet use was an 86% decline in observed novelty helmet use—from 40.2% in 1998 to 5.7% in 2002 (see Figure 2).

A comparison of the 2002 results with the 1998 data reveals some interesting trends regarding helmet use choice among occupants riding different types of motorcycles. In 1998, 70% of occupants observed on cruiser-style bikes wore novelty helmets, compared to 7.8% in 2002. Although a one-to-one correlation between prior novelty helmet use and no helmet use after the law change is not possible, results suggest that the majority of riders who previously wore novelty helmets chose to ride helmetless after the law change.

Helmet use is significantly related to motorcycle type, gender, and occupant type. Sport bike riders were among those most likely to be helmeted (79.7%), whereas riders on cruiser-style motorcycles were observed helmeted only 28.6% of the time. Female motorcyclists were more likely to wear some type of protective headgear compared to male riders (56.2% versus 50.3%). However, helmeted females were twice as likely as males to be observed wearing novelty helmets (10.1% versus 4.9%). An analysis of passenger data found that passengers were more likely than operators to be helmeted (54.9% compared to 51.3%) and twice as likely to be wearing novelty helmets (10% versus 5.2%). Further, female passengers were more likely than male passengers to wear protective headgear (56.5% versus 46.3%) and to wear novelty helmets (9.5% versus 2.2%).

Declines in observed helmet use rates in Florida are comparable to those in other states with recently amended universal helmet laws. Florida's observed helmet use of 52.7% is lower than that of Texas and similar to observed helmet use rates in Arkansas and Louisiana following helmet law changes. In Texas and Arkansas, helmet use approached 97% before the law change and declined to 52% in Arkansas and 67% in Texas the year after the law was changed. Louisiana had 100% helmet use before the helmet law change, but the use rate fell by 48 percentage points during observations conducted in 2001.

Helmet use rates in these states continued to decline in subsequent years after helmet law changes (observed helmet use was 53.2% in 2002), and if Florida follows similar trends, use rates could substantially decline.

Motorcycle Trends

Increased exposure, measured in motorcycle registrations and VMT, may in part explain increasing motorcycle death rates in Florida. Motorcycle registrations in Florida grew faster than the national average, 29.6% from 1999 to 2001 compared to 18% nationally. Sales trends suggest registrations will likely continue to increase as Florida's rate of growth for new-unit on-highway motorcycle sales has surpassed the national growth rate since 2000.

Florida's motorcycle VMT increased by 40% from 361 million mi in 1999 to 505 million mi in 2001, in contrast to a 10% reduction in total U.S. annual motorcycle VMT, from 10.6 billion mi in 1999 to 9.5 billion mi in 2001.

Motorcycle crashes have continued to increase in Florida since 1999. Motorcycle crashes in Florida increased by 29.3%, from 4,451 in 1999 to 5,766 in 2001. Further, the proportion of motorcycle crashes to all traffic crashes in Florida reached a high of 2.3% in 2001, the highest percentage since 1994.

Deaths attributable to motorcycle crashes are becoming a larger portion of the overall traffic crash problem in Florida. Florida's motorcycle crash–related fatalities have been steadily increasing since 1999. Between 1999 and 2001, motorcycle fatalities have increased by 66% in Florida, compared to a 28% increase in national motorcycle fatalities during the same period. In 2001, the proportion of motorcycle fatalities compared to all traffic fatalities reached an all-time high in Florida, 9.2% compared with 7.6% nationally.

Florida's fatality rate per registered vehicles and per VMT is higher than national averages. In 2001, Florida's fatality rate per 10,000 registered motorcycles was 9.0, compared to the national rate of 6.5, and the motorcycle fatality rate per 100 million VMT was 1.6 times the national rate (54.7 versus 33.4). Florida data indicate a decline in the rate from 2000 to 2001, 9.6 to 9.0. Variations in the fatality rate per VMT in Florida follow the same trends as the rate per registered motorcycles.

Helmet Use in Crashes

Helmet use rates among fatally injured motorcycle operators declined significantly following the helmet law change. FARS data show that

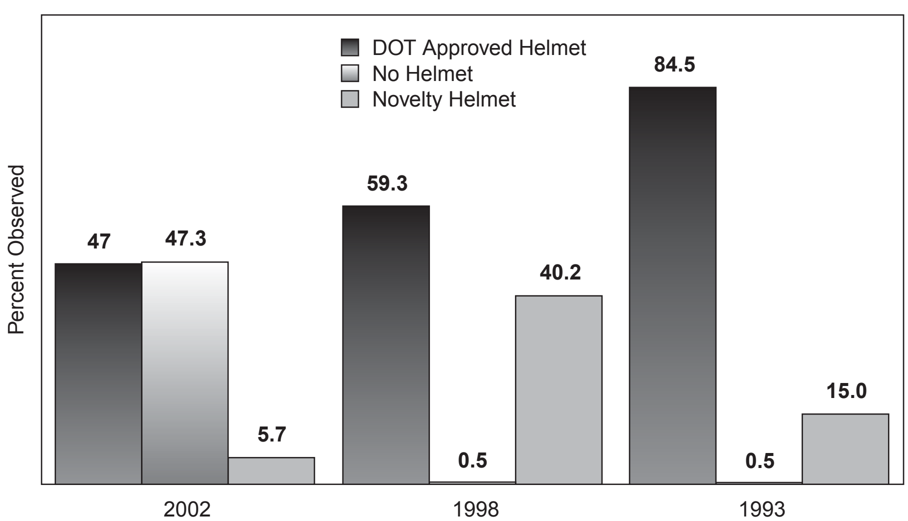

FIGURE 2 Observed motorcycle helmet use, Florida, 2002, 1998, and 1993 (SOURCE: CUTR).

whereas Florida operator helmet use in fatal crashes fluctuated between 82% and 89% between 1991 and 1999, helmet use rates in fatal crashes fell to 71% in 2000 and 45% in 2001 (nine points below the national average).

Helmet use among crash-involved motorcycle operators continues to decline even among younger riders required by law to wear helmets. In all crashes, a breakdown by age reveals that just over one-half (53.2%) of all crash-involved operators younger than 21 were helmeted, compared to 47% of all crash involved motorcycle operators 21 and older in 2001, according to DHSMV police-reported motor vehicle crash data.

Motorcycle Trends 18 Months Before and After Florida Law Change

Motorcycle registrations and VMT continue to increase. Motorcycle registrations and VMT increased from the 18-month period before the helmet law change to the 18-month period following the helmet law change, by 19.2% and 26%, from 363,321 to 433,066, and 553 million to 697 million, respectively.

The number of motorcycle crashes (including injury and fatal crashes) as well as the number of injuries and fatalities are increasing. Total motorcycle crashes, as well as the number of injury and fatal crashes, increased from the 18-month period before the helmet law change to the 18-month period following the helmet law change, and the largest percent increase was seen in fatal crashes (43.8%).

The Florida DHSMV classifies injuries sustained in crashes into fiv categories: Level 1: no injuries; Level 2: possible injuries; Level 3: nonincapacitating injuries; Level 4: incapacitating; and fatal. All injuries (Levels 2 through 4) increased by 15.6% from 7,082 in the 18-month period before the helmet law change to 8,190 in the 18 months after the law change. Fatalities increased by 42.3%, from 284 in the 18-month period before the helmet law change to 404 in the 18 months after the law change.

Overall crash rates and injury crash rates per registered motorcycle and per motorcycle VMT are on the decline, with the exception of fatal crash rates. Crash rates per 10,000 registered motorcycles and per 100 million motorcycle VMT declined from the 18-month period before the helmet law change to the 18-month period following the helmet law change, by 2.6% and 7.8% from 195 to 190 and 1,279 to 1,179, respectively.

Injury crash rates per 10,000 registered motorcycles and 100 million motorcycle VMT from the 18-month period before the helmet law change to the 18-month period following the helmet law change declined in every category except fatal crashes per 10,000 registered motorcycles and per 100 million motorcycle VMT, which increased by 20.8% and 13.8%, from 7.7 to 9.3 and 51 to 58, respectively.

The proportion of younger riders (under 21) killed in motorcycle crashes is increasing in Florida. Although most motorcyclists killed between January 1, 1999, and December 31, 2001, were riders age 21 years and older, the proportion of younger riders (under 21) killed increased from 7% in the 18-month period before the helmet law change to 11% in the 18-month period after the helmet law change.

RECOMMENDATIONS

Although this study presented an examination of recent motorcycling trends in Florida, many unanswered questions remain about the underlying causes for rising fatalities and death rates among Florida's motorcyclists. More motorcyclists are on the road riding greater distances, and if the issue is not addressed, the number of motorcycle crashes and deaths will likely continue to rise. Therefore, future action must focus on understanding why there are more motorcycle crashes and deaths and what can be done to improve the safety of motorcyclists on Florida's roadways.

Recommendation 1: Monitor Motorcycle Helmet Use on Florida Roadways

The protective effect of motorcycle helmets is clearly documented in the research; helmets can decrease the severity of injuries and reduce the likelihood of death and overall cost of medical care associated with motorcycle crashes. Helmets are effective, however, only if motorcyclists choose to wear them. The good news is that many riders continue to wear helmets, even in states with limited helmet use laws or no helmet laws. Because helmet use rates have continued to decline in years following helmet law changes in many of the states, Florida should continue to monitor helmet use rates periodically.

Recommendation 2: Control for All Variables That Contribute to Motorcycle Crash Involvement and Rising Fatality Rates

Although the increase in nonhelmeted riders may, in part, contribute to rising fatality rates among Florida's motorcyclists, changes in numerous other variables not controlled for in the study may also affect motorcycle crash involvement and fatality rates. Such factors, as noted in previous research, include higher population densities, weather, alcohol, speed, rider training, changing travel environments such as roadway type, urban versus rural travel, roadway traffic volumes, and available police resources. Whether a causal relationship exists between increased fatality rates and the change in the Florida helmet law and subsequent decline in helmet use cannot be determined from this analysis. Future research should include studies designed to control for these variables to better understand causal factors for rising rider death rates, including the existence of a universal helmet law.

Recommendation 3: Monitor National Studies of Motorcycle Crash Causation and Effective Countermeasures

Further information is needed to understand the causes of motorcycle crashes and other contributing factors to motorcycle crashes. As identified in the National Agenda for Motorcycle Safety, there is growing need and support for national studies that focus on comprehensive, in-depth motorcycle crash studies to identify ways to prevent crashes (*8*). NHTSA is considering an update to its previous motor vehicle crash causation study (*9*), in which motorcycle crash causation data would be collected and analyzed to help investigate reasons for the substantial increase in motorcycle crashes. The American Motorcyclist Association is working to have $2 million included in the Transportation Equity Act for the 21st Century for an in-depth motorcycle crash study that would provide detailed at-the-scene investigation of at least 1,000 crashes to determine causal factors in motorcycle crashes. Finally, a recent study funded by the Association of European Motorcycle Manufacturers documented the results of a comprehensive in-depth investigation of 921 motorcycle crashes in five European cities (*10*). Studies like these would be of benefit to states, including

Florida, to further understand causal crash factors and identify ways to address rising motorcycle fatality rates.

Recommendation 4: Investigate Ways to Determine More Accurate Motorcycle VMT

Motorcycle VMT is the best exposure measure available; however, nonuniform estimation procedures may result in underestimations of motorcycle travel in some states. Research is needed to establish more accurate and detailed motorcycle VMT and explore alternative measures to determine motorcycle exposure. For instance, more detailed motorcycle VMT could allow for a comprehensive crash involvement analysis based on roadway classification and temporal distribution of motorcycle crashes. Research could examine helmet use versus non-helmet use by VMT to determine differences in crash involvement rates between the two groups as well as differences in crash involvement rates based on roadway type and traffic volumes.

Recommendation 5: Conduct Focus Group Studies to Develop Strategies to Improve Motorcycle Safety

Finally, several questions related to rider behavior should be addressed through future quantitative research. Focus groups could be used to provide greater insight into why motorcyclists choose to ride with or without helmets on the basis of age, gender, type of motorcycle ridden, and occupant type. Survey methods could be used to obtain information about motorcyclists' crash experiences, training, exposure, helmet use, and alcohol use, which would be valuable for developing strategies for outreach and improving motorcycle safety.

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TRANSPORTATION RESEARCH RECORD: JOURNAL OF THE TRANSPORTATION RESEARCH BOARD

Peer Review Process

The ***Transportation Research Record: Journal of the Transportation Research Board*** publishes approximately 30% of the more than 2,500 papers that are peer reviewed each year. The mission of the Transportation Research Board (TRB) is to disseminate research results to the transportation community. The Record series contains applied and theoretical research results as well as papers on research implementation.

The TRB peer review process for the publication of papers allows a minimum of 30 days for initial review and 60 days for rereview, to ensure that only the highest-quality papers are published. At least three reviews must support a committee's recommendation for publication. The process also allows for scholarly discussion of any paper scheduled for publication, along with an author-prepared closure.

The basic elements of the rigorous peer review of papers submitted to TRB for publication are described below.

Paper Submittal: June 1–August 1

Papers may be submitted to TRB at any time. However, most authors use the TRB web-based electronic submission process available between June 1 and August 1, for publication in the following year's Record series.

Initial Review: August 15–October 31

TRB staff assigns each paper by technical content to a committee that administers the peer review. The committee chair assigns at least three knowledgeable reviewers to each paper. The initial review is completed by mid-September.

By October 1, committee chairs make a preliminary recommendation, placing each paper in one of the following categories:

1. Publish as submitted or with minor revisions,
2. Publish pending author changes and rereview, or
3. Reject for publication.

By late October, TRB communicates the results of the initial review to the corresponding author indicated on the paper submission form. Corresponding authors communicate the information to coauthors. Authors of papers in Category 2 (above) must submit a revised version addressing all reviewer comments and must include a cover letter explaining how the concerns have been addressed.

Rereview: November 20–January 25

The committee chair reviews revised papers in Category 1 (above) to ensure that the changes are made and sends the Category 2 revised papers to the initial reviewers for rereview. After rereview, the chairs make the final recommendation on papers in Categories 1 and 2. If the paper has been revised to the committee's satisfaction, the chair will recommend publication. The chair communicates the results of the rereview to the authors.

Discussion: February 1–May 1

After the Annual Meeting, discussions may be submitted for papers that will be published. TRB policy is to publish the paper, the discussion, and the author's closure in the same Record.

Attendees interested in submitting a discussion of any paper presented at the TRB Annual Meeting must notify TRB no later than February 1. If the paper has been recommended for publication, the discussion must be submitted to TRB no later than March 1. A copy of this communication is sent to the author and the committee chair.

The committee chair reviews the discussion for appropriateness and asks the author to prepare a closure to be submitted to TRB by April 1. The committee chair reviews the closure for appropriateness. After the committee chair approves both discussion and closure, the paper, the discussion, and the closure are included for publication together in the same Record.

Final Manuscript Submittal: April 1

In early February, TRB requests a final manuscript for publication—to be submitted by April 1—or informs the author that the paper has not been accepted for publication. All accepted papers are published by December 31.

Paper Awards: April to January

The TRB Executive Committee has authorized annual awards sponsored by Groups in the Technical Activities Division for outstanding published papers:

- Charley V. Wootan Award (Policy and Organization Group);
- Pyke Johnson Award (Planning and Environment Group);
- K. B. Woods Award (Design and Construction Group);
- Patricia F. Waller Award (Safety and System Users Group);
- D. Grant Mickle Award (Operations and Maintenance Group);

and

- John C. Vance Award (Legal Resources Group).

Other Groups also may nominate published papers for any of the awards above. In addition, each Group may present a Fred Burggraf Award to authors 35 years of age or younger.

Peer reviewers are asked to identify papers worthy of award consideration. Each Group reviews all papers nominated for awards and makes a recommendation to TRB by September 1. TRB notifies winners of the awards, which are presented at the following TRB Annual Meeting.

TRB Annual Meeting

The majority of papers are submitted to TRB for both publication and presentation. Chairs of committees that review papers for publication also make recommendations on presentations. After completion of the initial review, in addition to making the preliminary publication recommendations, chairs make presentation recommendations. This ensures high-quality paper sessions at the Annual Meeting. Authors of all papers on the program are asked to submit the revised versions of their papers electronically for a CD-ROM distributed at the Annual Meeting.

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