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J. Falcocchio, Department of Transportation Engineering, Polytechnic Institute of New York
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Development and Application of Traffic-Management Models

Adolf D. May and Abraham J. Kruger, Institute of Transportation Studies, University of California, Berkeley
Thomas J. Clausen, Berkeley City Planning Department

This paper summarizes the development of a freeway model and an arterial model and their application in assessing the impacts of traffic-management strategies. Previously developed models were modified to include energy and air-pollution impacts, and to include spatial and modal demand shifts due to freeway and arterial traffic-management strategies. The new freeway model was applied to a 20.2-km (12.6-mile) inbound section of the Santa Monica Freeway in Los Angeles during the morning peak period. Priority entry-control operations were found to be more effective than normal entry-control operations although an exclusive bus and car-pool lane was more effective than an exclusive bus lane. The new arterial model was applied to an 8-km (5-mile) section of Wilshire Boulevard in Los Angeles for two-way traffic operations during the afternoon peak period. Optimum signal-control strategies under existing street design conditions were found to be more effective than optimum signal-control strategies combined with either reversible lanes or exclusive bus-lane operations. Signal-control strategies under existing street design conditions were determined on a passenger basis and on a vehicle basis; these strategies resulted in a trade-off between passenger-time savings and reduction in air pollution and fuel consumption. Reversible-lane operations were found to be more effective than exclusive bus-lane operations. Future areas of research are identified.

Traffic-management research activities have been conducted at the Institute of Transportation Studies (ITS) at the University of California, Berkeley, for the past decade (1). Early research into macroscopic flow relationships and deterministic queuing analysis led to the development of the freeway simulation model FREQ (2). This model was extended to include mathematical search procedures capable of determining optimum redesign (3) and ramp-control strategies (4). Prior to undertaking the research described in this paper, ITS developed two decision models for freeway-corridor control (5) and priority-entry control (6). The freeway-corridor model combined the earlier developed freeway simulation model FREQ3 with the surface street model TRANSYT5 (7). The freeway priority-entry model FREQ3CP combined FREQ3 with a search procedure capable of determining optimal ramp control on a vehicle basis or a person basis.

During 1975 and 1976, the Traffic Management Group, one of five groups participating in a research project managing the future evaluation of the urban transportation system, modified FREQ3CP and TRANSYT6 to include energy and air-pollution impacts and to include modal and spatial demand shifts due to traffic-management strategies. The two modified models, FREQ4CP and TRANSYT6B, were applied to Santa Monica Freeway and Wilshire Boulevard to assess impacts and demand responses of various traffic-management strategies. Results of this research are documented in two reports that describe development and application of FREQ4CP (8) and TRANSYT6B (9).

FREEWAY MODEL DEVELOPMENT

FREQ3CP

The FREQ3CP (6) freeway model combines a simulation model with a search procedure capable of determining optimal ramp control on a vehicle basis or on a person basis. The input to the model consists of freeway-design parameters, origin-destination (O-D) traffic demand patterns, and linear programming objective and constraint specifications. The output is in three parts: simulation of existing traffic performance without control, optimal ramp-control strategy, and simulation of expected traffic performance with control strategy in effect.

The traffic performance for each subsection in each time segment is calculated and includes flow level, volume and capacity ratio, speed, density, travel time, total passenger-hours, total passenger-kilometers, and queuing characteristics. A directional freeway of 16 to 24 km (10 to 15 miles), including up to 20 on-ramps and 20 off-ramps, can be analyzed during every 10 to 15-min time segment during the peak traffic period. The model is macroscopic and deterministic, is written in ANS FORTRAN, is operated at CDC and IBM computer facilities, has been calibrated against field conditions, and has been applied to several different locations.

Energy and Air-Pollution Impact Extensions

FREQ3CP uses travel time as the primary impact measure. A study of other possible impact effects was undertaken, and energy and air pollution were selected for inclusion in the model. The results of previous energy (10) and air pollution (11) research were adopted, and energy and air-pollution algorithms were added to the existing model.

Three types of vehicles can be handled: passenger vehicle, gasoline-powered truck (or bus), and diesel-powered truck (or bus). For each vehicle type, fuel consumption rates are calculated based on average speed, volume and capacity ratio, and specified roadway design features. The user-specified roadway design features include gradient, curvature, and surface-condition features. Additional energy consumption due to stopping and starting, as well as idling, is included in the calculations. For the average vehicle, the three major pollutants (HC, CO, and NO_x) are calculated for both cruising and idling.

The revised model output includes energy and air-pollution rates for each subsection during each time segment and summary tables that indicate energy and air-pollution impacts (as well as travel time) of various traffic-management strategies.

Spatial and Modal Demand-Response Extensions

FREQ3CP did not include demand-shift responses caused by various traffic-management strategies. A study of possible demand responses was undertaken, and spatial and modal demand shifts were selected for inclusion in the model. The results of previous research on spatial and modal demand shifts (12, 13) were adapted, and demand-response algorithms were developed for the existing model. Although the algorithms were not computerized and added internally to FREQ4CP, the developed algorithms were used off-line. The resulting spatial and modal demand shifts were determined, and the O-D patterns were manually modified for FREQ4CP long-term computer runs. The basic equation used to estimate demand shifts is

$$\text{Demand shift} = \text{sensitivity} \times \text{stimuli} \quad (1)$$

where

demand shift = percentage of passengers shifted from one route (or mode) to the other,
sensitivity = attractiveness consideration in changing routes or modes (i.e., availability of parallel routes and available unused capacity for route shift, and availability and quality of bus service for mode shift), and
stimuli = difference in travel time (i.e., freeway and ramp times are compared with alternate route travel times for route shift, and changes in bus travel time and nonpriority vehicle travel time are compared for mode shift).

Demand shifts are calculated in sequence; spatial shifts are calculated first and then modal shifts are calculated. At present no iteration procedure is used.

Two sets of analyses are undertaken for each freeway traffic-management strategy: short-term analyses that do not include the consequences of potential demand shifts and long-term analyses that include the consequences of spatial and modal demand shifts.

FREQ4CP

A flow chart of FREQ4CP is shown in Figure 1. FREQ4CP consists of the previously developed FREQ3CP, which was extended to include energy and air-pollution impacts as well as spatial and modal demand responses.

The user specifies the freeway design features, the selected freeway traffic-management strategy, and the freeway demand pattern. FREQ4CP predicts the travel time, energy consumption, and air pollution for existing conditions without the selected freeway strategy in effect and for both short- and long-term consequences with the selected freeway strategy in effect. In addition, FREQ4CP automatically constructs various contour maps (speed, volume and capacity ratio, density, energy, and air pollution). The new model also produces summary tables of traffic performance, impacts, and demand responses.

FREEWAY MODEL APPLICATIONS

The FREQ4CP model was applied to a 20.2-km (12.6-mile) section of the inbound Santa Monica Freeway during the morning peak period from 6:30 to 10:30 a.m. The freeway section was divided into 38 subsections, and the morning peak period was divided into sixteen 15-min segments. There were 20 demand input locations and 18 output locations. Prior to initiating production runs, existing conditions were simulated to ensure that model predictions realistically represented actual field conditions.

The experiment design for studying the various traffic-management strategies is shown in Figure 2. Four groups of traffic-management strategies were studied: priority-entry control operations, normal vehicle-entry control operations, exclusive bus-lane operations, and exclusive bus and car-pool lane operations. Both the short- and long-term consequences of these strategies were analyzed. Selected strategies were further modified considering user equity and additional practical aspects.

Tables 1, 2, and 3 give results for all selected freeway traffic-management strategies. The impacts and demand effects of priority-entry control were just slightly better than those of normal vehicle-entry control. Both strategies had favorable short-term consequences and led to even more favorable long-term consequences. The incremental benefits of priority-entry control over normal vehicle-entry control would be greater if the buses had used ramps that were controlled and if future traffic demand levels increased.

The preferential bus and car-pool lane had more favorable short-term and long-term impacts and demand effects than the preferential bus lane. The selected preferential bus and car-pool lane strategy was to reserve one lane for vehicles that carried three or more persons.

The comparison between priority-entry control and preferential bus and car-pool lane presents a trade-off among different impacts and demand responses. The following table highlights the predicted long-term differences between these two strategies for the morning peak period. The difference (priority-entry control minus preferential bus and car-pool lane) between these two strategies is as follows (where 1 L = 0.3 gal, 1 kg = 2.2 lb, and 1 km = 0.6 mile):

Item	Difference
Travel time, passenger·h	−6058
Fuel consumption, L	+647
Pollution, kg	−2703
Travel, vehicle·km	+3393

Priority-entry control strategy results in less travel time and air pollution but higher fuel consumption and vehicle-kilometers of travel. These trade-offs, plus the approximate manual procedures used in calculating demand shifts between modes and alternate routes, preclude specific conclusions.

ARTERIAL MODEL DEVELOPMENT

TRANSYT6

The TRANSYT6 (7) arterial model combines a simulation model with a search procedure capable of selecting near-optimum signal settings on a vehicle basis or a person basis. The input to the model consists of arterial design parameters, traffic-flow patterns, traffic-signal settings, and selected traffic-management strategies. The output is in three parts: simulation of traffic performance under existing conditions, near-optimum signal settings, and simulation of expected traffic performance with new signal settings.

The traffic performance for each directional link is calculated and includes flow level, degree saturation, distance traveled, travel time, delay time, stops, and maximum queue lengths. TRANSYT6 can be used as a network model, as well as an arterial model, and can include a maximum of 50 signalized intersections and 300 directional links. The model is macroscopic and deterministic, is written in FORTRAN, is operational on several different computer facilities, has been calibrated against field conditions, and has been applied at numerous locations throughout the world.

Energy and Air-Pollution Impact Extensions

TRANSYT6 uses delay time and number of stops as the primary impact measures. A study of other possible impact effects was undertaken, and energy and air pollu-

Figure 1. Flow chart of the FREQ4CP.

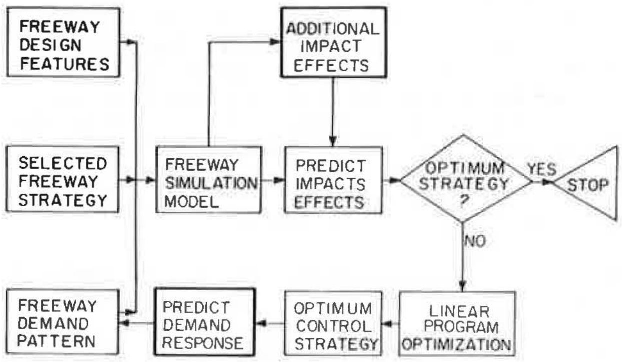

Figure 2. Design of experiment for freeway strategies.

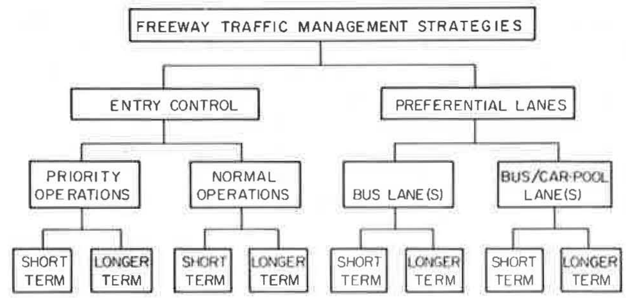

Table 1. Effects of freeway traffic-management strategies on travel time, fuel consumption, and air quality.

	Travel Time (passenger · h)		Fuel Consumption (L)		Air Pollutants (kg)							
					HC		CO		NO_x		Total	
Strategy	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent
Entry control												
With priority operation												
Short term	-1 934	-20	+773	+1	-65	-7	-511	-5	+261	19	-315	-3
Long term	-2 410	-25	-2719	-3	-125	-12	-1 129	-12	+209	15	-1 045	-9
Without priority operation												
Short term	-1 829	-18	+758	+1	-61	-6	-463	-5	+257	18	-267	-2
Long term	-2 380	-24	-2592	-3	-124	-12	-1 094	-11	+213	15	-1 005	-8
Preferential lanes												
Bus lane												
Short term	+14 639	+148	+2250	+3	+927	+93	+10 925	+113	-567	-40	+11 285	+94
Long term	+8 451	+85	-2885	-3	+507	+51	+6 452	+67	-550	-39	+6 409	+53
Bus and car-pool lane												
Short term	+11 110	+112	+2385	+3	+714	+72	+8 627	+89	-499	-35	+8 842	+73
Long term	+3 648	+37	-5208	-6	+156	+16	+1 989	+21	-447	-32	+1 698	+14

Note: 1 L = 0.26 gal; 1 kg = 2.2 lb.

Table 2. Effect of freeway traffic-management strategies on demand.

	Satisfied (%)		Transferred to Next Time Slice (%)	Diverted to Arterial Route (%)	Unsatisfied Queue at End (%)	Kilometers of Travel				Passengers in Vehicles (%)	
						Vehicle		Passenger			
Strategy	No Delay	Little Delay				Amount	Percent	Amount	Percent	Priority	Non-priority
Entry control											
With priority operation											
Short term	95	2	2	1	0	0	0	–	–	–	–
Long term	96	2	1	1	0	-21 022	-4	–	–	–	–
Without priority operation											
Short term	94	2	2	2	0	0	0	–	–	–	–
Long term	96	2	1	1	0	-21 022	-4	–	–	–	–
Preferential lanes											
Bus lane											
Short term					12	0	0	0	0	1.5	98.5
Long term					8	-21 022	-4	0	0	4	96
Bus and car-pool lane											
Short term					10	0	0	0	0	10.0	90
Long term					3	-26 460	-6	0	0	14.0	86

Note: 1 km = 0.6 mile.

Figure 3. Flow chart of TRANSYT6B.

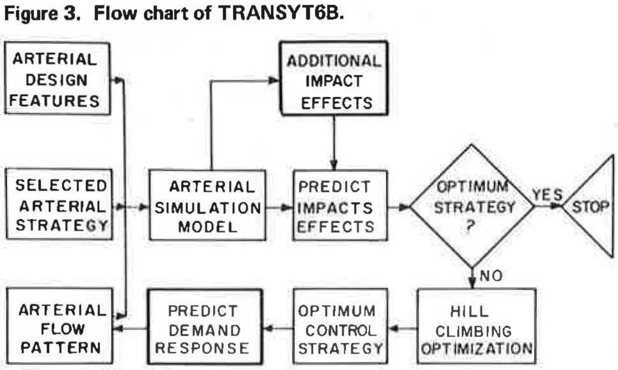

Table 3. Duration and extent of controls of freeway traffic-management strategies.

Strategy	Duration (a.m.)	No. of Ramps
Entry control		
With priority operation		
Short term	7:00 to 8:30	6
Long term	7:00 to 8:00	6
Without priority operation		
Short term	7:00 to 8:30	6
Long term	7:00 to 8:15	6
Preferential lanes		
Bus lane		
Short term	6:30 to 10:30	–
Long term	6:30 to 10:30	–
Bus and car-pool lane		
Short term	6:30 to 10:30	–
Long term	6:30 to 10:30	–

tion were selected for inclusion in the model. The results of previous energy (10) and air-pollution (11) research were adopted, and energy and air-pollution algorithms were added to the existing model. The procedures used in the arterial model are similar to those used in the freeway model previously described.

The revised model output includes energy and air-pollution rates for each directional link plus summary tables that indicate energy and air-pollution impacts, delay time, and number of stops of various traffic-management strategies.

Spatial and Modal Demand-Response Extensions

TRANSYT6 does not include demand-shift responses caused by various traffic-management strategies. A study of possible demand responses was undertaken, and spatial and modal demand shifts were selected for inclusion in the model. The results of previous spatial and modal demand-shift research (12, 13) were adapted, and demand-response algorithms were added to the existing model. The algorithms have been computerized and added internally to the TRANSYT6B arterial model and can automatically be employed by model users. The basic equation used for estimating demand shifts is

$$\text{Demand shift} = \text{sensitivity} \times \text{stimuli} \qquad (2)$$

where

- demand shift = percentage of passengers shifted from one route (or mode) to the other,
- sensitivity = attractiveness consideration in changing routes or modes (i.e., availability of parallel routes and available unused capacity for route shift and availability and quality of bus service for mode shift), and
- stimuli = difference in travel time (i.e., travel time on the studied arterial is compared with user-specified alternative route travel time for route shift, and changes in bus travel time and nonpriority vehicle travel time are compared for mode shift).

Demand shifts are calculated in sequence; spatial shifts are calculated first and then modal shifts are calculated. An iteration procedure is used in the spatial shift but not in the modal shift.

Two sets of analyses are undertaken for each arterial traffic-management strategy: short-term analyses that do not include the consequences of potential demand shifts and long-term analyses that include the consequences of spatial and modal demand shifts.

TRANSYT6B

A flow chart of TRANSYT6B is shown in Figure 3. TRANSYT6B consists of the previously developed TRANSYT6, which was extended to include energy and air-pollution impacts as well as spatial and modal demand responses.

The user may investigate traffic-management strategies that are concerned only with improving signal settings or may investigate strategies in which the arterial design features (preferential lanes or contraflow lanes) with or without improved signal settings are considered. TRANSYT6B predicts the travel time, energy, and air pollution for existing conditions without the selected arterial strategy in effect and for both short- and long-term consequences with the selected arterial strategy in effect.

In addition, the objective function was broadened so that minimizing delay time, number of stops, fuel consumed, air pollution, or any combination of these is possible. However, this feature has not been used, and a user input has not been developed. The new model also produces summary tables of traffic performance, impacts, and demand responses.

ARTERIAL MODEL APPLICATIONS

TRANSYT6B was applied to an 8-km (5-mile) section of Wilshire Boulevard (both directions) during the afternoon peak period studied. The arterial was divided into 276 directional links with 47 signalized intersections. Prior to initiating production runs, existing conditions were simulated to ensure that model predictions realistically represented actual field conditions.

The experiment design for studying the various traffic-management strategies is shown in Figure 4. Four groups of traffic management strategies were studied: optimizing signal control on a vehicle basis, optimizing signal control on a passenger basis, reversible-lane operations with optimizing signal control on a vehicle basis, and exclusive bus-lane operations with optimizing signal control on a passenger basis. Both the short- and long-term consequences of these strategies were analyzed. Sensitivity values selected for this operating environment were high for spatial shifts and average for modal shifts. Tables 4 and 5 give the results for all selected traffic-management strategies. Three of the four traffic-management strategies resulted in favorable short-term consequences, i.e., 3 to 10 percent reduction in travel time, fuel consumption, and air pollution. The exclusive bus-lane operation with optimizing signal control on a passenger basis was predicted to significantly increase travel time, fuel consumption, and air pollution in the short term. Optimizing signal control on a passenger basis and on a vehicle basis had the greatest short-term benefits.

The results of the long-term consequences are more difficult to interpret because of the spatial and modal demand shifts. The predicted long-term results of the exclusive bus-lane operations indicate little change in total travel: Passenger-hours of travel are reduced by 5.8 percent, fuel consumption is increased by 3.4 percent, and air pollution is increased by 2.0 percent. Unless the impacts are weighted in some fashion, the findings are inconclusive.

The predicted results of the other three traffic-management strategies were quite similar. The improvement in traffic operations on Wilshire Boulevard caused a significant demand shift to Wilshire Boulevard. In the long term, the impacts return approximately to their initial values. The significant change was the increased productivity on Wilshire Boulevard: It will handle 14 to 16 percent more traffic at the same level of travel time, fuel consumption, and air pollution as encountered before the study. Another interpretation is that traffic flows on parallel routes will be less and the traffic impacts will be improved. On a set of parallel arterials, therefore, seven improved arterials could handle the traffic of eight existing arterials without adverse impacts.

FUTURE RESEARCH DIRECTIONS

Research efforts will continue, and special attention will be given to the linear freeway and arterial traffic-flow models and to initial work linking these two linear models into a single corridor and network model.

Table 4. Effects of arterial traffic-management strategies.

		Travel Time (h)				Fuel Consumption (L)		Air Pollutants (kg)							
		Vehicle		Passenger				HC		CO		NO_x		Total	
Strategy	Vehicle[a]	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent	Amount	Percent
Signal control															
Short term															
Vehicle basis	Nonpriority	-81.8	-9.4	-97.6	-9.3	-234.0	-5.9	-9.0	-9.3	-109.6	-10.6	-4.5	-9.7	-123.1	-10.5
	Priority	-1.1	-5.0	-49.3	-4.9	-7.3	-6.7	-0.1	-4.8	-1.6	-7.5	0.0	0.0	-1.7	-7.0
	Both	-82.9	-9.3	-146.9	-7.2	-241.3	-5.9	-9.1	-9.3	-111.2	-10.6	-4.5	-9.8	-124.8	-10.4
Passenger basis	Nonpriority	-78.5	-9.0	-93.6	-8.9	-215.8	-5.4	-8.2	-8.6	-98.2	-9.5	-3.7	-8.0	-110.1	-9.4
	Priority	-1.4	-0.9	-65.4	-1.2	-10.0	-9.2	-0.1	-4.8	-2.0	-9.4	-0.1	-11.1	-2.2	-9.0
	Both	-79.9	-8.9	-159.0	-7.9	-225.8	-5.5	-8.3	-8.5	-100.2	-9.5	-3.8	-8.1	-112.3	-9.4
Long term															
Vehicle basis	Nonpriority	8.0	0.8	12.7	0.8	157	3.5	–	–	–	–	–	–	–	–
	Priority	0.1	0.0	6.3	0.3	-2	0.0	–	–	–	–	–	–	–	–
	Both	8.1	0.8	19.0	0.9	155	3.4	–	–	–	–	–	–	–	–
Passenger basis	Nonpriority	7.0	0.7	11.3	0.9	119	2.6	–	–	–	–	–	–	–	–
	Priority	-0.2	-0.7	-11.8	-1.2	-4	-3.9	–	–	–	–	–	–	–	–
	Both	6.8	0.7	-0.5	0.0	115	2.5	–	–	–	–	–	–	–	–
Signal control and design															
Short term															
Bus lanes	Nonpriority	508.1	58.2	607.5	57.6	1338.0	33.6	46.8	49.0	532.9	51.7	4.1	8.9	583.8	49.8
	Priority	-2.9	-13.1	-103.7	-13.4	-24.6	-22.6	-0.3	-14.3	-4.0	-18.8	-0.2	-22.2	-4.5	-18.5
	Both	505.2	56.4	503.8	23.1	1313.4	32.1	46.5	47.7	528.9	50.3	3.9	8.3	579.3	48.4
Reversible lanes	Nonpriority	-66.4	-7.6	-78.6	-7.5	-131.9	-3.3	-6.3	-6.6	-76.0	-7.4	-2.5	-5.4	-84.8	-7.2
	Priority	-0.2	-0.9	-12.8	0.2	-0.8	-0.7	0.0	0.0	-0.4	-5.6	-0.4	0.0	-0.4	-1.1
	Both	-66.6	-7.4	-91.4	-4.5	-132.7	-3.3	-6.3	-6.5	-76.4	-7.3	-2.5	-5.5	-85.2	-7.1
Long term															
Bus lanes	Nonpriority	18.6	2.1	19.8	1.8	168	4.1	–	–	–	–	–	–	–	–
	Priority	-2.9	-13.2	-139.7	-14.0	-25	-23.0	–	–	–	–	–	–	–	–
	Both	15.7	1.7	-119.9	-5.8	143	3.4	–	–	–	–	–	–	–	–
Reversible lanes	Nonpriority	6.4	0.6	9.4	0.8	196.2	4.2	–	–	–	–	–	–	–	–
	Priority	0.1	0.5	2.4	0.2	-2.0	-1.4	–	–	–	–	–	–	–	–
	Both	6.5	0.6	11.8	0.5	194.2	4.1								

Note: 1 L = 0.26 gal; 1 km = 0.6 mile; 1 kg = 2.2 lb.

[a]Total distance traveled: nonpriority vehicles, 18 554.8 km; priority vehicles, 340.2 km.

Figure 4. Design of experiment for arterial strategies.

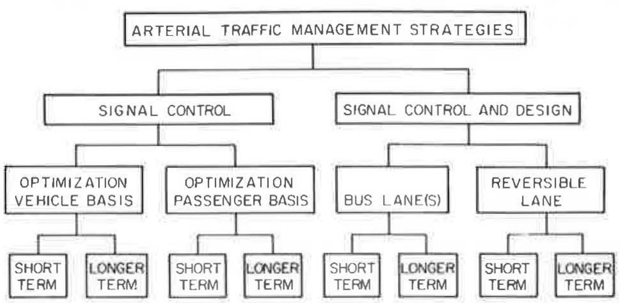

Table 5. Results of arterial traffic-management strategies.

Strategy	Base Conditions (km)	Strategy Results (km)	Change in Productivity (%)
Signal control			
Vehicle basis	18 773.9	21 479.5	14.4
Passenger basis	18 773.9	21 483.6	14.4
Signal control and design			
Bus lanes	18 773.9	18 481.6	-2.0
Reversible lanes	18 773.9	21 783.7	16.0

Note: 1 km = 0.6 mile.

FREQ4CP and TRANSYT6B will be extended to further evaluate demand responses, impacts, and control strategies in specified environments given alternative objective functions. Areas for possible research include the following:

1. Field validation and further refinement of spatial and modal demand shifts;
2. Extension of demand responses to include shifting demand over time and modifying total demand level;
3. Field validation and further refinement of energy and air-pollution impacts;
4. Extension of impact responses to include noise, safety, and operating costs;
5. Improvement of search procedures to obtain optimum control strategies that consider equity and additional practical aspects;
6. Extension of control strategies to include exclusive use of arterials for priority vehicles, bus and carpool lanes on arterials, and contraflow lanes on freeways;
7. Application of linear freeway and arterial models to additional operating environments and sensitivity analysis of operating environmental parameters; and
8. Provision for alternative objective functions and constraints and sensitivity analysis of the effect of these alternatives on evaluating the impacts of management strategies.

Management strategies affect traffic on a corridor and network basis; consequently, future research should also be directed to corridor and network models. Two approaches are contemplated: combining FREQ4CP and TRANSYT6B models or structuring a new modeling approach that is more macroscopic. The first approach will be initiated and will serve as a standard of comparison with the new modeling approaches. We anticipate that only feasibility studies of new modeling approaches will be undertaken in the coming year. Areas for possible research in combining FREQ4CP and TRANSYT6B models include the following:

1. Application of existing freeway corridor model, CORQ1C;
2. Provision of demand responses that include spatial, modal, time, and total demand responses;
3. Provision of impact responses that include energy, air pollution, noise, safety, and operating costs;
4. Improvement of search procedures to obtain optimum control strategies that consider equity and additional practical aspects;
5. Extension of control strategies to include integrated freeway and arterial traffic-management strategies;
6. Application of existing freeway corridor to additional operating environments and sensitivity analysis of operating environmental parameters; and
7. Provision for alternative objective functions and constraints and sensitivity analysis of the effect of these alternatives on evaluating the impacts of management strategies.

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Abridgment

County Evaluation of Traffic Engineering Activities

Tapan K. Datta and Brian Bowman, Wayne State University, Florida
Bruce B. Madsen, Traffic Improvement Association, Oakland County, Michigan

The National Highway Safety Act of 1966 was the result of national concern over traffic accidents and fatalities. Its enactment by the 89th Congress was based on the realization that uniform standards had to be established to effectively reduce safety deficiencies. In 1969, the National Highway Safety Bureau revised and published Highway Safety Program Standards, a manual prescribing standards for traffic engineering and operations. These standards attempt to accomplish the following:

1. Provide recommendations for the identification, surveillance, and correction of accident locations;
2. Establish uniformity in traffic-engineering operations, analysis control, and design of highway facilities; and
3. Ensure pedestrian safety.

To aid the various communities in Oakland County, Michigan, to achieve the standards of the highway safety act, the Traffic Improvement Association (TIA) of Oakland County, a private nonprofit organization, undertook a project to compare traffic-engineering operations in the county with appropriate safety standards and to develop corrective actions. This paper describes the data-collection procedure and summarizes the results

Table 1. Communities selected to participate in survey.

Class	Population Density (persons/km²)	Communities in Class	Communities Selected in Sample: Number	Communities Selected in Sample: Percent
1	>2317	7	6	86
2	772 to 2317	18	7	39
3	<772	35	17	49

Note: 1 km² = 0.4 mile².

and recommendations made to help achieve highway safety standards.

The size and diversity of the county and the complexity of variables involved in highway incidents made this task a significant one. Oakland County, in southeastern Michigan, is a part of the Detroit standard metropolitan statistical area (SMSA) and had a 1970 population of 907 871. Sixty communities spread over 2365.7 km²(913.4 miles²) maintain with the County Road Commission approximately 7401.4 km (4600 miles) of roadway.

Table 2. Summary recommendations for policy improvement.

Subject	Survey Results	Recommendations
Traffic-engineering functions in city charter and executive orders	Most communities do not include all traffic-engineering functions in city charter or executive orders	A formal standardized description of all traffic-engineering functions be prepared and recommended for adoption by local communities
Specific individual in community who has been formally assigned traffic-engineering responsibilities	27 percent of the communities do not formally assign traffic-engineering responsibilities to a specific individual	Specific communities be requested to formally assign traffic-engineering responsibilities to a specific individual
Official committee to review and handle traffic problems	72 percent of the communities do not have such a committee	All communities form such a committee or increase effectiveness of existing committee
Coordination between local agencies and county as a whole	70 percent of the communities indicated coordination greatly needs improving	Investigation be conducted of possibility of holding regular meetings of all traffic-engineering personnel in the county
Continuing education for policy makers	No community has such a policy; 93 percent indicated that a seminar on basic engineering approaches to traffic problems would have great value	Investigation be conducted to determine feasible means to disseminate basic traffic-engineering expertise to policy makers
Traffic-engineering manual for policy makers	90 percent of the communities indicated that summaries of basic traffic-engineering practices would have great value	Summaries of various traffic-engineering activities (i.e., warrants for traffic control) be prepared and distributed to all elected officials
Public support for excellence in traffic-engineering activities	50 percent of the communities indicated that some public support has been received for a few activities	Detailed procedure be developed for a public information dissemination plan and local agencies be advised to use this procedure
Traffic volume data	Communities indicated that a significant number of intersections and link locations do not have recent volume count data	Local agencies establish continuing programs for collecting traffic-volume data
Program to detect high-hazard locations	73 percent of the communities have no means of identifying high-hazard locations; 27 percent sometimes detect high-hazard locations by reconnaissance survey	A simple procedure be developed for detecting potential high-hazard locations and local agencies be trained to use it
Traffic-engineering priorities	93 percent of the communities have no traffic-engineering priorities	Traffic-engineering priorities be developed for entire county
Authorization of unwarranted traffic-control devices	Most communities authorize unwarranted installation of traffic-control devices	Communication between technical staff and governing body of local communities be increased to minimize or eliminate installation of unwarranted traffic-control devices
Written policies and procedures for performing traffic-engineering activities	87 percent of the communities have no written policies and procedures	Standard policies and procedures be developed and distributed to all communities for local adoption
Written job descriptions for traffic-engineering positions	87 percent of the communities have no written job descriptions	Standard job descriptions be prepared and distributed to local agencies
Written procedures for regulating traffic construction sites (in addition to the provisions of the manual)	87 percent of the communities have no written procedures for regulating traffic at construction sites	Written procedures be developed for traffic regulation at construction sites
Use of traffic-engineering professionals	18 percent of the communities in 1973 and 27 percent in 1974 used professionals more than 20 person-d/year	Increased use be made of traffic-engineering professionals
Assessment of traffic-engineering training needs	90 percent of the communities have no assessment of traffic-engineering training needs; 83 percent have never provided preservice training	A general procedure be developed that will allow individual communities to perform studies of training needs on a continuing basis

Table 3. Summary recommendations for system improvement.

Subject	Survey Results	Recommendations
Inventory procedures (traffic-control devices, roadway characteristics)	Most communities have no scientific method of record keeping; communities that keep records have manual systems, usually in the form of maps, drawings, or card files	Scientific method of inventorying and record keeping be pursued throughout the county
Capabilities of identifying non-skid-resistant pavement and procedures for correcting such deficiencies	No community has such a capability	Possibilities be investigated of procuring skid-testing equipment for the county and rectification procedures be developed
Capabilities of identifying substandard and deficient roadway lighting	70 percent of the communities have no means of identifying such deficiencies; those that have do not use scientific means	Standards and procedures be developed and adopted by local communities
Standards for locating utility poles along roadways	73 percent of the communities have no standards; those that have must have them checked with safety criteria	Standards be established
In-service training	54 percent of the communities provide no in-service training for professional development; 56 percent have sent employees to training programs offered by state universities	In-service training programs be increased in the county
Traffic-engineering staff	Only 3 communities have traffic engineers (by the standards of the highway safety act 10 communities should have traffic engineers)	Communities be requested to hire traffic engineers and traffic technicians as required by the highway safety act

PROCEDURE

A comprehensive questionnaire survey was made in the county to provide qualitative and quantitative measurements of current traffic-engineering information regarding organization, administration, personnel and operations, maintenance, budget, and community emphasis. This questionnaire consisted of 70 questions that dealt with the following general categories:

1. Traffic engineering problems—questions designed to elicit subjective opinions concerning perceived community traffic-engineering problems as well as county traffic-engineering problems;
2. Organization and administration—questions that pertained to formal policies and procedures for the initiation, performance, maintenance, and review of traffic-engineering functions;
3. Planning and implementation—questions concerned with the planning priorities and the implementation of highway-safety improvements;
4. Operations—questions that pertained to community traffic-engineering activities, methods of identifying hazardous conditions, management of accident-data inventory, highway features, and traffic-control devices;
5. Maintenance—questions related to methods and level of maintenance performed by the local community for traffic-control devices, highways, and lighting; and
6. Budget—questions directed toward determining how much of the community's total budget should be allocated to traffic-engineering activities.

COMMUNITY SURVEY

A stratified sampling procedure was used to select 30 candidate communities for the survey. In determining the stratifications, we recognized that the majority of traffic-engineering and safety problems occurred in that portion of the county where there is high travel demand and high population density. Thus, the number of samples in each category favored urbanized areas over nonurbanized areas. Table 1 gives the stratification used and the candidate communities selected for the survey.

The survey was administered in each community on a personal-interview basis by the community person who performs the traffic-engineering functions. Although answering the questions required approximately 2 h, most persons interviewed cooperated fully.

SURVEY RESULTS AND RECOMMENDATIONS

A summary of part of the survey results and recommendations is given in Tables 2 and 3. The subjects shown pertain to specific questions used in the survey, but the representation is not all inclusive.

The recommendations generated as part of this study were based on an evaluation of the current status of traffic-engineering activities within the county as determined from the questionnaire survey. The recommendations were aimed at specific problem areas in need of immediate attention to improve traffic-engineering activities. The recommendations were classified into two basic categories: policy and system. Policy improvements generally do not require much cost or personnel and may indirectly affect the accident experience in the entire county. System improvements produce direct results in terms of accident reduction if all other safety requirements are followed; these improvements often require a great deal of funds and personnel. Some of the recommendations, both policy and system, are also given in Tables 2 and 3.

CONCLUSIONS

The survey results and analysis clearly indicate the lack of traffic-engineering sophistication possessed by the majority of the sampled communities in Oakland County. The lack of conformance to highway safety standards may be typical not only of the sampled communities in Oakland County but also of the majority of small urban communities that, because of size or budgetary constraints, do not employ a qualified traffic engineer or technician to handle day-to-day activities. Efforts must be made at the county level or higher to aid agencies responsible for traffic operations.

The survey instrument developed as a part of this study is comprehensive and can be used by other communities. Typical policy improvements as presented here can easily be adopted by other communities to help achieve conformance to the highway safety standards.

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Abridgment

Evaluating Urban Highway Service

Marshall F. Reed, Jr., and Robert E. Heightchew, Jr., Highway Users Federation for Safety and Mobility

Differences are apparent in the quality of road services between one urban area and another as well as between one place and another within urban areas. Traffic moves more freely and quickly in some cities. Statistics indicate a wide difference in accident rates among cities. Some urban road systems are in better condition than others. Signal timing, lane striping, signing, and other traffic-control methods vary from city to city. Quantifying these variations, however, is not easy. Both data and standard analysis procedures for judging performance of urban road systems are lacking.

Recognizing these facts, we developed a composite rating system for assessing the quality of service of urban street and highway systems. The method and results of its application to 52 selected urbanized areas, as defined by the Federal Highway Administration, are given in this report. The objective is to stimulate transportation planners and urban officials to develop methods for periodic measurement of the performance of their urban road systems and to establish short- and long-range street and highway performance goals. Because the method has only been tested as a means for comparing street and highway service from urban area to urban area, the data used and the method outlined may not be the most appropriate for monitoring road performance in an individual urbanized area.

DATA BASE

The 1974 National Transportation Study, based on 1972 data, contains the most significant nationwide highway-performance information available. The following information was selected from this inventory to determine the performance of each street and highway network:

1. Total kilometers of street and highway,
2. Total annual vehicle-kilometers of travel on arterial streets and highways,
3. Total hourly capacity-kilometers available on arterial streets and highways,
4. Total land area, and
5. Total annual injuries per 160 million vehicle·km (100 million vehicle-miles) of travel.

URBANIZED AREAS ANALYZED

To keep the analysis small enough to be manageable and yet large enough to demonstrate clearly the process used and its results, we selected 52 urbanized areas from over 200 that were defined by the 1970 census.

In selecting the 52 urbanized areas, we considered urban size, location, and economy. All urban areas that had over one million people are included. Twenty selected areas had under 500 000 people. State capitals, resort centers, industrial cities, and agricultural trade centers are represented.

The boundaries of each urbanized area were established by federal, state, county, and city officials in the 1974 study. Included in each urbanized area is a central city of 50 000 or more persons and adjacent urban land that is expected to have a population density greater than 400 persons/km^2 (1000 persons/mile^2) by 1990. All system-performance measures are therefore 1972 data for road systems within 1990 urbanized boundaries.

PERFORMANCE MEASURES

Three measures of urbanized area street and highway performance were used in this analysis: accessibility, mobility, and safety.

Accessibility

Access to land is one of two basic functions or service features of roads. The accessibility provided by each urban road network was defined for this study as average road density, in kilometers per square kilometer (miles per square mile), of land area. As given in Table 1, road density in 1972 for the 52 urbanized areas ranges from 8.9 km/km^2 (14.3 miles/mile^2) of land area in St. Petersburg, Florida, to 2.4 km/km^2 (3.8 miles/mile^2) in Montgomery, Alabama, and Shreveport, Louisiana.

Mobility

Roads must provide for smooth flow of vehicle traffic. One method of evaluating the relative ease of traffic flow is to compare actual vehicle travel on street systems with the theoretical capacity of street systems to carry vehicle traffic. This method is performed by using calculated volume/capacity (v/c) ratios. The higher the v/c ratio is, the more crowded or congested the intersection, road section, or road system is. Arterial systems with numerically small ratios have less congestion, smoother traffic flow, and greater relative mobility. In this analysis an annual v/c ratio was used to represent the mobility factor. This procedure is a variation of the common practice of comparing hourly traffic volumes and capacities.

Annual vehicle-travel data were available from the 1974 study. The motor vehicle carrying capacity of each urbanized area arterial system, in terms of vehicle-kilometers of travel in one direction for 1 h, was converted to an annual basis first by doubling to obtain two-way hourly capacity values and then by multiplying by 8760 h/year. By dividing the reported annual vehicle travel on the arterial street and highway systems by the derived annual capacities of these systems, annual v/c ratios were calculated.

Table 1 gives the calculated annual v/c ratios for the arterial street and highway systems of the 52 selected urbanized areas. Pittsburgh's arterial system is judged the best in terms of mobility because of the calculated 0.073 v/c ratio. The data and calculations indicate the worst arterial street congestion in Madison.

Safety

The third measure of quality of an area's highway network is personal safety. Safety performance of urban roads is measured here as the number of motorists and pedestrians injured per 160 million vehicle·km. (100 million vehicle-miles) of travel. Table 1 gives the 1972 vehicle injury rate, as reported in the 1974 study, for each of the 52 urbanized areas. The low is 96.9 persons/160 million vehicle·km (100 million vehicle-miles) of travel in Columbus, Ohio; the high is 677.2 in Rochester, New York.

PERCENTAGE-OF-MEAN INDEX

Both planners and researchers would benefit from a technique that combines components of highway service for comparisons of time and areas. Owing to inadequate data and insufficient incentive for such tools, few performance-evaluation techniques have been developed.

Without resolving many of these problems, our analysis establishes a composite index for 52 urbanized area street and highway systems by calculating a percentage-of-the-mean index for each of the three performance factors and then adding those indexes. The mean for each factor always has a value of 100. In this index, bigger is not always better. Positive performance for v/c ratio and for accident rates is represented by low values, so these items have negative signs in the summation. The relation used for this combination is

Table 1. Percentage-of-mean and composite indexes for 1972 performance of urban roads in 52 urbanized areas.

Urbanized Area	Accessibility: Road Density (km/km^2)	Accessibility: Index	Mobility: V/C Ratio	Mobility: Index	Safety: Injury Rate	Safety: Index	Composite Index
Akron	1.4	82.1	0.155	87.1	139.72	52.4	142.6
Albuquerque	1.8	104.0	0.128	72.0	366.83	137.7	94.4
Atlanta	1.5	83.5	0.226	127.0	107.04	40.2	116.3
Austin	1.0	58.8	0.069	38.8	335.05	125.8	94.2
Baltimore	1.7	98.5	0.173	97.2	358.45	134.5	66.8
Baton Rouge	1.8	101.3	0.174	97.8	360.23	135.2	68.3
Boston	1.7	98.5	0.202	113.5	371.33	139.4	45.6
Charlotte	1.6	89.0	0.166	93.3	255.54	95.9	99.7
Chicago	1.8	101.3	0.227	127.6	302.14	113.4	60.3
Cincinnati	1.4	78.0	0.178	100.0	167.68	62.9	115.1
Columbus	1.3	76.6	0.164	92.1	96.91	36.4	148.0
Denver	2.2	123.2	0.155	87.1	238.63	89.6	146.5
Detroit	1.7	98.5	0.288	161.8	184.65	69.3	67.4
Erie	1.9	108.1	0.125	70.2	238.84	89.6	148.2
Flint	1.5	87.6	0.234	131.5	224.69	84.4	71.8
Fresno	2.5	142.3	0.137	77.0	175.25	65.8	199.5
Grand Rapids	1.8	102.6	0.173	97.2	263.57	98.9	106.5
Hartford	1.6	93.1	0.168	94.4	227.20	85.3	113.4
Honolulu	2.3	131.4	0.271	152.3	305.80	114.8	64.3
Houston	1.5	83.5	0.127	71.4	168.91	63.4	148.7
Jacksonville	1.2	68.4	0.172	96.7	344.63	129.4	42.4
Kansas City	1.8	102.6	0.130	73.0	252.54	94.8	134.8
Knoxville	2.8	160.1	0.185	104.0	147.07	55.2	200.9
Los Angeles	2.4	135.5	0.182	102.2	214.71	80.6	152.7
Madison	1.3	76.6	0.549	308.5	168.08	63.1	-95.0
Miami	2.6	149.2	0.272	152.9	589.61	221.3	-24.9
Minneapolis	1.9	105.4	0.205	115.2	225.20	84.5	105.7
Montgomery	0.9	52.0	0.123	69.1	139.12	52.2	130.7
Nashville	2.0	113.6	0.195	109.6	183.06	68.7	135.3
New York City	2.2	123.2	0.138	77.6	544.38	204.3	41.3
Norfolk	1.4	78.0	0.199	111.8	209.32	78.6	87.6
Omaha	2.0	116.3	0.139	78.1	320.71	120.4	117.8
Philadelphia	1.6	89.0	0.200	112.3	246.84	92.7	83.9
Phoenix	1.6	91.7	0.151	84.9	376.86	141.5	65.4
Pittsburgh	1.7	95.8	0.073	41.0	169.96	63.8	191.0
Portland	2.1	121.8	0.171	96.1	415.72	156.0	69.7
Providence	1.9	106.7	0.187	105.1	351.67	132.0	69.6
Richmond	1.3	73.9	0.197	110.7	190.89	71.7	91.5
Rochester	1.4	80.7	0.112	63.0	677.15	254.2	-36.4
Salt Lake City	1.8	99.9	0.149	83.7	259.46	97.4	119.8
San Diego	1.6	89.0	0.096	54.0	149.31	56.0	178.6
San Francisco	2.5	142.3	0.170	95.6	189.95	71.3	175.4
St. Louis	1.7	97.2	0.164	92.1	242.27	90.9	114.2
St. Petersburg	3.4	195.7	0.190	106.8	327.64	123.0	165.9
Seattle	1.5	86.2	0.206	115.8	287.96	108.1	62.4
Shreveport	0.9	56.1	0.196	110.1	205.84	77.3	68.7
Sioux City	1.7	97.2	0.116	65.2	222.12	83.4	148.6
Spokane	1.9	108.1	0.166	93.3	372.29	139.7	75.1
Tucson	1.3	71.2	0.162	91.0	362.23	136.0	44.2
Tulsa	1.6	90.3	0.108	60.7	128.45	48.2	181.4
Washington	1.4	79.4	0.206	115.8	244.56	91.8	71.9
Wilmington	1.9	105.4	0.206	115.8	205.86	77.3	112.4

Note: 1 km/km^2 = 0.4 miles/mile^2.

$$\text{Composite index} = \text{accessibility index} - \text{mobility index} - \text{safety index} \quad (1)$$

For a hypothetical, average area, the composite index would be composite index = 100 - 100 - 100 = -100. To translate the composite index scale to correspond to the individual index scales, a linear translation (200) was applied. Therefore, average index for the hypothetical area would be composite index = 100 - 100 - 100 + 200 = 100, and the mean area would still be represented by 100 percent of the mean of all characteristics. The final relationship expressed with variables becomes

$$\text{Composite index} = \text{accessibility index} - \text{mobility index} - \text{safety index} + 200 \quad (2)$$

This approach weights each of the three factors equally. This simplification was used because it was the best alternative; other weightings could be used with this system.

Also, our study implies that each variable continues to contribute to system performance as its value continues to increase. Clearly, an indefinite number of kilometers of road per square kilometer of land would be neither optimal nor desirable. For this analysis, optimum values for each variable were assumed to be outside the ranges studied, and linear relationships were assumed to be within each range.

PERFORMANCE OBSERVATIONS

The resulting composite index of highway performance measures for the 52 selected urbanized areas is given in Table 1. The observations listed below indicate the type of information a nationwide analysis of performance measures would yield.

1. The larger urban areas that have grown rapidly in recent years generally have higher highway-performance factors. San Diego, Los Angeles, and Houston are all in the top 10 of the composite index. However, fast-growing Phoenix ranks low mainly because this city has a poor highway-safety record.

2. Most old, large urbanized areas have low composite scores. New York, Boston, and Chicago have poor highway-performance ratings. Pittsburgh, how-

ever, ranks high because of its good safety record and excellent mobility rating.

3. The urban area rated high most consistently in terms of street and highway performance is Fresno, which ranks fifth in accessibility and twelfth in both safety and mobility and therefore has the second highest score in the composite index.

4. Grand Rapids, most nearly typical of the urban areas analyzed, scores 102.6 in accessibility, 97.2 in mobility, and 98.9 in safety and has a composite score of 106.5.

5. No single urbanized area ranks consistently low in all the indexes.

6. Columbus, highest scoring in the safety index, has an injury rate of less than half the arithmetic mean of the 52 selected urbanized areas, indicating a street and highway system that was designed and is being operated with strong emphasis on safe movement of motor vehicles and pedestrians.

7. St. Petersburg, which has an accessibility score of 195.7 and is the leader of that index, has almost twice the road kilometers per square kilometer of land of the average urbanized area and an accessibility score four times that of the lowest ranked city, Montgomery. The St. Petersburg urbanized area is apparently highly compact; most of its urbanized area is fully developed and well served by streets and highways. The Montgomery urbanized area apparently contains much underdeveloped land not well served by roads.

Since all urbanized areas are not included, the resulting indexes of highway service cannot be interpreted as national rankings. No doubt other urbanized areas have highway service characteristics both superior and inferior to those of the cities selected.

NEEDED RESEARCH

Traffic volume and roadway capacity data, as reported in the 1974 National Transportation Study, were used to assess mobility. However, average speed data segregated by various functional classes of urban road would more directly indicate vehicle mobility. Unfortunately, such data are not as yet universally available, and volume-to-capacity ratios are used instead.

Further study might show that other features of road performance in addition to accessibility, mobility, and safety might prove to be useful in analyzing urban road performance. An engineering appraisal of road surface might be included in further study because of the importance of road surface to travel comfort and to vehicle maintenance cost. But again such data are not generally available.

More study is required to translate performance measures into standards against which urban street and highway performance can be compared. Lacking standards for accessibility, mobility, and safety, we relied on the arithmetic mean for the selected cities as a basis for judging the comparative road performance. Further research might define, for example, an optimum road density as a benchmark for accessibility.

The question of weighting the performance measures is raised because the relative importance of the measures used in the analysis is unknown. For instance, the importance of mobility relative to accessibility is not clear. Lacking such information this analysis gave equal weight to each measure. Further research might reveal that accessibility, for example, is a relatively minor consideration, and safety and mobility are the primary measures of urban road performance. Particularly useful in this regard would be factor-analysis techniques applied to existing data.

CONCLUSIONS

This analysis of urban highway performance confirms the assumption that there are differences in quality of highway service in urbanized areas and that methods can be devised to assess urban road performance. However, lack of adequate data is a serious impediment to use of any method in comparing or monitoring urban road performance.

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Methodology for Evaluating Bus-Actuated, Signal-Preemption Systems

Samuel R. Seward and Robert N. Taube, Milwaukee Department of City Development

The objective of this research was to model the impact of bus-actuated, signal-preemption systems on delay experienced by buses at signalized intersections and to develop a methodology to evaluate these systems by location. The model developed is green-extension strategy that quantifies the effect of the system on bus and other traffic at intersections depending on the characteristics of the intersections. Based on random arrivals, equations quantify the travel-time savings and losses experienced. Then, the cost of the preemption system is developed, and a revenue-cost ratio for any location is developed. The application of this revenue-cost methodology to a local bus route resulted in a 14:1 revenue-cost ratio. Transportation planners who reviewed this result and methodology expressed the desire to emphasize the ability of this system to reduce bus running times enough to remove at least one bus from the route. This criterion was applied and a bus was removed in the test cor-

ridor. Another result of this review was the initiation of field checks to test the assumption of random bus arrivals. Although these checks are not complete, the preliminary results suggest that under most circumstances the random-bus-arrival assumption is valid. Furthermore, in the cases that are being identified by these field checks as not having a uniform distribution, the distribution may either lessen or enhance the feasibility of a signal-preemption installation. From these results we concluded that the methodology and the priority technique are both sound.

Delay of buses at controlled intersections constitutes 10 to 20 percent of the average bus trip time (1). Bus-actuated, signal-preemption systems minimize or eliminate bus delays at intersections by temporarily altering the traffic signal phase so that an approaching bus receives a green phase when it arrives. The development of signal actuation by buses was prompted by the difficulty of adjusting standard fixed-time signal controllers to platoon buses through a series of controlled intersections because bus travel time through the same route segment varies from run to run. This variance is caused mainly by variations in the number of passengers boarding and alighting and the time that each passenger takes.

Figure 1 (2) shows the difference in the normal movements of a platoon of traffic and a bus. Because of this mismatch between signal-timing characteristics and bus-operating characteristics, experiments have been conducted to test a variety of methods for minimizing bus delays at signalized intersections. Through the Urban Traffic Control System-Bus Priority System (UTCS-BPS), U.S. experiments in Washington, D.C., Miami, and Louisville have concentrated on the development of hardware and software for area control of a series of interconnected intersections (3, 4, 5, 6, 7, 8). In Europe the emphasis has been on understanding the impact of controlling isolated intersections (9).

In most of these experiments bus-actuated, signal-preemption systems proved to be feasible and, in fact, to provide significant time savings to the bus user and transit operator. Moreover, time savings can generally be gained without seriously affecting cross-street traffic. The reduction in mean travel time for buses produces a more attractive service and enables the same level of service to be provided with fewer buses; thus revenue is increased but cost is reduced. However, to date little work has been done on generalizing the results of these experiments, and design guides and warrants for bus-actuated, signal-preemption systems have not been developed. [The exception is a report by Ludwick (10).]

The purpose of this research, therefore, was to examine the operating conditions under which a signal-preemption system can be operated and to construct equations that describe the costs and benefits to buses and other traffic. These equations were then used to develop a method by which the economic desirability of installing a bus-actuated, signal-preemption system at any particular location could be evaluated. This paper summarizes the results of a literature search on this topic, describes planning guidelines for use of the technique, and describes the development of the methodology.

CHARACTERISTICS OF A BUS-ACTUATED, SIGNAL-PREEMPTION SYSTEM

System Components and Operating Characteristics

A bus-actuated, signal-preemption system allows the bus driver to communicate with signal controllers and "instruct" them to alter the phase of the signal so that the bus has a green phase available when it arrives at the intersection. The system must contain three basic components: an identification scheme, a communication link, and a logic unit incorporated into the controller's operations. The identification scheme most commonly used involves a radio transmitter carried aboard the bus; however, magnetic and optical detection schemes are also possible. The bus-carried radio transmitter emits an ultrahigh frequency (UHF) signal or uses a near-field transmission that is picked up by a loop antenna buried in the roadbed. The cost of these transmitters ranges from \$30 to \$50/bus. The estimated cost of on-site equipment is \$50 for an antenna and \$100 for a receiver. The cost of a single approach that uses two antennas and one receiver is \$400, plus modifications to the signal control and installation costs of approximately \$200. If an intersection contains more than one bus approach, the cost of modification to the controller could probably be shared. OPTICOM, a system developed by the 3M Company, uses an optical transmitter with a receiver that is mounted on the traffic signal standard, which eliminates some construction costs. However, emitter (sender) units cost more for this equipment.

Hardware technology, however, is advancing; thus the cost of signal-preemption systems is being reduced while effectiveness is being increased. An example is the Passive Bus Detector/Intersection Priority System (PBD/IPS) that was developed by the Federal Highway Administration (FHWA) (11). The PBD/IPS uses an inductive loop detector and transducer that identify various vehicles by a unique magnetic signature and thus eliminates the need for bus-carried equipment.

The communication link connects the identification scheme with either a centralized computer or a localized logic unit. Carrying the message that a bus is approaching and other related messages to a centralized logic unit, such as a computer, requires a complex network and a sharp increase in equipment and installation costs. Thus, an important factor in the choice of systems is the cost of such an extensive communication network.

The logic unit receives a stimulus from the detectors, after which the unit implements a preemption action, subject to any constraints incorporated within its algorithm. The most sophisticated logic unit possible is a computer, which could collect information from several sources and make a split-second decision on the granting of a preemption. The simplest logic unit would be mechanical and would properly plan the configuration of the system to control the preemption. Whichever method is used, the logic unit then produces a command that is carried to the relay logic interfaced with the standard traffic-signal controller and alters the cycle phase.

Signal Modifications

A preemption can be performed by extending the green phase, truncating the red phase, or interrupting the red phase. Red truncation and red interruption were found to be less effective and more difficult to implement. Therefore, these modifications were not considered further in this analysis.

Green extension, which was analyzed, consists of the elongation of the green phase when an approaching bus has been detected and the system has determined that the arrival time of the bus is within a period immediately following the start of the red phase and some maximum extension length. If the bus arrival were detected after the start of the red phase, then the extension would not be contiguous to the preceding green phase, and thus the

Figure 1. Time-distance difference between normal movements of traffic platoon and bus.

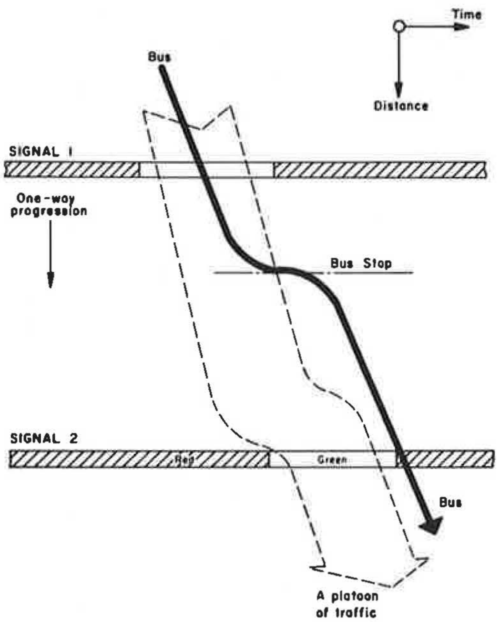
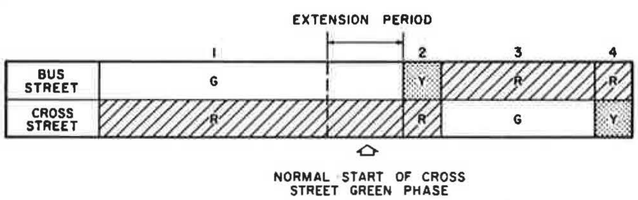


Figure 2. Signal modification to delay red phase and extend green phase.



green extension would not be possible. When the bus is expected to arrive within the period immediately following the phase change, then the red phase is delayed and the green phase is extended until the bus has entered the intersection or the maximum extension period has been reached (Figure 2).

System Logic Processes

The signal modification previously described is implemented by predetermined logic incorporated into the components of the system. These components determine whether an approaching bus needs a signal preemption to avoid stopping at an intersection and whether a bus is eligible to receive the particular signal modification the preemption system has available. The components must determine three events:

1. When a bus is detected;
2. When a bus is expected to arrive at the intersection (thus, whether it will probably need a priority); and
3. Whether the bus-arrival time allows the bus to be eligible for a particular signal modification.

Although complicated, the process can be accomplished by using simple mechanical components if the system is configured correctly.

When a bus is detected, bus-arrival time at the intersection is assumed to be equal to the detection time plus the average travel time from the detector to the intersection. Then, if the arrival is expected at a time when priority is possible, the signal modification is put into effect. For this signal modification to occur, however, the travel time between the detector and the intersection must also be greater than or equal to the maximum length of the preemption period. This time factor is necessary to ensure the use of the full preemption period. For example, a bus that is expected to arrive at 10 s after the start of the red phase will not get a preemption from a 10-s green-extension strategy if this bus is detected only 6 s before its arrival at the intersection because the red phase will have begun 4 s before its arrival. Thus, the bus must be detected 10 s before its arrival at the intersection for the system logic to determine whether the phase change should be delayed.

Operational Limitations

The maintenance of pedestrian safety affects the operation of any signal-preemption system. If pedestrians cross with vehicle flow, pedestrians must be stopped from entering the intersection and pedestrians already crossing must clear the intersection before the end of the green phase. This usually is performed by flashing DON'T WALK or DON'T START signals. The length of pedestrian clearance time depends on the geometrics of the intersection. The standard speed for pedestrian movement used by traffic engineers is 1.22 m/s (4 ft/s). Thus, the time necessary for clearance (y) is

$$y = (\text{distance } x/1.22 \text{ m/s}) \quad (1)$$

As a result, the distance a pedestrian must travel to cross the bus-street width determines the limit to which the preemption can encroach on the normal green time of the cross street.

Another constraint that must be dealt with in the design of a signal-preemption system is adequate clearance time for cross-street traffic. In essence, the designer of the system must balance two conflicting objectives: expedite bus travel and at the same time not unduly delay cross-street traffic. Thus the designer must constrain the preemption period based on minimum green time necessary for vehicle clearance if that preemption period is longer than pedestrian clearance time. If the existing green time for the cross street exceeds the longer of the two minimum clearance times, the excess amount is slack time. This slack time is the portion of cycle length not necessary to maintain cross-street traffic flow and pedestrian safety and can be shifted, when needed, to the green phase on the bus street to avoid delay.

COSTS AND REVENUES OF SIGNAL-PREEMPTION SYSTEMS

To develop a warrant for bus-actuated, signal-preemption systems, we had to perform the following:

1. Obtain cost data for necessary equipment;
2. Assign values to time savings associated with signal preemption;
3. Devise equations that describe the effects of signal preemption on traffic conditions; and
4. Devise equations that describe the relation between system costs, return per preemption, and frequency of bus use of preemption system.

The warrant was then applied by calculating revenue-cost ratios for installation of preemption equipment at candidate intersections.

The methodology understates the return from the system because generalizing the results of preemption installation is not possible with respect to attracting new riders. The modal split for a given corridor is the result of the relative attractiveness of all modes and the characteristics of the trip makers, not the absolute performance of the preemption installation. Thus, although signal preemption would increase bus ridership and system revenue, these benefits are not counted in this methodology.

Automobile operating costs were not considered because those costs are not primarily dependent on time as are bus operating costs. Also, the equations do not consider failures of the system, e.g., failure of a bus to make use of a granted preemption because of chance delay between detection and arrival at the intersection. We felt the occurrence of delays resulting from some uncontrollable traffic or passenger conflict would be rare, and thus the frequency of such occurrences can only be determined through experimentation with the preemption strategy and the methodology.

Bus-Operating Savings

For each preemption used, the travel time experienced by the bus that actuated the preemption is reduced. The time saved is equal to the red time not experienced by the bus that was granted the preemption.

Since only eligible buses can be granted a preemption, time saved is dependent on the arrival time of the eligible bus and the length of the red phase that is not experienced. If over a long period of time buses arrive randomly at the intersection, the bus arrivals will range from the last second of the normal green (when extension is first actuated) to the last second of the extended green. Therefore, the average time of arrival is one-half the length of the green extension. This conclusion also assumes that some prior decision has to be made to detect the presence of an upstream bus that will arrive sometime during the 10-s extension.

Hence, the time saved by a bus entering an extended green phase is the full length of the red phase missed minus the average time of arrival, which is one-half the extension period. This time is converted into its cost equivalent by a dollar value per minute of bus-operating time as shown below:

$$\text{Bs} = \{[\text{cross-street green time} - (\text{max extension length}/2)]/60\ \text{s/min}\} \times \text{value of operating time} \qquad (2)$$

where Bs = bus-operating savings.

Bus-Passenger Savings

For each preemption used the time saved by the bus is passed on to the passengers it carries because they also do not experience a red phase. These passengers perceive this time saved at some monetary value. Assuming an average perceived value of time, we can calculate a bus passenger's savings, which is multiplied by the number of passengers on the bus to determine total value of passenger time saved.

$$\text{BPs} = \{[\text{cross-street green time} - (\text{max extension length}/2)]/60\ \text{s/min}\} \times \text{perceived value of travel time} \times \text{average number of passengers per bus} \qquad (3)$$

where BPs = bus-passenger savings.

Automobile-Passenger Losses

To determine automobile-passenger loss (AP_L) experienced during a preemption action, we assumed that the total queue of cross-street traffic would be delayed. This assumption is conservative because only with perfect progression and operation at capacity would the entire platoon of cross-street traffic be stopped and delayed. Under any other conditions only a portion of the queue would be delayed the full length of the extension.

The queue length (in passengers) is determined by the volume of cross-street traffic and the average occupancy per automobile. The perceived value of travel time of automobile passengers (including driver) is equal to the earlier value for bus-passenger travel time. The number of vehicles delayed for an additional cycle by the preemption action is determined by the minimum green time and the associated failure rate. The level chosen for this methodology was a 10 percent failure rate for the peak period, usually 3 h, which approximates a 30 percent peak-hour failure rate (12). If we assume that this failure rate will cause an average increase in the failure rate of 10 percent, then this additional delay must be accounted for in the equation because 1 out of every 10 cross-street vehicles present during a preemption will be delayed the maximum extension period and will be unable to successfully clear the intersection during the cross-street green phase and will thus experience further delay. This assumption is also conservative because failures will probably be limited to hours within the peak periods, yet the method assumes failures due to preemption actions will occur throughout the total operating period. The resulting equation is as follows:

$$\text{AP}_\text{L} = [(\text{max extension length}/2)/60\ \text{s/min}] + (0.1)\{[(\text{max extension length}/2) + \text{cross-street red}]/60\ \text{s/min}\} \times \text{preceived value of travel time} \times \text{number of passengers per automobile} \times \text{average number of cross-street vehicles per cycle} \qquad (4)$$

Automobile-Passenger Savings

To determine how many automobile passengers will gain time by an extended green phase, we assumed that additional volume can be anticipated beyond the platoon of automobiles that would have normally cleared the intersection in a perfectly progressed system. This additional volume might be generated by previous preemption actions, by automobiles entering the link from side streets, or by failures at upstream intersections. In a less than perfectly progressed system, there is potential for late arrivals that would benefit from an extension. Travel time savings of automobile passengers equal to the length of the bus-street red phase minus one-half of the green extension period would result if these late arrivals or additional volumes appeared. However, these occurrences are difficult to predict and are not general conditions.

Automobile passengers who have saved travel time would still be behind the normal traffic platoon and would have difficulty maintaining their savings unless the bus continues to travel with the extended platoon and to preempt signals. If the bus leaves the traffic flow to make a service stop and successive signalized intersections are progressively timed, then there is a high probability that the savings to the automobile passenger will be lost at the next intersection. Because of the uncertainty of maintaining the savings and the efforts to present conservative estimates for bus-actuated signal preemption, the possibility of automobile-passenger savings (APs) was not considered in the revenue-cost analysis. Thus, the revenue per preemption (R/P) used of a green-extension preemption scheme of bus priority is

$$\text{R/P} = \text{Bs} + \text{BPs} - \text{AP}_\text{L} \qquad (5)$$

Total Preemptions Used

Once the return per preemption is determined by using the preceding equations, the next task is to estimate the total number of preemptions granted during the life span of the equipment (P/LS). This estimate is a function of bus frequency, cycle length, extension length, and total life span of equipment and is calculated by the following equation:

P/LS = extension length/cycle length x weekday bus volume
x number of equivalent weekdays per year
x number of years per life span (6)

The major assumption in the computation of the total preemptions used per life span is that the proportion of preemptions granted equals the length of the maximum extension period during the total cycle length. Also, we assumed that all preemptions granted are used. The goal of the dual detection scheme is to minimize unused granted preemptions and thus give validity to this assumption.

Computation of Revenue-Cost Ratios

When all of the cost and revenue components of a signal-preemption system are known, the next step is to determine the revenue-cost ratio for the system and thus its feasibility. Because actuators can be used to operate several installations on a route or corridor, the revenue-cost ratio (R/C) equation takes the following form:

R/C = (revenue per preemption used x number of preemptions used per life span)/[on-site equipment costs + (actuator cost/number of sites using these actuators) + engineering and maintenance costs] (7)

This revenue-cost ratio is then used to determine the economic desirability of the installation of a preemption system at any particular location. If the revenue-cost ratio is greater than one, the installation is justified. The equation implies that, although preemption equipment must be justified on the basis of the revenue-cost ratio at the candidate intersection, the optimal configuration is to convert as many intersections as possible on a single corridor. Thus, the expense of installing actuators on buses is amortized by the largest possible number of intersections, and the unit cost per intersection is reduced as much as possible. Moreover, a passive detection or identification scheme, which uses some mechanism other than bus-mounted transmitters, would eliminate the actuator cost entirely and further improve the revenue-cost ratio.

APPLICATION OF THE METHODOLOGY

The methodology described was used to evaluate the economic desirability of installing a bus-actuated, signal-preemption system in a street corridor in Milwaukee that was outside of the central business district and had a high bus frequency. The selection of this study area, containing 11 signalized intersections, from 122 similar sites was based on the following:

1. The intersections had to be situated on one local bus route;
2. The corridor had to be intersected by only three arterials so that only a minimum of cross-street traffic existed;
3. The route had to consist of a pair of one-way arterials; and
4. Only one intersection could contain a cross-street bus flow other than that resulting from route branching.

The objective of this methodology is to screen and warrant individual intersections for installation of bus-actuated, signal-preemption systems. This methodology is an iterative process that is data intensive. The adequacy of the revenue-cost ratio for determining system feasibility depends on the level of detail obtained, which is a policy decision.

The first step of the process, screening of intersections, is more general and uses as criteria bus frequency and major conflicts (or lack of conflicts) with pedestrian or cross-street bus flow. Sufficient bus frequency is considered to be 10 or more buses/h.

Signalized intersections in the CBD were not considered because of conflicts with pedestrian movement. Other intersections were dropped from consideration because of sufficient probability that gains received by through buses would be canceled by cross-street bus flow. An arbitrary limit to the combined bus frequency was chosen as sufficient cause to drop an intersection from consideration. If a minimum of two buses, one from each major bus-flow direction, would arrive at the intersection within a period of less than five cycle lengths, then the probability that these two buses will arrive within the same cycle and in such a manner as to conflict with the movement of the other was considered to be too great. Thus, 3 out of 5 intersections containing some sort of cross-street bus flow were not considered candidate intersections. The number of potential intersections was then reduced from 11 to 8 and the approaches from 20 to 11.

In the second step, the slack time at each signal was calculated from the pedestrian and cross-street traffic clearance required at each intersection being examined. Sufficient slack time was available for at least a 10-s green extension in 19 out of the 20 approaches examined. The approach that did not have slack time was part of the intersection containing five legs; therefore, 10 of the 11 approaches qualified under both criteria.

In the third step, a preliminary revenue-cost analysis was performed on candidate intersections. An estimated average return per preemption was determined from the equations derived in the preceding section. The data necessary to compute these equations include the cycle length, signal split, traffic volumes, and average loadings. In addition, assumptions must be made regarding the bus-operating cost and the passenger's perceived value of travel time. Knowledge of the slack time available determines which preemption length is possible and therefore what proportion of the total number of buses in the primary direction will receive a preemption. The total number of buses is easily obtainable from bus schedules. This information is adequate to estimate the average daily return for a candidate intersection. The cost was determined in the following manner.

1. Actuator cost was determined to be \$30/actuator and the number of actuators needed is determined by the specific characteristics of the bus route (7). The test route has approximately 50 vehicles in operation during peak periods; therefore, 50 actuators are required at a cost of \$1500.
2. On-site equipment cost was estimated from the previous experiments. The Louisville experiment projected the average cost per intersection (more than 26 intersections) to be \$500 (7). The Washington experiment estimated that the antenna would cost \$50 and the receiver would cost \$100. Using these estimates, we estimated that a single approach using two antennas and one receiver and requiring approximately \$200 worth of modification to the traffic control would cost approxi-

mately \$400/approach. In cases in which intersections contain more than one approach, shared cost of signal modifications may be possible and total cost reduced. However, no such assumption was made in this analysis. Thus, for the test corridor, which contained 10 approaches to be equipped, a total cost for the on-site equipment was estimated to be \$4000.

3. Equipment engineering and maintenance costs incurred during the assumed life span of the equipment were estimated to equal 100 percent of the total equipment cost. The test corridor has a \$1500 actuator cost and a \$4000 on-site equipment cost. A \$5500 engineering and maintenance cost will be incurred during 10 years, and the total cost of the system during its use is the sum of the cost estimates for the actuators, on-site equipment, engineering, and maintenance, which is \$11 000 or \$1100/approach.

4. The revenue-cost ratio was then determined by using the methodology presented in this paper and time value of \$15/bus-operating h and \$1.25/h of traveler's time. The average daily return per approach ranged from \$2.40 to \$7.52, and the average daily return for the total system was \$50.47. A break-even time of 218 equivalent weekdays was determined by dividing total cost per day by the total average daily return. The computed revenue-cost ratio for the entire system was 14:1, and individual intersections ranged between 4.5:1 and 19.8:1. Therefore, we concluded that the installation of a green-extension capability at the identified locations is not only feasible but also economically desirable.

As a result of these preliminary findings, a consortium of transportation planners from the community reviewed and commented on the methodology and the implementation potential of bus-actuated, signal-preemption systems. There was little argument as to the feasibility of signal preemption and the costs of installing such equipment. However, there was skepticism as to the feasibility of implementing this transportation improvement because the prime benefits are based on travel-time savings. Although the majority of transportation improvements are justified by time savings to the traveler, the planners felt that a more tangible benefit would have more influence on officials responsible for public expenditures. Therefore, the suggestion was given and followed that the installation of signal-preemption equipment be based primarily on the ability to reduce bus requirements.

The definition of reducing bus requirements was further limited to the ability to eliminate a bus from service and thus reduce labor cost without reducing the level of service offered. To eliminate a bus from service requires that the accumulation of average bus travel-time savings from all the intersections on a bus route (in both directions) be equal to or greater than the bus headway. The accumulated travel-time savings on two major routes in Milwaukee were estimated to be 5.9 and 6.9 min, large enough to eliminate a bus from peak-hour service.

Another result of this review was the questioning of the assumption of random bus arrivals. Preliminary field tests were conducted that generally support the assumption of a uniform distribution of arrivals especially in situations in which there is wide spacing between traffic signals or an intermittent passenger service stop or both. When the spacing is short and the bus movement is not interrupted by a passenger-service stop, buses tend to arrive predominantly during the green phase. These field checks were not conclusive. They indicated the need to further investigate the assumptions underlying this methodology and to validate the methodology by further experimentations with the application of bus-actuated signal preemption.

CONCLUSIONS AND RECOMMENDATIONS

Previous research on the bus-actuated, signal-preemption system has concentrated on proving the feasibility of a particular strategy. The feasibility was established by measuring whether a significant decrease in the bus-travel time and the number of stops made for traffic signals occurred in a demonstration or simulation test. This measurement then proved that under the conditions existing at the demonstration site the preemption system was effective. However, no generalizations have been drawn from these experiments, and planners have had no assurance that the system could be successfully installed in particular geographic locations.

In this paper bus-signal preemption is evaluated on an intersection-by-intersection basis and manually calculated, single-intersection results are provided. The methodology development and the results of the test application of the methodology have led to four significant conclusions:

1. By examining the operations of signal-preemption systems, we may derive general equations that describe the savings and losses from preemption;
2. Intersections can be equipped with a dual-detector, green-extension scheme without requiring areawide, computerized traffic control systems;
3. As a result of the modest equipment cost and the high efficiency of green extension, revenue-cost ratios as high as 20:1 are possible and even single location systems can be justified; and
4. Bus-actuated, signal-preemption systems can increase the economic efficiency of an intersection.

The following areas seem most fruitful for the application of bus-actuated, signal-preemption systems.

1. Additional field checks and experiments should be conducted to test the assumptions presented in this report and thus verify or modify this methodology as warranted.
2. The methodology presented here should be expanded and modified to include the full range of preemption strategies. European experiments sometimes use manipulation of the cycle rather than alteration. Schemes such as compensating cycles or double green cycles are being tested and should be treated in subsequent methodologies.
3. Equipment involved in these preemption strategies should be further developed to lower the costs involved in their use. Further research and development in the technology of vehicle detection or identification, such as the federally funded Passive Bus Detector/Intersection Priority System, should be encouraged.
4. Research should be done to determine whether priority can be given to buses and emergency vehicles by using different preemption techniques for each but the same equipment.
5. The interaction of preemption systems with other bus-priority measures should be investigated.

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Estimation of Delay at Traffic-Actuated Signals

Kenneth G. Courage, University of Florida
Paraskevi Papapanou, Rensselaer Polytechnic Institute

Field measurement of delay at traffic signals is a costly and cumbersome process, and the use of analytical models to estimate delay is, therefore, of interest to the traffic engineer. A model originally developed by Webster has gained widespread use and acceptance in the estimation of delay at pretimed signals where signal timing remains constant from cycle to cycle. The original version of this model has been modified for application to traffic-actuated signals where signal timing is determined on the basis of vehicle presence information received from detectors in the roadway. This paper describes the modifications to Webster's model, which consist primarily of the substitution of values in the second (random arrival) term based on maximum cycle length rather than on optimal or average cycle lengths. The delay calculations that result from the modified version are compared with the values for pretimed operation based on the original model. Both versions of the model are compared with a simulation model and found to produce satisfactory approximations. Delay under traffic-actuated control is lower than delay under pretimed control. The difference depends on the degree of saturation of the approach lanes. The maximum difference is observed at 75 percent saturation. No difference is observed at very low saturation levels because very little delay accrues under these conditions. The difference also approaches zero at very high saturation levels because the actuated controller becomes constrained by the maximum interval timer to operate in a pretimed mode.

Delay is well recognized by the traffic engineer as a useful measure of effectiveness in a traffic-control system. Motorists view traffic delay with great disfavor, and economists agree that delay in movement of traffic is costly. Estimation of delay is, therefore, an important topic in the analysis of transportation systems.

Delay may be estimated either by field measurement or by analytical or simulation models. Although field measurement produces the most accurate results, the procedures are somewhat costly and time consuming. Furthermore, field measurement techniques cannot be applied to hypothetical situations such as proposed signal installations. Analytical approximations are, therefore, of interest to the traffic engineer.

The best recognized analytical treatment of delay estimation has been performed by Webster (1, 2). Webster demonstrates that satisfactory delay estimates may be obtained for any signalized approach when one is given the traffic volume, capacity, and signal timing (cycle length and effective green time) for that approach. The analytical process becomes, however, substantially more complicated when the signal timing varies with demand as in the case of traffic-actuated signals. A complex stochastic queuing model evolves from this analytical process, and this complex model is not adaptable to a practical solution because of the simplifying assumptions that must be made. The purpose of this paper, therefore, is to examine an analytical model that can be used to produce a useful approximation of delay at intersections where fixed signal timing does not exist. This examination is accomplished by refining Webster's model for pretimed control rather than by developing a separate, theoretical model. This refinement technique is further investigated by simulation to determine whether the techniques can be applied in a practical sense to estimate delay at vehicle-actuated signals.

WEBSTER'S PRETIMED DELAY MODEL

Webster demonstrates (1) that delay at pretimed signals may be approximated by the sum of two separate components.

1. The component due to uniform vehicle arrivals may be derived analytically in the form

$$D_1 = [C(1 - \lambda)^2]/[2(1 - x)] \quad (1)$$

where

D_1 = delay per vehicle, seconds,
C = cycle length, seconds,
λ = proportion of green time given to the approach, and
x = degree of saturation of the approach, volume/capacity.

This component expresses the delay that would be experienced if the traffic stream were composed of equally spaced vehicles that arrive in a uniform manner.

2. The component due to random arrivals was developed semiempirically in the form

$$D_2 = x^2/[2q(1 - x)] \tag{2}$$

where

D_2 = delay per vehicle, seconds, and
q = flow on the approach, vehicle per second.

This component expresses the additional delay that results from the random-arrival characteristics of the traffic stream.

The total delay per vehicle may be expressed as

$$D = 0.9/(D_1 + D_2) \tag{3}$$

where the value of 0.9 is an empirical correction factor. The D_1 and D_2 terms are commonly referred to as Webster's first and second terms respectively.

APPLICATION TO TRAFFIC-ACTUATED CONTROL

For purposes of this analysis, the control strategy is assumed to:

1. Distribute available green time in proportion to demand on critical approaches and
2. Minimize wasted time by terminating each green interval as soon as the queue of vehicles has been properly serviced.

This control strategy closely approximates the operation of the traditional traffic-actuated controller that has been properly timed. The delay estimates will, therefore, reflect the best operation that can be expected from traffic-actuated control. Inappropriate setting of operating parameters (initial interval, extension interval, and so forth) will degrade performance of the controller.

Delay will be lower under traffic-actuated control than under pretimed control throughout most of the volume/capacity (v/c) range for two reasons.

1. Cycle length will tend to be shorter under traffic-actuated control since individual phases will be terminated as soon as queues are serviced.
2. Cycle failures will be fewer in which termination of green signal before a queue is completely serviced causes extra delay to waiting vehicles.

Both of these factors must be taken into consideration in the development of a model for estimating delay at actuated signals. The question of cycle length is addressed by Webster, who derives the optimal cycle for pretimed operation as

$$C_o = (1.5L + 5)/(1.0 - Y) \tag{4}$$

where

C_o = optimal pretimed cycle for minimum delay,
L = sum of all lost times due to starting and stopping critical movements on each cycle, and
Y = overall degree of saturation of critical movements (i.e., the proportion of green time required for the movement of traffic).

For traffic-actuated operation, the appropriate cycle length is the average cycle length that will ensure that all of the excess time (beyond that which is needed for the movement of traffic) is dissipated in the starting and stopping process. The proportion of excess time available may be determined as 1.0 - Y, where Y is the proportion of time required. Therefore, the average cycle length may be expressed as a single ratio of the starting and stopping time to the proportion of time available for starting and stopping or

$$C_a = L/1.0 - Y \tag{5}$$

where C_a is the average cycle length. The optimal cycle length for pretimed operation will, therefore, always be higher than the average cycle length under actuated operation. The extra time allocated to C_o will appear as slack time, necessary to provide for stochastic variation in the number of vehicles that must be serviced on each cycle. This slack time will reduce the efficiency of the operation and result in an increased delay.

The question of cycle failures is addressed in Webster's second term, which takes into account the probability of a given phase being terminated before the queue is serviced. This probability is much lower under traffic-actuated control because the termination of the phase is initiated by the satisfaction of the queue. In fact, premature termination should only occur when the preset maximum green time is reached.

A reasonable approximation of delay under traffic-actuated operation should, therefore, be achieved by assigning a maximum cycle length to the operation and by basing the values used in Webster's second term on the maximum cycle length rather than on the optimal or average cycle lengths. This procedure will lower the estimated delay by increasing the effective green time used in the second term.

Based on this analysis we expect that under low to moderate volumes the delay caused by a vehicle-actuated signal will be lower than the delay caused by a pretimed signal. This lower volume can be explained by the fact that, when volumes are low to moderate, the signal responds to demand and does not allow slack time between phases or queues at the end of green. When volumes increase, however, we expect that the actuated signal will often operate under its maximum time settings and, when the volumes reach the saturation level, the operation of a vehicle-actuated signal will not differ from a pretimed signal because the signal will be continuously operating under maximum settings.

The following table illustrates the use of different cycle lengths for the two types of signal control.

Type of Signal	Cycle Used in First Term	Cycle Used in Second Term
Pretimed	Optimum	Optimum
Actuated	Average	Maximum

The first term (delay due to uniform arrivals) gives approximately the same delay for both types of control when the cycle length of the pretimed signal is equal to the average cycle length of the actuated signal. In such cases, the delay between pretimed and actuated signals is caused by the randomness of arrivals (delay expressed by second term). In actuated signals, small demand fluctuations do not cause as much random delay as in pretimed operation because the green times can be extended until demand is satisfied. If, however, these fluctuations cause the green to be extended to its maximum without satisfying the demand, then the benefits of the actuated operation no longer exist.

The solution to the problem of minimizing delay is, therefore, long cycles to accommodate random fluctuations (minimize random delay) and short cycles to accommodate regular demand (minimize uniform delay). This solution can only be applied to the vehicle-actuated signals, but in pretimed signals a compromise between average and maximum cycle can be made. The results are as expected: As long as there is a difference between average and maximum cycle length, actuated signals will result in less delay; but, when cycle lengths

Figure 1. Effect of maximum cycle length on intersection delay under different volume conditions.

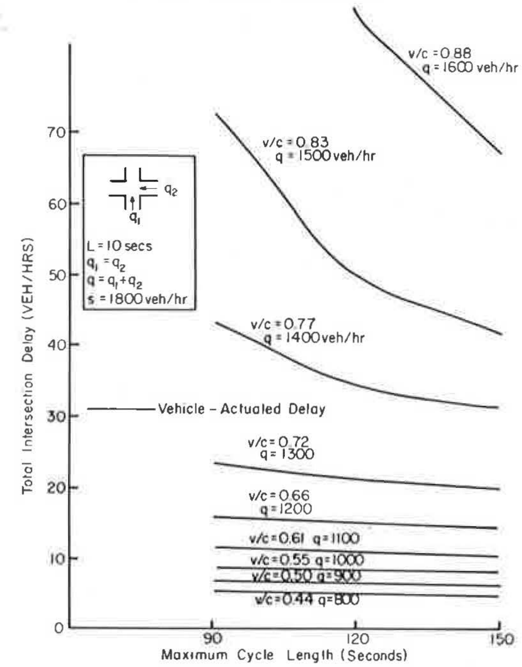
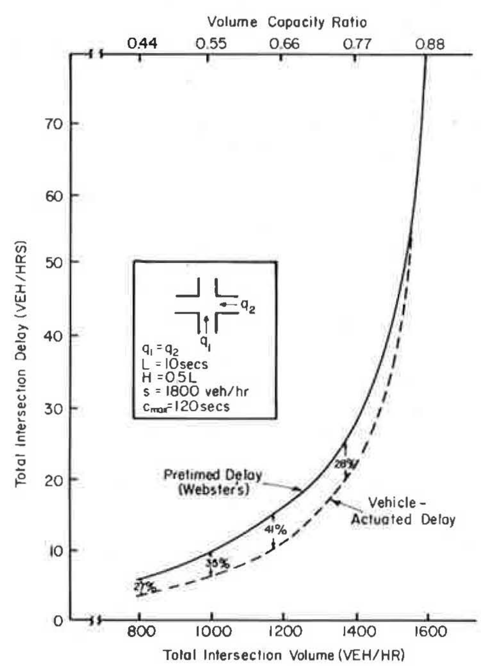


Figure 2. Relationship between pretimed delay and vehicle-actuated delay at an intersection with equal volumes at each approach under different volume conditions.



become equal, the resulting delays are the same for both types of control. Figure 1 demonstrates the variation of vehicle-actuated delay for maximum cycle lengths in the range of 90 to 150 s. Total intersection volumes from 800 to 1600 were considered, and corresponding intersection delays were calculated by using the model described in the table. Figure 1 shows that delay at a vehicle-actuated signal is less dependent on maximum cycle length when volumes are low to moderate (v/c ratio from 0.44 to 0.72). The maximum cycle length, however, becomes increasingly significant at higher volumes (v/c ratio higher than 0.75). At low volumes the maximum cycle length is rarely reached and, therefore, the random delay is very small. When the volumes increase, however, the maximum cycle length is reached more often, and the random delay increases significantly. Under these conditions the maximum cycle length becomes the actual operating cycle instead of simply a limiting condition.

Figure 2 illustrates the variation of delay for pretimed and vehicle-actuated signals for a range of total intersection volumes from 800 to 1600 vehicles/h. The delay for pretimed signals was calculated by using Webster's delay model, but the delay for actuated signals was calculated by using the modified version. Figure 2

Figure 3. Relative and absolute benefits of vehicle-actuated signal control over pretimed signal control to v/c ratio.

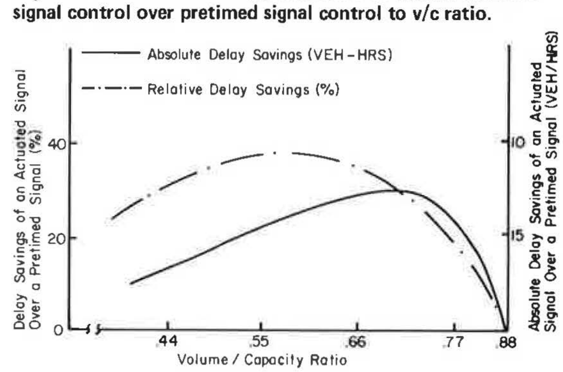


Figure 4. Comparison of simulation model and Webster model using equal approach volumes.

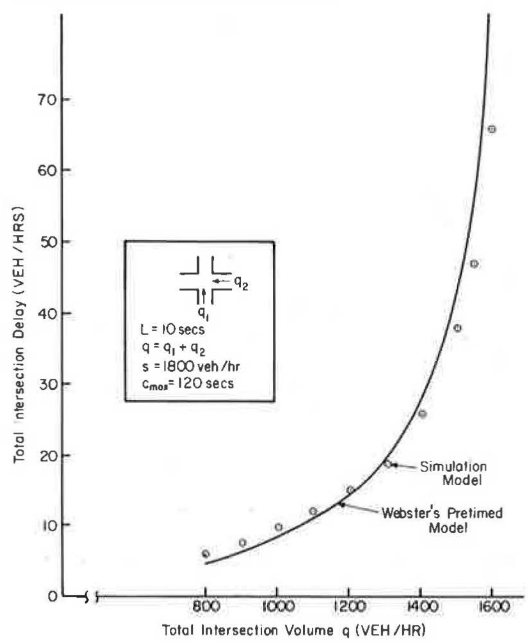


Figure 5. Comparison of simulation model and Webster model using unequal approach volumes.

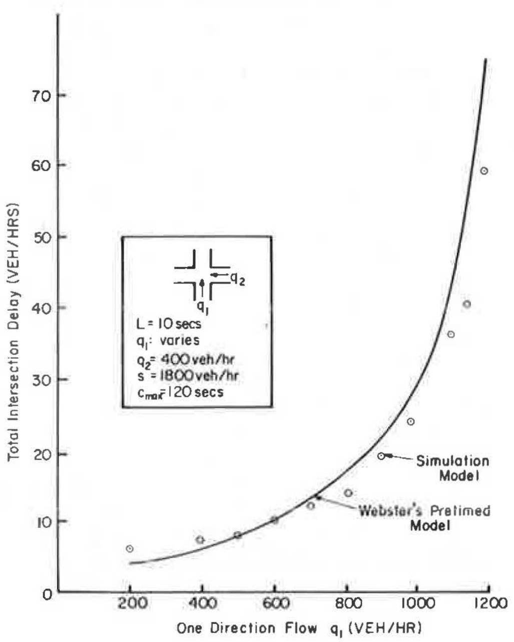
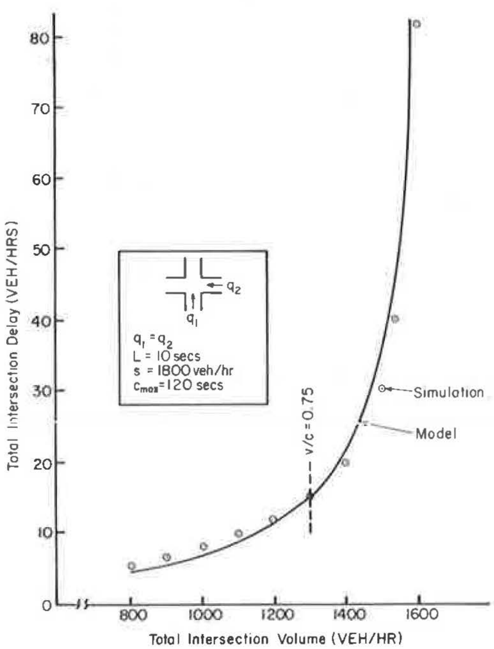


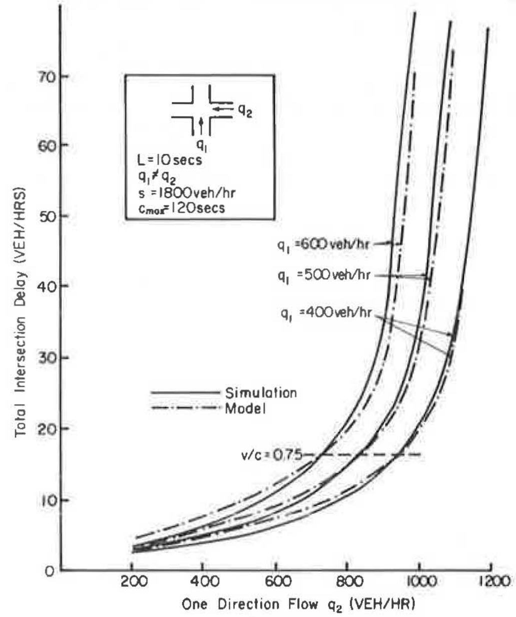
Figure 6. Comparison of simulation model with vehicle-actuated model using equal approach volumes.



shows that the delay savings due to actuated signal control are small in low volumes and keep increasing up to a maximum savings of 41 percent at 1200 vehicles/h and v/c ratio of 0.66. After this point the savings start decreasing until they become zero at 1600 vehicles/h and v/c ratio of 0.88. In this particular example, for a v/c ratio of 0.44 and of 0.88, the delay savings under vehicle-actuated control lie within 27 to 41 percent and have an average equal to 26 percent.

Figure 3 shows the delay savings for this example plotted as a function of the v/c ratio. From Figure 3 one can estimate that under low to moderate volumes there is an average savings of 34 percent when compared

Figure 7. Comparison of delay model results with simulation results using unequal approach volumes.



with pretimed delay. This percentage, however, drops sharply after the v/c ratio of 0.66 and becomes zero at a ratio equal to 0.88. In addition, in Figure 3 the absolute delay savings are also plotted as a function of the v/c ratio. Here the maximum absolute delay benefits occur at a v/c ratio of 0.77 although the maximum relative delay savings occur at a v/c ratio of 0.66. This change in savings happens because the delays are higher at a v/c ratio of 0.77 and, therefore, the absolute benefits are higher also. After this point, the absolute benefits drop sharply and become zero at a v/c ratio of 0.88.

MODEL VALIDATION

In the development of the delay model for vehicle-actuated signals, several assumptions and approximations were made. We felt, therefore, that the model should be tested under various conditions to investigate the model's validity and applicability to realistic situations. This testing was accomplished by exercising the model under various volume levels and comparing these results with the result produced by a simulation model for the same volume levels.

The simulation model used here consists of two submodels: the intersection simulator and the traffic signal emulator. The intersection simulator generates the vehicles in the system and records system variables such as length of queue and time in queue. The emulator superimposes either the pretimed or the vehicle-actuated traffic-signal operation.

The intersection simulator generates arrivals according to a Poisson distribution and, depending on the status of the signal given by the emulator, allows arrivals to stop or depart.

The simulator scans the system every second, records the new arrivals and departures, calculates the number of vehicles in the queue in each approach over the entire simulation period, and provides the total intersection delay for the given period.

Two kinds of traffic signal emulators were used: pretimed and vehicle-actuated. The pretimed signal emulator simulates a pretimed signal that displays green, amber, or red at fixed intervals; however, the vehicle-actuated signal emulator allocates right-of-way in the same manner as a traditional, actuated controller.

The simulation model is based on a four-legged intersection of two one-way streets. The green times, cycle lengths, and other inputs to the pretimed emulator were calculated by using Webster's method.

The validation proceeded according to the following strategy. First, the simulation model was tested against Webster's pretimed delay model by using a pretimed signal emulator. Because Webster's delay model has gained widespread use and acceptance, comparison of the simulation with Webster's model should provide sufficient evidence of the validity of the simulation model. The actuated signal delay model was then tested against the validated simulation model under various conditions.

Delays at a pretimed signal were calculated for a range of total intersection volumes from 800 to 1600 vehicles/h (the intersection becomes oversaturated after this point) and equal volumes in both directions. Simulation was performed for the same volume ranges; the results are plotted in Figure 4. Figure 4 shows that under the entire range of volumes the delays obtained by simulation are very close to the delays obtained by the delay model. The simulation model was also tested against Webster's model for unequal volumes in two directions, and the results are shown in Figure 5. Again, the results demonstrate trends that are similar to the case of equal volumes in the two directions. Based on these two comparisons, we concluded that the simulation model is successful in reproducing the delay estimates provided by Webster's pretimed model and is, therefore, a useful tool for validating the modified version for traffic-actuated operation.

In validating the modified version, the model was first tested with equal volumes in both directions. The results are plotted in Figure 6. Figure 6 shows that the model results are very close to the simulation results. The model tends to underestimate the delay slightly under low volumes and to overestimate slightly under heavy volumes. The average difference, however, lies within a 10 to 15 percent range and is reduced to zero when the total intersection volume is approximately 1350 vehicles/h and the v/c ratio is 0.75.

Another series of simulation runs was performed to test the model for different volumes in two directions. Figure 7 illustrates the results of these runs for various volume levels. For each curve, vehicles per hour in one direction is shown. Vehicles per hour in the other direction ranged from 200 to 1200.

Similar trends between simulated and computed delays can be distinguished in the three sets of curves. The computed delays are lower than simulated delays at low to moderate volumes (v/c ratios from 0.44 to 0.74) by an average difference of approximately 10 percent and become equal when the total intersection volume is equal to 1350 vehicles/h and v/c ratio is 0.75. After this point, the computed delays become slightly higher than the simulated delays. The average difference is 1 percent for the 400-vehicles/h curves and 8 percent for the 600-vehicles/h curve.

SUMMARY AND CONCLUSIONS

In this paper a macroscopic model for estimating delays at vehicle-actuated signals was proposed. The model was tested by simulation and has given satisfactory results for a wide range of applications. The model is a simple, yet adequate, model that requires little computational effort even for a complex, multiphase signal operation. Based on the same principles as the most widely accepted model for estimating delay at pretimed signals, this macroscopic model is offered as a useful tool for a quantitative comparison of the two basic types of signal control.

ACKNOWLEDGMENTS

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Abridgment

Cost-Effectiveness of RUNCOST Evaluation Procedure

Peter S. Parsonson, Georgia Institute of Technology

Currently there is emphasis on low-capital programs of transportation-system management (TSM). Regulations issued in 1975 by the Urban Mass Transportation Administration and the Federal Highway Administration require each urbanized area to develop a plan containing a TSM element and a transportation improvement program (TIP). The programs are designed to meet the short-range needs of urban areas through the efficient use of existing facilities. The goal is to reduce traffic congestion and to facilitate the flow of traffic. According to the regulations (1), one of the major categories of TSM action concerns the "efficient use of existing road space through traffic operations improvements to manage and control the flow of motor vehicles." Better signalization and the progressive timing of traffic signals are examples of such TSM action and are the general focus of this paper.

Methodology of before-and-after evaluation of signal improvements is not standardized. The list of candidate measures of effectiveness is long (2), and only a fraction of the measures is included in the typical evaluation. Once the measures of effectiveness (MOE) are selected for a project, a wide range of choice remains in scope and depth of field data collection and office analysis. One available option is the use of the computer program RUNCOST to quantify before-and-after values of vehicle-operating cost, fuel consumption, and pollutant emission. The value of these MOE must be compared with the added expense of using RUNCOST.

The objective of this paper is to indicate the cost and effectiveness of using the RUNCOST procedure as a component of the evaluation methodology for traffic-signal systems.

The availability of the computer program RUNCOST was reported in 1972 (3). RUNCOST was written by the Federal Highway Administration (FHWA) in 1970 to eliminate the tedious process of manually entering the Winfrey tables of vehicle operating cost (4). The Winfrey tables give the cost per vehicle-mile to operate an automobile and four types of trucks at uniform speeds ranging from 0 to 80 mph. The Winfrey tables also provide the additional costs of accelerating or decelerating these vehicles through speed-change cycles. Operating costs include the expense of fuel, tires, engine, oil, maintenance, and depreciation.

The floating automobile must be equipped with a tachograph for the Winfrey tables to be used to full advantage. The tachograph charts are curves of speed versus time. The curves for selected floating-automobile runs are reduced in the office to series of coordinates of speed and time that are punched onto computer cards. The principal RUNCOST output for each run is the Winfrey cost (in cents) to operate each of the five types of vehicles according to speeds and speed changes recorded on the tachograph chart.

In addition to the Winfrey tables, the RUNCOST program also computes the dollar value of time of the run for each of the five vehicle types based on dollar values of time specified by the user. The program uses only the total time of the run and not the tachograph curve of speeds versus time. (Therefore time costs can be obtained from floating-automobile studies employing only stopwatches and manual calculations. Time costs are reported by RUNCOST as a useful by-product but are not the justification for using a tachograph and the RUNCOST program.)

The first application of RUNCOST to the evaluation of a signal system was reported by Chapman and Clark (5) for the Charleston, South Carolina, hybrid traffic control system (HTCS). The HTCS is a 90-intersection, grid signal system with a control center that unites analog and digital computers. Raynor (6) reported the characteristics of the system in 1970. The Chapman and Clark RUNCOST evaluation indicated annual network savings of \$631 000 due to reduced operating costs. These savings amortized the \$550 000 cost of the HTCS in less than 1 year. (If the annual network savings of \$4 676 000 due to reduced time costs had been considered as well, the amortization period would have been found to be approximately 1 month.)

By comparison, an earlier evaluation by Chapman and Raynor (7) that did not use RUNCOST found that the amortization period lasted approximately 3 years. That manual evaluation considered only the reduction in stops and determined through macroscopic analysis the savings in vehicle-operating costs and also stopped-delay-time costs that were due to the reduction in stops.

The RUNCOST analysis was much more microscopic and comprehensive in its treatment of speed changes. Chapman and Clark concluded that the reduction in amortization period from 3 years by manual analysis to less than 1 year by RUNCOST is significant and that "the refinement provided by the RUNCOST program is justified in spite of extra effort and cost" (5).

In 1974 the RUNCOST program was expanded to include printout of fuel consumption and pollutant emission. The fuel consumption calculation is based on the Winfrey tables (4). The calculation permits the user to apply an adjustment factor to each of the five types of vehicles to account for changes in fuel consumption rates. The pollutant emissions are calculated separately for nitrogen oxides, hydrocarbons, and carbon monoxide for each of the five types of vehicles and are based on previous work by Curry and Anderson (8, p. 103, Table B-6). The emissions loaded into the program are for the 1968-1969 base vehicle. The user is permitted to apply a single emission adjustment factor to all types of emissions to account for changes since 1968-1969. Curry and Anderson (8) give some guidance in this regard. The program considers emissions at uniform speeds and for stops, but not emissions due to speed changes other than stops. A sample output of the expanded RUNCOST program is included as an appendix.

APPLICATION OF RUNCOST TO NORTHSIDE DRIVE

During 1973 to 1975 the city of Atlanta replaced 21 old, noninterconnected, volume-density controllers along an 11.65-km (4.5-mile) length of Northside Drive with new, actuated controllers that are interconnected and supervised by a digital computer at City Hall.

In 1975 the expanded RUNCOST program was applied to the evaluation of that signal-system improvement. The MOE for the evaluation included the following:

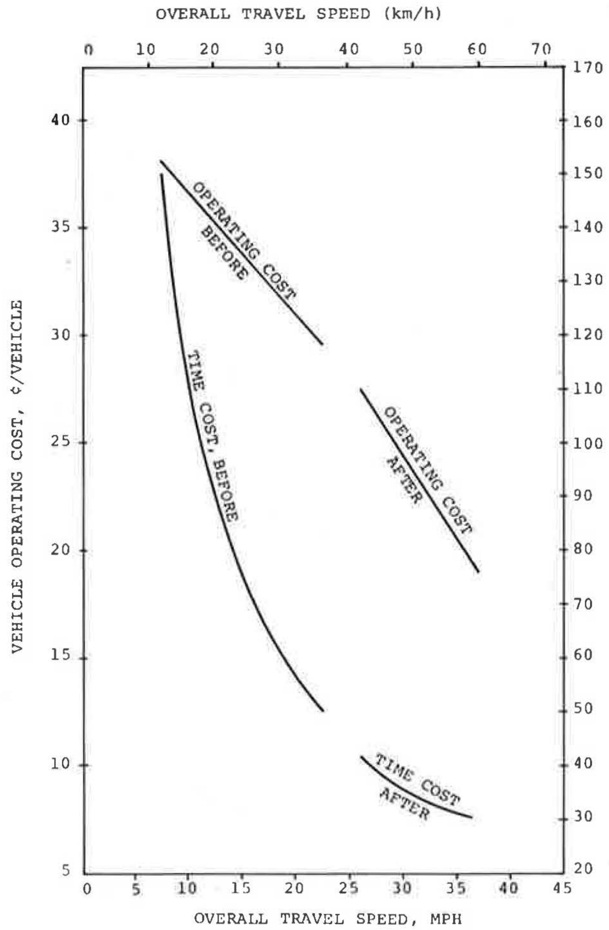
1. Level of service, A to F;
2. Stop probability, percent;
3. Average overall travel speed, kilometers per hour;
4. Vehicle operating cost, dollars;
5. Time cost, dollars;
6. Fuel consumption, liters;
7. Pollutant emission, kilograms; and
8. Volume, vehicles per day.

The results of the evaluation are reported in other studies (9, 10). A summary is given below.

Procedures and Costs

RUNCOST requires that the floating automobile be equipped with a tachograph. Montroll and Potts (11) and Parsonson (3) have reported the use of an instrument that has a 24-min by 7-revolution clock and a speed range of 0 to 129 km/h (0 to 80 mph). This instrument is a standard truck tachograph that has been modified slightly to give the expanded time scale required for traffic studies. The brake-signaler device is connected to a dash-mounted pushbutton for use as an event recorder. The instrument costs only a few hundred dollars, is quite reliable, and can be installed, serviced, and calibrated by most speedometer shops. Although the tachograph can be used for runs on short arterials without any peripheral equipment, usually including an inexpensive, dash-mounted, digital clock that reads

Figure 1. Operating cost and time cost versus speed during peak periods for zone 1.



seconds is desirable; including a portable tape recorder is also desirable. Both of these units operate from a dc-to-ac converter connected to the vehicle's battery. The cost of the equipment, installed, totals approximately $500. The driver can easily operate the equipment under all traffic conditions; an observer is never needed.

The office procedures begin with the labeling of all event marks on the tachographs. These event marks are the signalized intersections and, for short arterials, can be labeled directly. For longer runs the tape recording can be played back to eliminate the possibility of error in event-mark identification. At this point the stops are recorded on office forms. Then the average speed of each run is calculated manually by using travel times obtained either from the tachograph or from the tape recorder.

Only a few floating-automobile runs are selected for RUNCOST computer analysis. These runs should cover the range in observed speeds and the variation in vehicle-type distribution at different times of day. In the simplest case the vehicle population in a control section is observed to have approximately a constant distribution among the five types throughout the day. Then the only variation is in speed, and three runs in each direction of travel can be selected to cover the range encountered. Therefore the RUNCOST computer analysis would involve only six floating-automobile runs for the before condition and six for the after, for a total of 12 runs.

Figure 1 includes a relationship between vehicle-operating cost and overall travel speed. The data points that determined this relationship show some scatter from run to run to be about the best line. In view of this scatter, the recommendation for three runs in each direction might seem inadequate. However, Atlanta experience to date with four arterial sections indicates that the best line does not vary significantly by direction nor does the best line vary much from arterial to arterial (when plotted as operating cost per vehicle-kilometer of travel). The suggested total of 12 points is usually adequate to determine the relationship. As data accumulate from several projects, the need for additional RUNCOST analyses decreases.

Because a one-way, floating-automobile run in a single-control section is typically no more than 10 min in duration, the 12 runs represent 2 h of field data.

The coding of a run is the translation of the tachograph curve into an equivalent series of coordinates of speed and time. To code 1 h of field data, 2 to 3 h of clerical time are required. An additional ½ to 1 h is required to punch these cards and the associated control cards. Therefore the 2 h of field data from a single, simple-control section will require a maximum of 1 person-d for coding and punching.

By comparison, the Northside Drive evaluation was complicated by the fact that the vehicle-type distribution varied substantially between the peak periods and the midday off-peak periods. Also, there were three control sections. In all, 68 floating-automobile runs were selected for RUNCOST analyses. They represented 10 h of field data. Coding and punching of the cards required 5 d of clerical or subprofessional time. Computer charges average approximately $1.00/h of field data analyzed.

Results of the Evaluation

The three control zones combined showed an improvement in overall travel speeds of over 14.5 km/h (9 mph), quite constant throughout the day. The probability of having to stop at a given intersection along Northside Drive was reduced to only one-third to one-fourth of its before levels.

Figure 1 proceeds from the RUNCOST printouts and effectively demonstrates the reduction in vehicle-operating cost and motorist-time cost that is produced by a signal improvement that increases vehicle speeds. The absence of overlap between the before and after curves indicates that, during peak traffic periods in this control section, the worst traffic condition observed after the signal improvement was better than the best condition observed before signal improvement. The control section of Figure 1, designated as zone 1, is 3.65 km (2.27 miles) in length.

The time costs in Figure 1 were computed by RUNCOST (9). The adopted values for automobiles attempt to take into account the purpose of the trip. Commuter-period trips were assigned a value of $4.80/vehicle/h, off-peak trips during working hours were assigned a value of $3.20/vehicle/h, and leisure-time trips were assigned a value of $1.60/vehicle/h. Values of time to commercial vehicles ranged from $4.66 to $7.77/vehicle/h depending on the size of the truck.

The calculation of operating costs was the principal justification for developing RUNCOST. Time costs are merely a useful by-product and can be obtained without using RUNCOST. If operating costs were very small compared to time costs, there would be little incentive to use RUNCOST. Therefore, the cost-effectiveness of RUNCOST depends in part on the value of time per vehicle per hour adopted for the study. If, for example, values lower than the $4.80, $3.20, and $1.60 reported in the preceding paragraph were to be adopted, then operating cost would become a larger proportion of the total benefit. The justification for using RUNCOST would increase. The lower the assigned value of time is, the more cost-effective RUNCOST becomes.

The Northside Drive evaluation was conducted during a period of rapidly rising gasoline prices. The RUNCOST program permits the user to apply an inflation factor to adjust the operating costs from the 1969 base level to the current level. A study of the prices of gasoline and automobiles in 1969 and 1975 yielded an inflation factor of 1.32 for Northside Drive. This figure was applied not only to the RUNCOST analyses of the after study performed in 1975 but also to the analyses of the before study in 1973. (If there had been no signal improvement, the before traffic conditions of 1973 would have continued to 1975 and would be paid for by motorists at 1975 prices.)

Figure 1 is derived from floating-automobile runs performed in the through lanes. Left-turn lanes and cross-street approaches at key intersections were investigated as well by using stopped-time-delay studies following the Berry-Vantil method. These supplemental studies showed that the selected approaches also experienced a reduction in costs of operation and time. The savings to through-traffic motorists and to motorists on the selected approaches were summed over a typical week. The weekly savings in vehicle-operating costs amounted to \$18 000 or \$919 000 annually. The time-cost savings were about twice as large: \$36 000/week or \$1 833 000/year.

The cost of the Northside Drive signal project was \$277 500. The annual savings in vehicle-operating costs alone, amounting to \$919 000, are sufficient to repay in triple the project cost to the motorists in the course of a year. If the dollar value of time savings is also considered, the total annual benefits of \$2 802 000 amortized the project cost in approximately 1 month.

The RUNCOST analysis also showed that fuel consumption was reduced by 22 percent, a saving of 284 million liters/year (0.75 million gal/year). Pollutant emission was reduced by 38 percent, which amounts to 0.5 kg (1.1 million lb)/year.

SUMMARY AND CONCLUSIONS

1. The original version of the RUNCOST program was developed principally to determine vehicle-operating costs. Time costs are reported as a useful by-product but can be obtained without RUNCOST. Therefore, the lower the assigned value of time per vehicle per hour is, the greater the relative importance of operating costs are and the greater the cost-effectiveness of using RUNCOST is.
2. RUNCOST was first applied to the evaluation of the Charleston, South Carolina, signal-system project. In view of the microscopic and comprehensive treatment of speed changes by RUNCOST, the evaluators concluded that the use of RUNCOST was cost-effective.
3. RUNCOST has since been expanded to include the printout of fuel consumption and pollutant emission. The expanded program was first applied to the evaluation of the Northside Drive signal system in Atlanta. Using the relatively high current prices for gasoline and automobiles and the hourly values of time stated above, we found that the saving in vehicle-operating cost was 50 percent as large as the saving in time cost. Therefore operating-cost savings were not at all insignificant as compared to time-cost savings. When we consider that vehicle-operating costs are much more tangible than motorist time costs, we can readily conclude that the operating-cost capability of RUNCOST is highly effective.
4. RUNCOST was effective in determining the reduction in fuel consumption and pollutant emission for Northside Drive.
5. RUNCOST requires an equipment outlay of approximately \$500. No more than 1 person-d of office work at the clerical level would be added to the simplest signal project. Northside Drive, with its three control sections and other complications, required 5 person-d. This office cost amounted to only 5 percent of the evaluation budget. Computer charges are insignificant.
6. These findings indicate that the RUNCOST evaluation procedure is highly cost-effective and should be considered for inclusion in any floating-automobile study of traffic-signal-system improvement.

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Methods for Field Evaluation of Roadway-Delineation Treatments

W. A. Stimpson and W. K. Kittelson, Alan M. Voorhees and Associates, Inc., McLean, Virginia
W. D. Berg, University of Wisconsin

The objective of this study was to establish the relationships between various traffic-performance measures and accident probability on two-lane rural roads. This information would enable researchers to evaluate new roadway-delineation treatments without having to collect before-and-after accident data over a period of many years. Accident data and speed and lateral-placement measures were collected for typical roadway sections of both tangent and curvilinear alignment. Multiple linear regression analysis was then used to develop an accident-probability model. Important influencing variables were centrality of vehicle placement within the traveled lane, difference in lateral placement along the roadway section, skewness of the speed distribution, pavement width, and shoulder width. Procedures for the field evaluation of delineation-treatment effectiveness were then specified.

Roadway-delineation practices have developed over the years primarily as a result of field experience and limited subjective evaluation by engineering and maintenance personnel. Relatively few in-depth studies have been conducted, and most have dealt with limited aspects of specific delineation treatments. For instance, new devices such as raised pavement markers have undergone intensive material and maintenance testing, but their best use as part of an overall system for roadway delineation has received insufficient attention. Some before-and-after accident studies have been made, but they have required lengthy time periods to conduct; also, these studies have addressed either isolated spot-location problems or extensive distances of diverse highway features that make proving true cause-and-effect relationships difficult. Limited diagnostic field tests have been run by using teams of engineers, police, and lay drivers, but the results have been difficult to reconcile with serious questions of cost-effectiveness. Also difficult to interpret, especially for the policy maker, are the findings of studies that have used traffic-performance measures as basis for evaluation. A valid issue arises over whether certain statistically significant changes in speed and lateral-placement data are of any practical consequence.

In an effort to address these problems, the Federal Highway Administration (FHWA) recently initiated a two-phase delineation research project (1). The objective of the first phase was to establish the relationship between various traffic-performance measures and accident probability on two-lane rural roads. This information would permit traffic engineers to evaluate new delineation treatments without having to collect before-and-after accident data over a period of many years. The objective of the second phase was to apply the methodology to the field evaluation of potentially more cost-effective delineation treatments. This paper summarizes the results of the first phase of this research.

EXPERIMENTAL DESIGN

The scope of the research study was confined to application of delineation treatments for three general types of roadway characterized by type of alignment described below. In each case, the roadway was to have two lanes, have an average daily traffic (ADT) volume of fewer than 500 vehicles, and be located in a rural environment.

Type of Alignment	Description
Tangent	Predominantly straight, horizontal curves of 3° or less, more than 5 km (3 miles) long and desirably 16 km (10 miles) long
Winding	Predominantly curved, curves greater than 3° and tangents less than 457 m (1500 ft) between curves, more than 5 km (3 miles) long and desirably 16 km (10 miles) long
Isolated horizontal curve	More tangent than winding, curve greater than 3° and desirably isolated from other curves by 0.8 km (0.5 mile) or more

Given the delineation situations to be modeled, we had to develop appropriate traffic-performance measures. Although vehicular speed and lateral placement were expected to be among the primary raw measures, the proper formulation of these and other possibilities warranted a systematic investigation. The first step in this investigation was a review of published literature for known accident relationships.

Previous research had shown that type of delineation can influence speed and lateral placement; however, establishing a statistically significant relationship with accident experience had been difficult (2, 3, 4, 5, 6). Studies clearly showed that complex interactions occur among highway geometrics, delineation treatments, environmental conditions, traffic-performance measures, and accident experience. For example, horizontal alignment, lane width, and delineation might relate directly to the number of excursions from the proper lane, but the expected accident rate would also be affected by the lateral distance available for recovery described, in large part, by the width of the shoulder or the opposing lane.

In the context of this study, a traffic-performance measure was defined as any measurable parameter that describes the flow of traffic at a point or over a section of two-lane highway. These measures can take the form of various statistics such as mean, variance, skewness, or percentile. The objective of this study was to develop models that relate accident rate to traffic-performance measures for three general geometric situations. Critical to the model development was the collection of data for those traffic-performance measures most likely to be related to accidents and at the most appropriate locations along the test section. To supplement engineering judgment, a selection methodology using the information-decision-action (IDA) sequence file and an accident-prior-movement (APM) analysis was applied (2).

For a specific geometric situation, the IDA analysis defines the desired driver action, determines the decision necessary to effect these actions, and then specifies the information needed by the driver to make the required decision. The most useful elements of the IDA analysis for its application to this study were the actions required by the driver to properly negotiate a particular situation. These actions could be translated into traffic-performance measures.

The APM approach to identifying appropriate traffic-performance measures for a given situation was to define the possible accident types that can occur and determine possible vehicle movements that could have preceded each type of accident. Traffic-performance measures could then be chosen to describe or quantify those prior movements.

Traffic-Performance Measures for Tangent Roadways

A tangent section can be categorized as a steady-state situation. A steady-state situation means that a driver's task requirements are limited to maintaining continuous adjustive control, both lateral and longitudinal. Except for transitional situations that arise, such as at intersections or during passing maneuvers, an IDA model for the rural-tangent section is characterized by a lack of change. Only three control actions are required of the driver on a tangent. Speed should be maintained, position in lane should be maintained, and a reasonable distance from the vehicle in front should be maintained. The traffic-performance measures that numerically describe these actions are speed-profile statistics, lateral placement (including the frequency of shoulder and centerline encroachments), and headway.

For the purposes of the APM analysis, four basic accident types are likely to occur on a two-lane tangent section without any intersections or other situations that would require a change in the driving task. These types include head-on and side-swipe accidents for opposite-direction vehicles and rear-end and run-off-road accidents for same-direction vehicles. Possible prior movements for these accident types include high speed, rapid deceleration, shoulder encroachment, centerline encroachment, and short headway. The appropriate traffic-performance measures are the same as those identified through the IDA analysis.

Traffic-Performance Measures for Curvilinear Roadways

The driving task is much more demanding for winding-roadway sections and isolated horizontal curves than for tangent sections. Adjustments to the steady-state control behavior associated with tangent roadways are required to safely negotiate the curvature. Guidance for these adjustments must be provided at certain key locations to inform the driver of the necessary actions.

For two-lane curved alignments, the IDA and APM analyses also identified speed and lateral placement as the primary indicators of driving behavior. However, four specific locations were suggested for measurement points:

1. Advance of curve,
2. Point of curvature,
3. Curve midpoint, and
4. Point of tangency.

Because the task of driving through a curved section usually results in adjustments to both speed and lateral placement, the relative extent to which these parameters change between consecutive measurement points should reflect the degree of driving difficulty and therefore degree of hazard.

Site Selection

Several criteria were established for selecting a variety of sites at which data on accident experience and traffic performance would be collected. Initially, a large number of 4.8 to 8.1-km (3 to 5-mile) sections of two-lane highways in six eastern states were identified on the basis of the site-selection matrix given in Table 1. Resource constraints dictated that field experiments could only be conducted for a maximum of approximately 36 sites. Therefore, the search process was directed toward locating one acceptable site for each cell of the site-selection matrix. For purposes of analysis, all boundaries were established to provide a broad range of delineation-treatment, roadway-situation combinations. The actual frequency with which each combination might be found in the field was of secondary importance. In addition, any site that had not experienced an accident over a 5-year period, regardless of type of delineation present, was deleted from consideration (only five potential sites were rejected by this criterion).

Potential sites were then field inspected and evaluated for acceptability. Each site was required to have one or more subsections with a reasonably small gradient and a horizontal alignment that would have the following characteristics:

1. A pure tangent section that is at least 1.1 km (0.68 mile) long and ends in horizontal curves no sharper than 3°;
2. An S section that has two consecutive, reversed curves that are separated by a tangent no longer than 152 m (500 ft), are approximately equivalent, and are 5° or sharper to be clearly distinguishable from the tangent section; and
3. An isolated horizontal curve that is isolated from other curves by 0.5 to 0.8 km (0.3 to 0.5 mile) and is on a highway that is more tangent than winding.

Accessible, safe, and reasonably inconspicuous parking places were required for the vehicle housing the data-collection equipment. Potentially significant roadside features that might uniquely affect vehicle tracking or accident occurrence were avoided. The pavement had to be reasonably free of cracks and sound to allow attachment of electronic tape switches. Shoulders that afforded a significant visual contrast to the pavement were avoided especially at sites without edge lines. In all cases, the existing delineation could be neither badly worn nor newly installed.

Data Collection

The site-selection process resulted in the identification of 32 field sites. Two sites each in the tangent-roadway and horizontal-curve sections could not be found. On the basis of the potential traffic-performance measures identified by the IDA and APM analyses, we decided to concentrate on two observable parameters, speed and lateral placement. Measurement was accomplished by using pairs of resistance-based electronic tape switches. The location of each pair of switches, or trap, was determined on the basis of the IDA analysis. Typical tape-switch deployment configurations are illustrated in Figures 1 through 3. For tangent situations, traps were placed about 183 m (600 ft) apart.

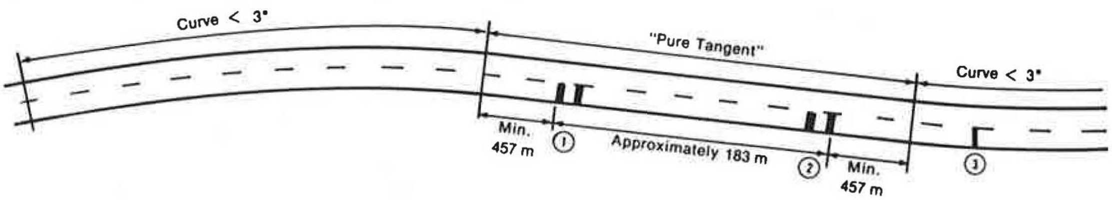
The tape switches were connected to a vehicle placement and event monitor (VPEM). The VPEM contained D'Arsonval meters for lateral-placement measurement and high-precision digital clocks to collect data for speed calculations. A pneumatic tube counter was also installed well downstream of the monitored area to obtain an hourly volume profile. As a vehicle crossed successive tape switches, the 50-mark lateral-placement meters and digital clocks would stop at the measured values until they were manually reset. This prevented confusion of readings when vehicle platooning occurred,

Table 1. Site-selection matrix.

Roadway Section Type	Degree of Curvature	Roadway Width (m)	Volume[a]	Shoulder Width (m)	Painted Centerline Only	Painted Centerline and Edge Lines	Painted Centerline and Edge Lines Plus Post-Mounted Delineators
Tangent		4.9 to 5.5	500 to 2000	≥1.2	x	x	—
		5.8 to 6.4	500 to 2000	<1.2	x	x	—
		5.8 to 6.4	500 to 2000	≥1.2	x	x	—
		5.8 to 6.4	2000 to 5000	<1.2	x	x	—
		5.8 to 6.4	2000 to 5000	≥1.2	x	x	—
		6.7 to 7.3	500 to 2000	<1.2	0	x	—
		6.7 to 7.3	2000 to 5000	≥1.2	0	x	—
Winding		4.9 to 5.5	500 to 2000	<1.2	x	x	—
		5.8 to 6.4	500 to 2000	<1.2	x	x	—
		5.8 to 6.4	500 to 2000	≥1.2	x	x	—
		5.8 to 6.4	2000 to 5000	<1.2	x	x	—
		5.8 to 6.4	2000 to 5000	≥1.2	x	x	—
Isolated horizontal curve	3 to 6	4.9 to 5.5	500 to 2000	<1.2	x	x	—
	3 to 6	5.8 to 6.4	2000 to 5000	≥1.2	x	—	x
	3 to 6	6.7 to 7.3	2000 to 5000	≥1.2	—	x	x
	>6	4.9 to 5.5	500 to 2000	<1.2	x	0	—
	>6	5.8 to 6.4	500 to 2000	≥1.2	—	x	x
	>6	5.8 to 6.4	2000 to 5000	≥1.2	0	—	x

Notes: 1 m = 3.3 ft.
x = site found; 0 = desirable site could not be found; — = no site was sought.
[a] Based on 1975 average daily traffic.

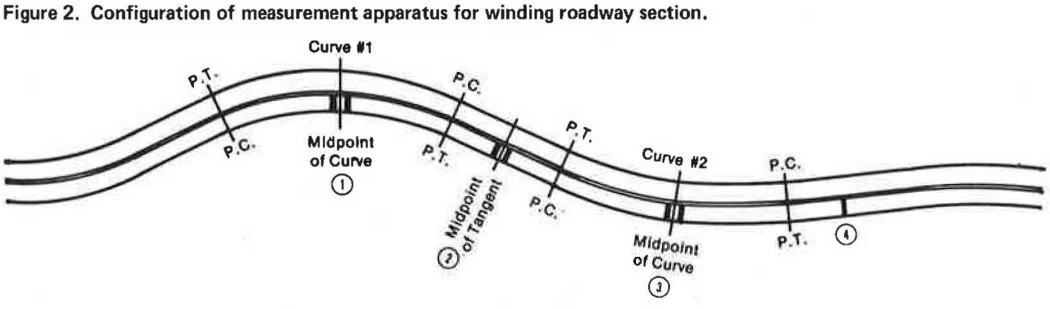
Figure 1. Configuration of measurement apparatus for tangent roadway section.



Station	Location	Measurement
1 & 2	Points on tangent section no closer than 457 m (1,500 ft.) from nearest curve	Speed Lateral Placement*
3	Any point beyond Station 2	Volume

*Includes centerline and shoulder encroachments.

Figure 2. Configuration of measurement apparatus for winding roadway section.



Station	Location	Measurement
1	Midpoint Curve #1	Speed Lateral Placement*
2	Midpoint of Tangent	Speed Lateral Placement*
3	Midpoint Curve #2	Speed Lateral Placement*
4	Any point beyond Station 3	Volume

*Includes centerline and shoulder encroachments.

and allowed accurate recording of values before subsequent free-flowing vehicles arrived.

Reading of the lateral-placement meters was generally to the nearest half or whole mark, or to within 1 to 2 percent of the calibrated length of the tape switch. Possible additional measurement errors consisted of 0.2 percent within the VPEM (determined under laboratory conditions) and 1 to 2 percent related to the tape-switch calibration process. In total, the error in the determination of true lateral placement for an individual activation was expected to be 2 to 4 percent of the calibrated switch length, or 7.6 to 15.2 cm (0.25 to 0.50 ft). Because there was no reason to suspect a significant systematic bias in this error, the individual deviations from true lateral placement were of little consequence when averaged over a large sample.

The error of the speed measurement was a function of the clock resolution (±0.01 s), the trap length, the amount of error in the physical layout of the trap, and the magnitude of the vehicle speed. In general, for a trap length of 6.71 ± 0.06 m (22 ± 0.2 ft) and a speed of 80.5 km/h (50 mph), the speed estimate would be ±3.2 km/h (2 mph) of the true value. This result is comparable to the accuracy expected from a radar speed meter.

The choice of the sample size to be used in the conduct of the data-collection effort was one of the more important decisions to be made in the planning phase of the project. An assumption of normally distributed speed and lateral-placement observations plus previous estimates of typical population variances were used in a standard statistical formula to determine the required sample size for estimating the true population mean. For a significance level of 95 percent and a confidence interval of ±3.2 km/h (2 mph) to estimate mean speed, a minimum sample size of 100 observations would be required. With this number of observations the typical confidence interval for lateral placement estimation is about ±6.4 cm (2.5 in), which is an acceptable value slightly less than the measurement error of the tape-switch system.

The sample size required to accurately estimate the variance was determined by expressing the confidence interval in terms of sample variance, points in the chi-square distribution, and alternative values for degrees of freedom (d.f.), i.e., sample size minus 1. We found that a larger sample size is required to obtain the same degree of accuracy found above in the estimation of the mean. The results of this analysis indicated that, to maintain an error of no more than ±10 percent in the estimate of standard deviation for lateral-placement observations, a sample of 150 observations would be desirable. A sample of 100 was considered the practical minimum and yields a confidence interval of ±14 percent at the 95 percent significance level.

On the basis of the sample-size analysis and the relatively high person-hour costs associated with sampling under low-volume conditions, the basic speed and lateral-placement data were collected for a minimum of 100 observed vehicles during both day and night conditions respectively. Although delineation is most critical under adverse weather conditions, particularly at night, the infrequent and unpredictable nature of rainfall precluded the possibility of collecting sufficient wet-weather data for accident modeling purposes.

In all cases, lateral-placement measurements were referenced to the outside edge of the traveled lane, which was defined as the physical edge of pavement for roadways without edge lines and the midpoint of the edge line for roadways with edge lines. In the latter case, any pavement or stabilized material outside the edge lines was considered to be part of the shoulder width. Other data recorded for each site included the average daily traffic volume, width of traveled way, speed limit, length and degree of curve (if any), and type of delineation.

STATISTICAL ANALYSIS

To provide insight into driver performance as related to safety and to help guide the modeling effort, a distributional analysis of the speed and lateral-placement data among the three general roadway types was undertaken. The analysis revealed that the mean and variance of speed did not vary significantly between traps within a given site or between day and night visibility conditions. However, when lateral-placement data were compared, readily observable differences appeared for winding and horizontal curve sites (Figures 4 and 5).

In cases where there was a travel path from an inside curve to an outside curve on sections of winding roadway, the lateral-placement profile showed an increasing displacement from the shoulder. This straightening of the roadway was most pronounced under night conditions. In cases of isolated horizontal curves, there was little variation in lateral placement between the advance point and the point of curvature; however, between the point of curvature and the curve midpoint, motorists tended to move closer to the shoulder for the inside curve and closer to the centerline for the outside curve. Again, the night displacements were more pronounced. As the magnitude and frequency of these displacements increase, the accident potential of the roadway also seems to increase.

In addition to the basic speed and lateral-placement data discussed above, a number of other traffic-performance measures were derived for the accident-modeling effort. These were generally arithmetic functions of speed or lateral-placement statistics normalized by ADT volume, shoulder width, or width of the traveled lane.

Accident Data

Accident data were obtained for each study site for a minimum of 2 years during which the existing delineation was present. The data base encompassed a period from as early as 1969 through 1975. Data were always based on multiples of 12-month periods to avoid introducing possible seasonal biases. In determining accident rates for a tangent or winding section, we included accidents that occurred anywhere on the entire section length (4.8 to 8.1 km or 3 to 5 miles).

In determining accident rates for an isolated horizontal curve, we included accidents that occurred within a subjectively established zone of influence extending 229 m (750 ft) beyond the points of curvature. However, because isolated horizontal curves represented spot locations rather than extended sections of roadway typical of tangent and winding situations, the observed number of accidents was extremely low. Therefore, to provide a larger data base on which to compute accident rates, additional horizontal curve sites were selected wherever possible for each of the 10 cells of the site selection matrix.

The objective of the accident modeling was to relate accident histories to the traffic-performance measures collected at the sites; therefore, we hypothesized that certain subsets of the accident data would be more highly correlated than the entire set of accidents. For example, we assumed that traffic-performance measures collected during nighttime conditions would be more closely related to night accidents alone than to all accidents. Therefore, the accidents were grouped into the following subsets.

1. Total accidents were considered to be all accidents except those occurring during snowy- or icy-pavement conditions or during fog conditions. Snow- and ice-related accidents were deleted to eliminate the unfavorable bias for northern states as opposed to southern states. Also, when snowy, icy, or foggy conditions occur, traffic-performance measures are likely to be quite different from those observed during field data collection.

2. Non-intersection-related accidents were considered to be the portion of the total accidents that did not occur in or near any intersection within the study section.

3. Light-condition accidents were considered to be the total accidents that occurred, grouped into daytime and nighttime subsets, to correspond with the traffic-performance measures observed within day versus night hours.

4. Pavement-condition accidents were considered to be the total accidents that occurred, grouped into wet- and dry-pavement condition subsets.

5. Delineation-related accidents were considered to be the portion of the total accidents that were identified as being possibly related to the presence or absence of delineation. An accident that involved any one or more of the characteristics given in the table below was classified as not related to delineation.

Accident Type	Characteristic
Collision	Train
	Animal
	Fixed object within travel lanes

Figure 3. Configuration of measurement apparatus for isolated horizontal curve.

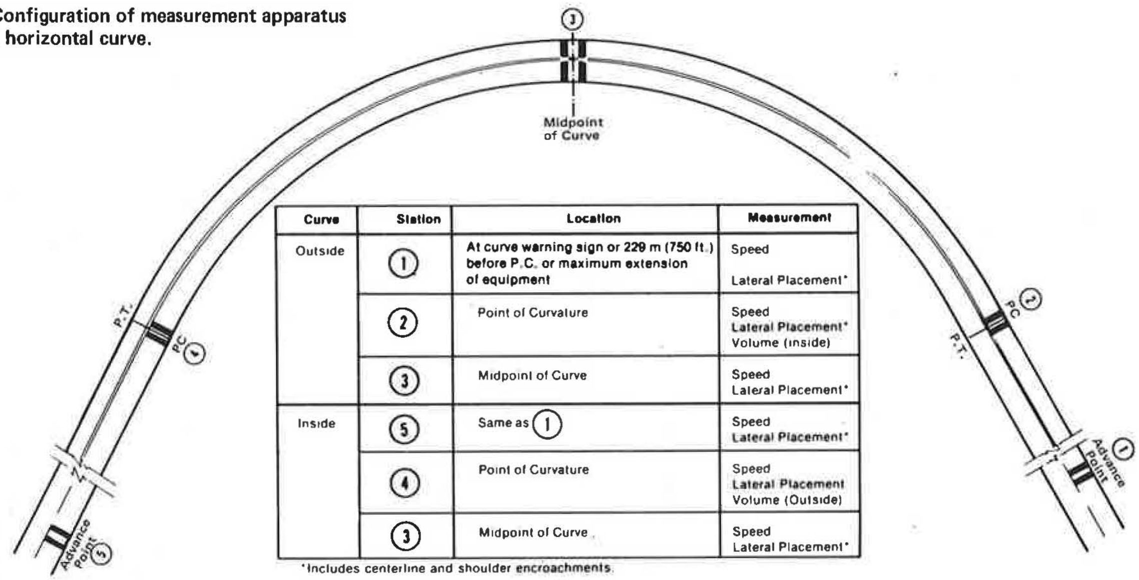
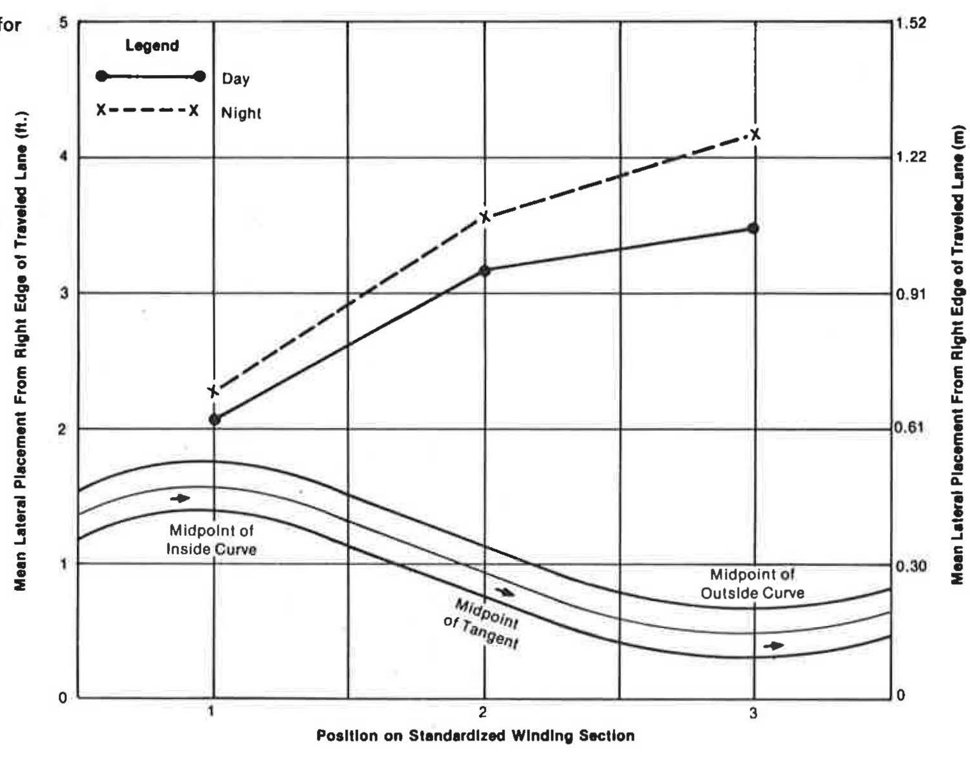


Figure 4. Lateral-placement profile for winding roadway section.



Accident Type	Characteristic
Maneuver	U-turn Starting Improper turning Parking Backing
Traffic control	Police officer Railroad crossing
Major factor	
Driver-related	Improper turn Backing onto roadway Stopped on roadway Avoiding animal or object
Vehicle-related	Defective equipment Struck by object
Roadway-related	Road defect
Vehicle	Farm truck Emergency vehicle

Table 2. Number of sites and accidents by roadway section type.

Roadway Section Type	Number of Sites	All Reported Accidents	Total Accidents[a]	Other Accidents[b] Day	Night	Total
Tangent	12	490	449	105	90	195
Winding	10	410	360	92	83	175
Horizontal curve	20	78	71	16	19	35
Total	42	978	880	213	192	405

[a] Excludes snow-, ice-, and fog-related accidents.
[b] Delineation-related, nonintersection, dry-pavement accidents.

Selected characteristics of the accident data are shown in Tables 2 and 3. Single-vehicle, run-off-the-road accidents were the most prevalent on all three roadway types especially during the hours of darkness, when only about 20 percent of the ADT occurred. Also apparent is the very low number of accidents that occurred at the sampled horizontal-curve sites. An analysis of severity data indicated that, overall, 3 percent of all accidents resulted in a fatality and 43 percent resulted in personal injury (these percentages are slightly higher for the isolated horizontal-curve situation).

Accident Modeling

Using traffic-volume data supplied by each state, we calculated accident rates for the selected field sites for both daytime and nighttime periods. For tangent and winding sections, accident rates were expressed in units of accidents per million vehicle-kilometers traveled and were calculated by using the following equation:

$$\text{Accident rate} = [(10^6)(N)]/[(L)(\Sigma_j ADT_j)(P_f)(L_f)] \quad (1)$$

where

N = number of accidents occurring during a given time period and under a specifically defined set of roadway, surface, and lighting conditions;
L = section length in kilometers (1 km = 0.6 mile);
ADT_j = ADT during time period j, either a portion of a year or a year;

Figure 5. Lateral-placement profile for horizontally curved roadway section.

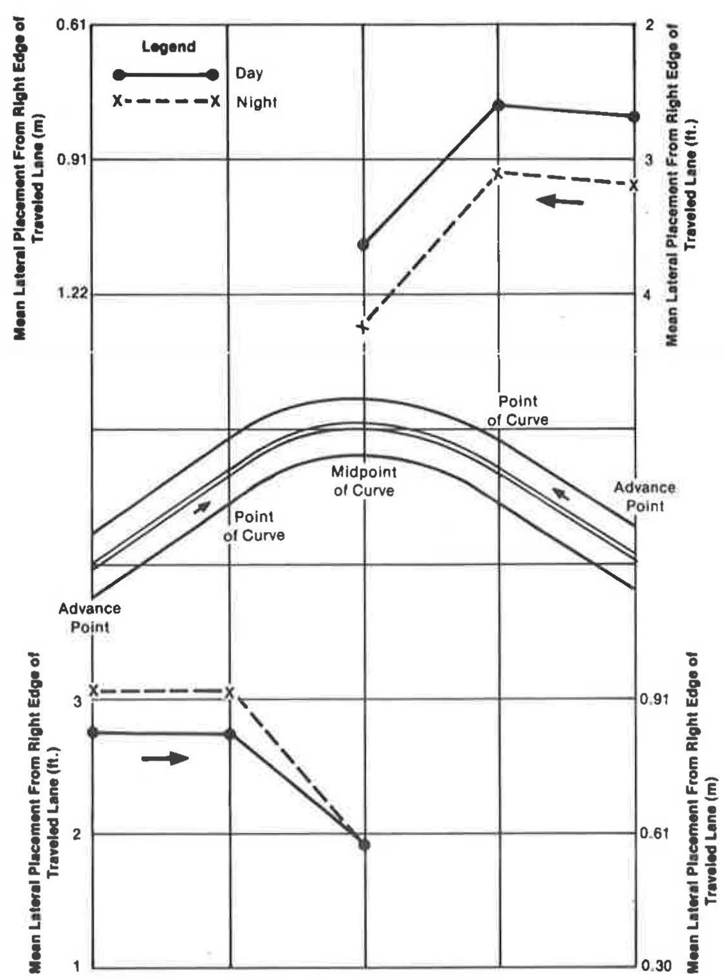


Table 3. Percentage distribution of accidents by selected characteristics.

Accident Type	Tangent: Total Accidents[a]	Tangent: Other Accidents[b] Day	Tangent: Other Accidents[b] Night	Winding: Total Accidents[a]	Winding: Other Accidents[b] Day	Winding: Other Accidents[b] Night	Horizontal Curve: Total Accidents[a]	Horizontal Curve: Other Accidents[b] Day	Horizontal Curve: Other Accidents[b] Night
Single vehicle									
Run-off-road	36	35	63	64	62	73	52	69	74
Fixed-object	16	4	2	7	5	5	13	13	0
Other	1	2	1	1	2	0	1	0	0
Multiple vehicle									
Head-on	3	3	3	2	3	1	4	0	0
Sideswipe (same direction)	9	17	7	5	4	4	4	13	5
Sideswipe (opposite direction)	5	8	8	8	11	13	8	6	5
Rear-end	11	17	7	4	3	1	9	0	0
Angle	17	11	4	6	4	0	5	0	5
Other	2	3	4	3	4	4	4	0	11

[a] Excludes snow-, ice-, and fog-related accidents.
[b] Delineation-related, nonintersection, dry-pavement accidents.

P_f = factor to account for the average percentage of the time period during which the weather conditions at the time N can be expected; and

L_f = factor to account for the average percentage of ADT occurring under the ambient light conditions present at the time of N.

Because the isolated curve was being considered as a point location, L was omitted from the above equation leaving the accident rate expressed in accidents per million vehicles. Many different accident rates (reflecting permutations of the above parameters) were calculated and later analyzed. However, the accident rate ultimately used for the tangent and winding situations considered only those delineation-related accidents occurring at night on dry pavement and away from the influence of intersections.

The accident modeling was initiated by assuming that values of certain traffic performance measures (as suggested by the IDA and APM analyses) plus geometric variables could be used to independently predict potential accident hazard. The traffic-performance measures would indicate the manner in which drivers traverse a given section of roadway; the geometric variables would in effect define the available factor of safety inherent in the roadway design. Extreme values of traffic-performance measures in combination with a limited factor of safety would be expected to result in an above-average accident rate.

After preliminary analysis of the speed and lateral-placement data, we combined the tangent and winding roadway situations to form a single general roadway data set. Isolated horizontal curves were retained as a separate situation because of the difference in the accident-exposure measure. A number of traffic-performance measures for both daytime and nighttime dry-pavement conditions were then developed from the speed and lateral-placement data. Generally, these derived variables expressed the change in the average vehicle trajectory between two specifically defined stations, normalized by a geometric element to represent an available margin for driver error.

Alternative accident-probability models were then formulated and tested by using multiple linear regression analysis. A statistically significant regression model could only be developed for delineation-related, nonintersection accidents that occurred on extended sections of two-lane roadway during nighttime, dry-pavement conditions. The model shown below explained almost 81 percent of the accident-rate variance within the sample at the 95 percent significance level. The standard error of estimate was 1.33.

$$AR = 5.01 + 0.610CI + 58.2DPV + 2.03SI - 0.886RW - 0.501SW \qquad (2)$$

where

AR = number of nighttime, delineation-related, nonintersection accidents per million vehicle kilometers (dry-pavement condition only);
CI = centrality index;
DPV = difference in lateral-placement variance;
SI = skewness index;
RW = roadway width measured between outside edges of the two traveled lanes (meters); and
SW = shoulder width (meters).

The centrality index is expressed as

$$CI = (\overline{LP}_e - \overline{LP}_c)/0.1LW \qquad (3)$$

where

$\overline{LP}_e$ = mean lateral placement of the right vehicle tire with respect to the right edge of the traveled way (meters),
$\overline{LP}_c$ = mean lateral placement of the left side of the vehicle with respect to the centerline of the roadway (meters), and
LW = width of traveled lane (meters).

As the value of the centrality index approaches zero, lateral clearance on each side of the average vehicle is maximized. For the winding-roadway situation, the centrality index was computed for the midpoint of the inside curve, shown as station 1 in Figure 2. The difference in lateral-placement variance is expressed as

$$DPV = (LP^2_{s1} - LP^2_{s2})/LW \qquad (4)$$

where LP^2_{si} = variance of lateral placement with respect to the right edge of the traveled way, measured at station i (square meters).

In the case of tangent roadways, the locations of stations 1 and 2 are illustrated in Figure 1. As shown in Figure 2, for winding-roadway situations, stations 1 and 2 represent the midpoint of the inside curve and the midpoint of the intervening tangent respectively. The distance between the two measurement points was approximately 152 m (500 ft).

The skewness index (SI) describes the skewness of the distribution of vehicle speeds. As this statistic becomes increasingly positive, a higher percentage of the traffic stream is traveling at a rate exceeding the mean speed. For the winding roadway the skewness index was computed for the midpoint of the inside curve, shown as station 1 in Figure 2.

In each case, the sign of the regression coefficient corresponds to the expected relationship between the variable and accident potential. Increasing values of the three traffic-performance measures indicate greater deviations in vehicle trajectories and, therefore, greater potential hazard. Increasing values of roadway and shoulder width, on the other hand, indicate greater margins for error in vehicle guidance and, therefore, reduced potential hazard. A sensitivity analysis revealed that the relative influence of the traffic-performance variables on potential accident rate ranges from two to three times that of the roadway- and shoulder-width variables.

CONCLUSIONS

In considering the application of the model to the evaluation of new delineation treatments, we must emphasize certain limitations. First, the model was developed from data collected at a limited number of field sites. Validation of the significant variables and their relationship to accident potential is a need that remains to be fulfilled. Second, the model can only be used to compute the expected rate of delineation-related, nonintersection accidents that occur during hours of darkness and on dry pavements. Thus, the equation developed should not be considered capable of accurately predicting the overall accident rate for any particular section of rural highway.

The potential benefits of delineation under other conditions, including the isolated horizontal-curve roadway section, must be evaluated by different methods. One approach would be to experimentally evaluate traffic-performance measures derived from speed and lateral-placement data. If statistically significant changes occur when type of delineation is varied, these changes could then be interpreted in terms of their potential effect on hazard. This type of evaluation procedure will, in fact, be applied in the second phase of this research study. It is anticipated that the additional experience with the model and the traffic-performance measures will permit a more definitive assessment of their appropriate role in traffic-safety studies.

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Critique of the Traffic-Conflict Technique

J. C. Glennon, W. D. Glauz, M. C. Sharp, and B. A. Thorson,
Midwest Research Institute, Kansas City, Missouri

This examination of the utility of the traffic-conflict technique includes an evaluation of previous studies and a discussion of recent results of a Federal Highway Administration (FHWA) study. The FHWA study attempts to develop a rigorous experimental design by using traffic conflicts as the basic response variable to measure the effectiveness of implementing various access-control techniques. Although some of the studies conclude that the traffic-conflict technique is a reliable tool for predicting accident potential, these conclusions are not well supported. The concept of conflict analysis should not be abandoned, however, but a more rigorous data base should be acquired before the reliability and utility of conflict analysis can be assured.

Traffic accidents are the ultimate measure of safety for a highway location. Because attempts to estimate the relative safety of a highway location are usually fraught with the problems associated with the unreliability of accident records and the time required to wait for adequate sample sizes, the traffic-conflict technique (TCT) was developed as a substitute measure. Originally developed by the General Motors Research Laboratories (GMR) in 1967 (2), TCT was conceived as a method of measuring accident potential at intersections. Conflicts were defined as evasive vehicular actions and characterized by braking and weaving maneuvers. Based on the results of a large study by FHWA (3) in 1971, TCT has gained popularity as an evaluative tool. The Washington Department of Highways is using TCT as a diagnostic tool to determine appropriate countermeasures at high-accident locations. Other researchers and organizations have suggested the technique as a priority-ranking criterion for programming the implementation of spot improvements. Furthermore, FHWA has incorporated TCT as a research tool in its contract research program. This report evaluates the state of the art of TCT and the results (1) of recent attempts to develop a rigorous sampling plan using traffic conflicts as the basic-response variable to measure the effectiveness of access-control techniques at commercial driveways.

The GMR procedure defines a traffic conflict as an evasive action of a driver. Evasive actions are evidenced by brake-light indications or weaving maneuvers (lane changes) forced on a driver by an impending accident situation or a traffic violation. GMR defines five types of conflicts in accordance with the basic types of accidents at intersections: left-turn, weave, cross-traffic, red-light violation, and rear-end (4). A GMR traffic-conflict survey is a 1-d surveillance of two intersection-approach legs. Observations from two opposite legs are recorded in a 10-h counting day (7:30 a.m. to 12 n. and 12:45 p.m. to 6:15 p.m.). Two team members observe the same approach leg at the same time; one observer counts conflicts and the other observer records volume data for all movements. Data samples of 15-min length are taken alternately on each approach leg; after each period, 15 min is given to record the data and move to the opposite approach. Thus, $2\frac{1}{2}$ h of data are collected each day on each approach leg.

CRITICAL REVIEW OF PREVIOUS STUDIES OF TCT

Many studies have attempted to establish the potential utility of TCT. Study objectives have ranged from developing equations that predict accidents from conflict counts to using TCT as a diagnostic tool to identify specific safety deficiencies of intersections. This review traces the evolution of the conceptual elements of TCT and examines the various ways TCT was expounded.

Federal Highway Administration

TCT was evaluated by FHWA (3) in cooperation with the highway and transportation departments of Washington, Ohio, and Virginia. In addition to field tests, analyses were made of statistical relationships between traffic accidents and traffic conflicts. The technique was also evaluated to determine if conflict data were advantageous in assessing the need for safety improvements.

The GMR technique was used at 886 intersection-approach legs scheduled for engineering improvements although 420 approaches were counted after construction of the improvements. FHWA then performed regression analyses to determine the relationships between conflicts and accidents, and correlation coefficients were calculated for four of the five traffic-conflict types. Results of the FHWA analysis were as follows:

1. The data supported the hypothesis that conflicts and accidents are associated;
2. Safety deficiencies at intersections can be pinpointed more quickly and reliably by using TCT than by using conventional methods;
3. TCT is particularly valuable at low-volume rural intersections where accident-reporting level is low;
4. TCT, because of its usefulness in pinpointing intersection problems, should lead to low-cost remedial actions;
5. TCT can be applied, with minor modification, to locations other than intersections;
6. The effect of intersection improvements may be demonstrated by TCT shortly after completion of a spot improvement; and
7. The general surveillance information obtained with TCT may be valuable in improving the operations of intersections.

The very brief FHWA report basically covered only the gross correlation between conflicts and accidents, and no attempt was made to account for parameters other than intersection type. For example, a reasonable supposition is that both accidents and conflicts are highly correlated to traffic volume, yet this dependence was not examined. [Further analysis of FHWA data regarding the correlation of accidents and conflicts in which average daily traffic (ADT) is the partial is given below in the section on estimation of sample-size requirements.] Therefore, even the most significant correlations may not reflect a causal relationship.

Most of the FHWA conclusions are not supported by the data. Only the first conclusion, that accidents and conflicts are associated, is supported because most of the correlations were significant. But, the significant correlations explain only a small portion of the total accident variability. Therefore, the conclusion that TCT can pinpoint safety deficiencies and the other conclusions that follow are only hypotheses.

Ohio Department of Transportation

When the FHWA research program ended, the Ohio Department of Transportation pursued an evaluation of TCT (5). The initial results of this research indicated that FHWA predictions were not suited for the Ohio data. Paddock and Spence felt that, even though their data were used by FHWA, the Washington and Virginia data biased the equations toward urban trends and ignored the basically rural data of Ohio.

Major emphasis in the Ohio program was on the relationship between intersection accidents and conflicts. Other applications, such as analysis of freeway gore areas, weave sections, and other areas of traffic conflicts, were also considered. Also, the additional parameters of cross-street volume, percentage of commercial vehicles, and so forth were expected to improve the prediction ability of TCT.

The Ohio data base was enlarged to 611 approach legs; again the GMR technique was used but with just a single observer. A series of regression models was generated to determine the optimum means of projecting accidents in Ohio. The Ohio study concluded that an accurate 2-year accident prediction was a function of one or more of a number of conflicts and volume variables. The regression equations resulted in the following errors in predictions of accidents per year for the percentage of confidence levels shown.

Data Class	Approach Legs	50%	75%	95%
All data	611	±1.2	±2.0	±4.2
Signalized	220	±1.5	±2.4	±4.6
Unsignalized	391	±1.1	±1.8	±3.8

As a result of the study, the investigators concluded that TCT was a usable accident-prediction tool; they also concluded that accurate prediction equations could be generated with additional development.

This is the only study in which multiple regression of the variables was attempted. But the conclusions must be interpreted in light of the analysis. In fact, the multiple-regression equations are more sensitive to the various measures of conflict opportunities (traffic-volume counts). Therefore, these equations do not establish TCT as a usable accident-prediction tool, but simply substantiate many earlier studies that document a high positive association between accidents and traffic-volume measures.

Washington Department of Highways

After the FHWA study, the Washington Department of Highways also continued the application and refinement of TCT (6). Major effort was centered in

1. Relating conflicts to accidents,
2. Determining the reliability of the accident data,
3. Determining the interrelationships between different types of conflicts and their contribution to the total number of conflicts, and
4. Establishing priorities for corrective action based on the number of conflicts.

The 240 intersections studied were divided into four categories: signalized and channelized, signalized only, channelized only, and nonchannelized and nonsignalized intersections. Within each category, five priority groups were established based on the number of conflicts per hour.

Pugh and Halpin concluded that TCT is a valuable tool for assessing accident potential. Although the accident-conflict correlations were rather low, the conflict data seemed beneficial in establishing priority groups that ensured that accidents would be reduced when conflicts are lessened. The average number of unweighted accidents for total accidents during 3 years was as follows:

Priority Group	Conflicts per Hour	Intersections	Avg Unweighted Accidents
1	40 and over	39	23.4
2	27 to 39	38	15.8
3	19 to 26	34	14.0
4	10 to 18	55	9.2
5	9 and under	74	6.3

Another conclusion was that the lack of year-to-year consistency in accident data sets practical limits on the probability of predicting accidents from conflicts.

Similar to the FHWA study results, the Washington study results show only a positive association between conflicts and accidents. The results do not indicate that conflicts will explain much of the accident variability. Therefore, the conclusion that conflicts are a valuable tool for assessing accident potential is unsupported.

The results given in the tabulation above seem encouraging but are of little practical value because

1. They probably also show that both conflicts and accidents have a high positive association with traffic volume, and
2. More important, they ignore the site parameters that might explain why the number of conflicts of some intersections in a group vary significantly from the group average.

The conclusion on the reliability of accident records has some interesting implications. This lack of reliability does not necessarily mean that conflicts are a poor measure of safety but rather that determining whether they are reliable measures is difficult because of the need to rely on accident records that are highly variable.

U.K. Transport and Road Research Laboratory

Three studies (7, 8, 9) by the U.K. Transport and Road Research Laboratory (TRRL) examined TCT for its use in assessing the safety of intersections. Spicer criticizes the GMR technique on the basis that recording all conflicts, without grading them by severity, gives results more highly correlated to a count of intersection maneuvers than to accidents. Therefore, Spicer extended the conflict method by classifying conflicts by severity. The TRRL research used a tower-mounted, 16-mm camera that ran at a speed of two frames per second and gave a continuous record of the events at the intersections. The film allowed a before, during, and after study to be made of each conflict. The involvement of other vehicles, flow count in all directions, and measurements of maneuver and delay time were also determined from the film.

After the written reports and the film were reviewed, all conflicts were identified and graded from 1 to 5 by severity of event, which ranged from simple precautionary braking or lane changing to emergency action followed by collision.

Conflicts were separated into two classifications: serious conflicts and all conflicts. These classifications were further divided by maneuver type (e.g., rear-end) and place of occurrence. Conflicts as well as the number of injury accidents (from accident record files) were tabulated by time and location on the highway. Rank correlation coefficients were then calculated, and significance was tested. Rank correlations were also calculated for traffic flow versus serious conflicts, all conflicts, and accidents to test the hypothesis that flow levels are not related to accidents or conflicts. If more than two vehicles were involved, that fact was noted for both serious and nonserious conflicts. In addition, the crossing behavior of a sample of drivers, categorized by age of driver, was analyzed. The following conclusions were drawn for these studies:

1. A simple definition of a conflict as a situation involving one or more vehicles in evasive action does not provide a measure of accident potential that correlates closely with accident data;
2. Conflicts defined as serious correlate well with reported accidents of location and time;
3. Study of the circumstances before the serious conflicts revealed that in 75 percent of the conflicts vehicles other than the two immediately concerned with the conflict were present;
4. The conflict and accident rates increased with the increase in vehicle flow;
5. Vehicle speed, time of crossing, and the crossing path taken were factors in accident causation, as implied by conflict data;
6. For the six intersections of the third study, the number of serious conflicts was directly proportional to the number of injury accidents (the number of injury accidents in 3 years was essentially equal to the number of serious conflicts in 10 h for all six sites);
7. At three sites the locations of the conflicts identified the locations of the reported injury accidents in order of importance;
8. No clear relation was shown between traffic flow and the serious conflict or injury-accident rates (the effects of vehicle flow and speed patterns on the conflict and accident rates appear to be complex); and
9. Data on serious conflicts can provide information enabling the ranking of intersections in order of safety.

The TRRL research appears to indicate the most promising approach in relating conflicts to accidents. Although the sample size of six intersections seems small, the reliability of the regression fit (conclusion 6) cannot be disputed at face value. But based on the characteristics of the study and the seemingly tenuous results of other studies, the almost perfect correlation between 3-year accident records and 10-h conflict counts seems incredible. This doubt stems from what appears as a highly subjective determination of conflict occurrence, the widely different and unusually complex intersections used, and the illogical conceptual connection between serious conflicts and severe accidents. A serious conflict is logically related to accident occurrence but not necessarily to severe (injury) accidents.

This study draws the same conclusion as the Washington study in that conflicts can be used to rank intersections by order of safety. Again, this capability has no practical advantage because

1. Intersections can be ranked as well by using traffic-volume measures, and
2. Ranking intersections this way does not necessarily identify hazardous intersections (those that have a significantly higher number of accidents than the average for similar intersections).

This study, in agreement with the FHWA study, also concludes that conflict measures can pinpoint specific safety deficiencies. The derivation of the conclusion that the location of conflicts identified the location, in order of importance, of the reported injury accidents was not documented in the TRRL reports.

Canada Ministry of Transport

The primary intent of the study by the Canada Ministry of Transport (10) was to summarize the state of the art of the prediction and analysis of accidents at intersections. Consideration was also given to assessing the efficiency of various accident-predictor models, especially the concept of traffic conflicts.

A pilot investigation of TCT was conducted on three intersections to investigate the definitions and observational techniques of TCT. Results of the pilot study included the development of a training manual and the selection of a conflict definition that eliminated precautionary or anticipatory actions.

In a continuation of the study, intersections in four Canadian cities were studied simultaneously. Some of the major changes in the GMR methods and procedures applied to this study included

1. A study period of two 14-h d (accumulated in four 7-h d) to ensure fuller coverage of all traffic conditions;
2. A study team of four observers and a field supervisor (each observer was assigned to a separate approach leg or intersection area);
3. Counting traffic volumes, separated by maneuver, for one 14-h period immediately following the conflict count; and
4. Sampling intersection exposure times for through vehicles and all types of maneuvers during morning-peak, off-peak, and afternoon-peak periods.

A total of 59 unsignalized intersections were used in the analysis including 13 T-intersections; 37 four-legged, two-way intersections; and 9 four-legged, one-way intersections. Accidents and conflicts for each intersection were divided into five categories, similar to the GMR technique.

The most significant part of the analysis was testing the correlation between many of the variables. Among the comparisons were conflicts and accidents, accidents and time-volume exposure index, accidents and total volumes, conflicts and time-volume exposure index, accidents and conflicts by intersection, accidents and conflicts by time of day, accidents and all variables (multiple-regression equation), and conflicts and violations.

Cooper concluded that the application of TCT was neither efficient nor practical. But, by using the detailed observational techniques of conflict analysis, he felt that hazardous locations could be identified by noting their operational deficiencies. Further, conflicts were very dependent on volume and could not account for differences in accidents when corrected for volume exposure. Also, conflict definitions, depending on the degree of subjectivity (or objectivity), cause problems of either uneconomical and impractical collection procedures or poor results in predicting accidents. A measure of volume exposure resulted in better correlation when intersections were ranked by gross accidents. But the application of TCT did appear beneficial in identifying high accident rates within an intersection.

Cooper appears to have correctly interpreted his results by stating that (a) the traffic-conflict measures were neither efficient nor practical, (b) conflicts were very dependent on volume, and (c) the degree of objectivity in the conflicts definition causes a trade-off between the cost and practicality of collection procedures and the precision of accident prediction. He also drew essentially the same conclusion as the FHWA and TRRL studies by stating that the application of conflict measures appears beneficial for identifying high-accident areas within an intersection. Unfortunately, as with the other reports, this report does not quantitatively document the derivation of this conclusion.

University of Toronto

The purpose of the University of Toronto study (11) was mainly to determine the circumstances under which TCT could generate more reliable accident-rate predictions than those obtained from accident history. Expressions of the variance in predicted accident rates were derived for both methods to aid in solving this problem. The variance resulting from using accident records was based on theory plus accident records for 1800 intersections in Toronto from 1970 through 1974. An expression for the variance using traffic conflicts was based on the ratio of accidents to conflicts and its variability, from TCT data reported by others, and the assumption that conflict counts are from a Poisson distribution.

The study concluded that TCT is a more accurate accident-rate predictor than accident records at locations with fewer than four accidents per year or when the accident history is very short. The duration of the conflict count seemed to have a limited effect on the accuracy of estimation, and an economical and practical choice of 1 d was recommended. The use of two operational definitions for conflicts was recommended; one definition was for low-accident-rate conditions and the other definition was for locations characterized by a high number of annual accidents. However, if a single operational definition of conflict is desired, the use of a restrictive definition was recommended. This definition was thought to be applicable to locations of relatively high accident rate although it also works well for low-accident-rate situations.

Hauer's conclusion that a site with a low accident rate or a short accident history or both is a maximum candidate for TCT to be superior is undoubtedly correct qualitatively, but his quantitative conclusions are incorrect for several reasons.

Hauer postulates a stochastic model in which the accidents within a year are a sample from a Poisson distribution, but the Poisson parameters are a sample from a probability distribution of parameters. The annual observations of accidents are a series of n samples each of size 1, drawn from n Poisson densities (as opposed to a simpler model that would regard the series as a sample of size n from one Poisson density).

Hauer then incorrectly derives an expression for the variance in the estimate of future accidents. The expression is incorrect because of at least two criteria: (a) The expression implies that the accident history is not adequate information to estimate the site characteristics, but that other external accident experiences must be available; and (b) the expression implies that no accident history, no matter how extensive, can improve the prediction of the accident rate beyond a certain barrier. This reasoning, of course, is not compatible with elementary statistical principles, which state that the precision of estimating a mean value improves (the variance decreases) as the sample size is increased. We could describe in detail the problems with Hauer's derivations, but this is not necessary because his model has been treated by Hald (12).

In the TCT situation, Hauer postulates that the number of conflicts per unit time, at the site of interest, can be sampled and is Poisson distributed. He also assumes that the probability of a conflict resulting in an accident can be determined from a world sample of accident-to-conflict ratios. That probability is apparently taken as a universal value for a given definition of conflict, and its variance is due only to sampling error. This assumption seems inconsistent with Hauer's model of accidents in that he allows the accident mechanism to vary every year; in the TCT model, however, a universal ratio of accidents to conflicts applies to all sites. These assumptions seem very generous to TCT because they imply that the variance in the accident prediction could be reduced to any desired minimum level by simply increasing the amount of (global) data used to estimate the probability of a conflict resulting in an accident. However, TCT is probably treated harshly in the numerical examples because of the limited data sources used.

ESTIMATION OF SAMPLE-SIZE REQUIREMENTS

As a part of a larger study of alternative techniques for controlling direct access on arterial highways (3), traffic conflicts were examined as a basic response variable. The objective was to develop detailed before-and-after experimental procedures, by using conflict measures, that would allow highway agencies to precisely quantify predictive-accident reduction values for implementing various access-control techniques under a variety of site conditions.

Initial Development of Sample-Size Requirements

The basic plan was to use conflicts recorded on the two highway approaches to three-legged driveway intersections. Data from the Ohio study (5) were selected to develop preliminary sample-size requirements because these were the only raw data available at the time this task was started. The sample-size requirement was the number of days of conflict-data collection (both before and after an engineering change) necessary to detect a certain percentage reduction in predicted accidents with a desired confidence level. The selected Ohio data included 38 unsignalized T-intersections (which would be similar to driveway intersections).

To relate conflicts to accidents, linear regressions of conflicts as a predictor of accident rate were constructed by using total conflicts per approach (with approaches combined) and normalized by approach ADT as well as approach ADT plus crossroad ADT. In no case was the estimated prediction equation encouraging; i.e., the correlation coefficients were a maximum of 0.27.

Because of the low correlations, the conflict measure was used to compute effectiveness measures. Reinforcement of this decision was provided later when the raw data from the FHWA study (3) were obtained and analyzed. Although the FHWA study had reported a high correlation (84 percent) between conflicts and accidents at 94 unsignalized T-intersections, the subsequent analysis of the raw data showed a much lower correlation coefficient (0.59), meaning only 35 percent (0.59^2) of the variability in accidents is accounted for by the effects of conflicts.

This interpretation of the correlation coefficient is correct in that it describes how useful conflicts are in predicting accidents, but the interpretation may not be meaningful as a descriptor of the genuine causal relationship between conflicts and accidents because both of these variables may be more related to other site variables than to each other. For example, if both accidents and conflicts are positively correlated to traffic volume, the partial correlation coefficient between accidents and conflicts (the partial being ADT) is a better estimator of the genuine association between accidents and conflicts. This coefficient for the FHWA data is 0.35, which is significantly smaller than 0.59. Both conflicts and accidents are correlated to ADT so that the implied association between conflicts and accidents appears stronger than the genuine relationship. In addition, other variables not observed in the FHWA data may also interact with the conflict-accident relationship.

The Ohio data contained between-site variability, whereas the intended before-and-after experiments would eliminate or minimize this component of variance. Therefore, the sample sizes predicted from the Ohio data are too large for before-and-after studies. For this reason the theoretical value for the within-site standard deviation was estimated by using a Poisson assumption about the distributional form of conflicts at a site, and the Ohio data were used to estimate the Poisson parameter. The resulting theoretical sample sizes for a $1 - \alpha$ confidence level are given in the following table. A 5-h conflict count was used because it was considered to be more practical than the $2\frac{1}{2}$-h conflict count used in the Ohio data. The entries represent the necessary sample pairs at a site before and after treatment. Although these requirements appear large they represent a marked improvement over predictions made with less information.

Δ (%)	90%	95%	99%
10	20	28	49
20	5	7	12
30	2	3	5

Field Studies of Conflicts

The field studies were originally planned to validate the experimental design. Because of the difficulties encountered in the determination of sample-size requirements, the field studies were conducted instead to (a) test the ability of the conflict measure to show a significant change based on the implementation of an access-control technique and (b) estimate within-site variability of conflicts for determining more reliable sample-size estimates.

Project time constraints precluded before-and-after experiments; therefore, a matched-pair study method was used. Three experiments were conducted, which involved 10 d of observation per matched-pair experiment (5 d/site). Generally, each site of a matched pair was identical to the other except one site had uncontrolled access and the other had an access-control technique of interest. Surveys were conducted on weekdays from 10 a.m. to 6 p.m.; data collection was divided into fifteen 20-min counting periods, and 10 min were allowed between periods for computations and rest.

The within-site standard deviations of conflicts per day are given in the following table with the corresponding coefficients of variation.

Experiment	Site	Standard Deviation	Coefficient of Variation (%)
1	1T	27.6	24.0
	1C	19.6	11.7
2	2T	28.5	17.8
	2C	26.9	19.9
3	3T	17.4	8.7
	3C	15.7	19.6

The within-site variance of the conflict distributions did not vary significantly from site to site. The average standard error is 23.2 (coefficient of variation of 16.2 percent).

If the observed conflicts and traffic volume were independent, the variance of their ratio would be relatively larger than the variance of the conflicts. However, if conflicts were entirely explained by traffic volume, the ratios would have zero variance. In the field-study data, the coefficient of variation of the ratio was somewhat less than that of conflicts themselves. Therefore, some connection apparently existed between conflicts and volume, but it was not strong. Unfortunately, the volume did not vary greatly during the validation studies; therefore, the relationship between conflicts and volume remains to be determined.

Development of Sample-Size Requirements

The field experiments furnished data on conflict distributions needed to estimate sample sizes necessary for detecting reductions in conflicts. If $\overline{X}_1$ is the average number of conflicts in a (5-h) sampling period before an improvement and $\overline{X}_2$ the number afterward, the sample size necessary for detecting a change of size $\overline{X}_1 - \overline{X}_2$ with $1 - \alpha$ confidence level is given by

$$n = (2Z^2_{\alpha/2}s^2)/(\overline{X}_1 - \overline{X}_2)^2 \qquad (1)$$

where

n = number of sampling periods,
s = average standard error, and
$z_{\alpha/2}$ = standard normal variate associated with the desired $1 - \alpha$ confidence level.

Here, $s = 23$ (determined from the six conflict dispersions), and $\overline{X}_1 - \overline{X}_2$ was chosen to span the region of interest. The conflict sample sizes listed in the table below are based on a 95 percent confidence level and are given for five before levels of conflicts. For example, twenty-six 5-h observations before and after treatment are necessary to detect a 25 percent change in conflicts if the before level of conflicts is 50 in 5 h.

Change (%)	50	100	150	200	250
25	26	6	3	2	2
20	41	10	4	2	2
15	72	18	8	4	3
10	162	41	18	10	6
5	650	162	72	41	26

The objective of the field observations of conflicts was to estimate the effect of the improvement on conflicts. In addition, the influences of highway ADT, crossroad ADT, and location on conflicts need to be observed. The effects and interrelationships of these factors will thus dictate when the improvement has a satisfactory effect. A regression curve from the Ohio data allows estimation of the expected number of conflicts as a function of highway and crossroad ADT. These are given in the table below. For example, for a highway of 18 000 ADT with crossroad ADT of 1000, about 138 conflicts could be expected in 5 h. Thus, using s = 23, the sample size necessary to detect a 10 percent reduction with 95 percent confidence is 22.

	Highway ADT			
Crossroad ADT	< 10 000	10 000 to 15 000	15 000 to 20 000	> 20 000
< 500	18	72	124	178
500 to 1500	32	84	138	190
1500 to 3000	58	112	164	218
> 3000	118	170	224	276

ASSESSMENT OF TCT

Based on the state-of-the-art review and the evaluation and discussion in previous sections, the current reliability of TCT for estimating accident potential is questionable. Although some studies have concluded that TCT is a reliable tool, these conclusions are not well supported. Also, for each study with a positive conclusion there is at least one study that indicates the opposite. Therefore, although accident potential may be predicted by conflict counts, existing data do not define the population characteristics of conflicts well enough to estimate the sample size of conflicts needed to reliably predict accident potential.

Basically, there are three practical applications that a reliable traffic-conflict measure could be used for:

1. To identify and rank locations for safety improvements,
2. To diagnose specific safety deficiencies at a location for the purpose of determining specific countermeasures, and
3. To measure the safety effectiveness of implemented countermeasures by using the before-and-after study technique.

Simply ranking the accident potential of intersections is not a realistic use of conflict measurements. If ranking is all that is desired, the more straightforward measure of traffic volume is appropriate. A more realistic use of conflict measures is to predict which intersections will have accident rates unusually higher than the expected average for similar kinds of intersections. This kind of evaluation would indicate the locations that could be made less hazardous through application of accident countermeasures.

For all three potential uses of conflict counts, existing relationships do not allow practical sample sizes. This fact must be particularly kept in mind in diagnostic studies that attempt to pinpoint specific deficiencies at an intersection by using component parts (e.g., left-turn conflicts) of the total conflict count. A basic problem with existing data and relationships is that they are ill-defined. Data have not been adequately stratified, and analyzed accordingly, for significant conditional parameters such as highway ADT, crossroad ADT, number of approach legs, number of lanes, and type of traffic control. Also, reliable estimates of the within-site variability of conflicts are not available.

Another very distinct problem in using existing data and relationships on conflicts is that conflict definitions and sampling procedures vary significantly. With conflict definitions, an additional weakness is that none has a completely objective base. The field determination of a conflict occurrence depends on the observer's judgment of temporal variables such as the initial gap between leading and following vehicles or the magnitude of deceleration. Use of the brake-light application as a criterion creates additional sampling error because of the proportion of vehicles with nonoperative brake lights.

This discussion is not intended to quench enthusiasm on the conflict-analysis concept but rather to caution potential users and, more importantly, to encourage a more rigorous development of an appropriate data base. For conflict-analysis techniques to be useful, they must embody appropriate definitions and sampling procedures that allow a practical (cost-effective) method to reliably predict the expected annual average number of accidents for a particular site condition.

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Determining Hazardousness of Spot Locations

James I. Taylor,* University of Notre Dame
Harold T. Thompson,* University of Oklahoma

This paper presents a procedure to assess hazardousness at spot locations on all highway facilities except freeways and in central business districts. Indications of hazardousness included in the rating procedure are number of accidents per year, accident rate in terms of annual traffic volumes, accident severity, volume/capacity ratio, sight distance, traffic conflicts, erratic maneuvers, driver expectancy, and information-system deficiencies. A raw-data format was selected for each of these indicators, and a scaling technique was developed that permits the combination of inputs from the several indicators to produce a hazardousness rating on a scale from 0 to 100. The procedure may be used even if data on all indicators are not available for a given site (level of confidence in the results diminishes). Sixteen traffic engineers and safety experts, representing 14 states, were invited to two workshops to review the procedures formulated and to assist in establishing the weights to be assigned to each of the indicators. The general concept underlying the convergence-of-evidence procedure is highly acceptable to the safety personnel who participated in the workshops. Of special note is the development of a workable form for obtaining subjective evaluations of hazardousness in various highway situations.

Within the past decade, safety-improvement programs have received increased attention by the federal government and the various state highway and transportation agencies. Substantial funds have been, and are being, allocated to safety improvements of various types. Implicit in all these programs is the need for a systematic process for identification of hazardous locations and a method for assigning priorities to the treatment of the high-hazard locations identified.

Virtually all identification and priority schemes currently in use are based on computerized accident-record systems. These procedures, although efficient, relatively easy to implement, and generally acceptable, have limitations. The following are some examples of the limitations:

1. Accident records are not always available for all classes of roadways within a given jurisdiction;
2. Considerable disagreement exists as to the specific accident measures that should be included in the hazardous-location identification process (e.g., number of accidents, accident rate, severity, and trend with time), the appropriate form for the measures selected, and the relative weights to be assigned to each measure;
3. Year-to-year consistency of accident experience is lacking at specific locations; and
4. Past accident experience is not appropriate where major changes in geometrics or traffic-control measures have been implemented (e.g., initial signalization of an intersection, change to one-way street operations, major channelization projects, and changes in speed limit) or where major changes in traffic characterictics have occurred (e.g., increased traffic volumes associated with the opening of a new shopping center or apartment complex).

The objective of this study was to develop a procedure for ranking hazardous locations for all highway facilities except freeways and central business districts (CBDs). Both accident and nonaccident measures, or predictors, are included in the formula proposed for establishing the degree of hazardousness (the potential for accidents in the near future) at spot locations within the highway system. If accident measures are available and appropriate, these measures are to be supplemented with the nonaccident measures; if applicable accident records are not available, the nonaccident measures can be used to assess hazardousness in a manner consistent with the more comprehensive formula.

The procedures developed do not address selection of appropriate remedial treatments for the sites identified as hazardous or cost-effectiveness of alternative investment programs.

HAZARDOUSNESS-RATING FORMULA

Potential Indicators

A comprehensive list of both accident-based and nonaccident-related candidate indicators, for inclusion in the procedure, was compiled by a search through relevant literature and as a result of suggestions by traffic engineers and safety experts. The accident-based indicators included in the initial list and derivable from most state accident-record systems are

1. Number of accidents per year,
2. Accident rate,
3. Accident severity,
4. Trend in accident numbers, and
5. Night-to-day ratio.

The objective nonaccident indicators (requiring quantitative measurements but relatively free of subjectivity in data-collection procedures) included in the initial list were

1. Traffic conflicts,
2. Erratic maneuvers,
3. Speed,
4. Speed variance,
5. Acceleration noise,
6. Lateral-placement variance,
7. Headway distribution,
8. Average daily traffic (ADT),
9. Volume/capacity ratio,
10. Percentage of unfamiliar drivers,
11. Traffic violations,
12. Skid resistance,
13. Sight distance, and
14. Access points in vicinity.

The other nonaccident indicators, requiring subjective evaluation on a "good" to "bad" scale, initially considered were

1. Driver expectancy,
2. Adequacy of information system,
3. Evidence of driver errors, and
4. Environmental factors.

Final List of Indicators

The original list of 23 potential indicators was pared to 9 for inclusion in the hazardousness-rating formula (HRF). Personnel from eight state highway and transportation agencies and one major city assisted in selecting the indicators most appropriate for the intended purposes. The 9 indicators are

1. Number of accidents per year,
2. Accident rate,
3. Accident severity,
4. Volume/capacity ratio,
5. Sight distance,
6. Traffic conflicts,
7. Erratic maneuvers,
8. Driver expectancy, and
9. Adequacy of information system (later altered in form to information-system deficiencies to be consistent with procedural format).

Philosophy of the Hazardousness-Rating Procedure

A single procedure flexible enough to be applicable at various types of sites (such as signalized and unsignalized intersections, horizontal curves, and lane drops) is highly desirable because funds must often be allocated to spot improvements as a comprehensive category. The procedures developed are appropriate for the various spot types and, further, can be used even if data on all indicators are not available for a given site (level of confidence in the results would be diminished).

Each indicator is a measure of hazardousness in some degree but is not entirely satisfactory in defining overall hazardousness. The concept underlying HRF is that the composite hazardousness rating provided by the degree of convergence of evidence of the individual indicators provides a reasonably accurate prediction of future accident experience (e.g., restricted sight distance is definitely a factor in the hazardousness at a given location, but analyses of sight-distance restrictions do not provide accurate estimates of future accident experience). The same is true of each of the other indicators, including any of the indicators based on records of past accident experience. Some indicators are better than others; this variance is reflected in the differing weights assigned to the individual indicators.

General Form of HRF

The general form of HRF is

$$HI = \{\Sigma[W_i(IV)_i]\}/\Sigma W_i \tag{1}$$

where

HI = hazardousness index for site under study,

W_i = weighting factor for indicator i,
IV_i = indicator value for indicator i (described below under scaling), and
ΣW_i = sum of weighting factors for all indicators used at study site.

Indicator values range from 0 to 100; larger numbers indicate higher degrees of hazardousness. The sum of the weighting factors for all nine indicators included in HRF is 1.00. However, if data are not available for all the indicators, HI can be normalized to a scale of 0 to 100 by dividing the summation of the weighted indicator values used by the sum of the weights that correspond to the indicators used; i.e., no matter which indicators are available, the range of potential HI at a given site is 0 to 100. Therefore, all sites are rated on a single scale. The greater the value of ΣW_i is, however, the greater the confidence in the results of the rating procedure will be.

Scaling

For the HI derived from the weighted combination of the individual inputs to be meaningful, the raw data for each indicator must be scaled to a value of 0 to 100. Further, the hazardousness implied by a particular IV for one indicator must be consistent with that implied by the same IV for all other indicators. Charts for converting raw data to IVs for each of the nine selected indicators were developed. Four control values were used to establish each of these charts.

1. A value of 0 was used for an indicator raw score that indicated the site made no contribution to hazardousness. For example, a site at which there had been no accidents within the past 3 years would be assigned an IV of 0.
2. A value of 33 was used for an indicator raw score that separated hazardous and normal sites. For example, a site at which there had been an annual average of 2.0 accidents within the past 3 years would be assigned an IV of 33.
3. A value of 67 was used for an indicator raw score that separated very hazardous and critical sites. For example, a site at which there had been an annual average of 10 accidents within the past 3 years would be assigned an IV of 67.
4. A value of 100 was used for an indicator raw score that indicated a higher degree of hazardousness. For example, a site at which there had been an annual average of 50 accidents within the past 3 years would be assigned an IV of 100.

The chart for converting the number of accidents per year to an IV, as based on the four control values described above, is shown in Figure 1. A similar rationale was applied to the other indicators in deriving control values and developing the transformation charts.

Sixteen traffic engineers and safety experts, representing 14 states, were invited to two workshops to review the procedures formulated by the project staff, to assist in establishing the control values to be used on the transformation charts, and to assist in establishing the weights to be assigned to each of the IVs. The methodology was described by the project staff and was then applied by the workshop participants at 12 sites before the final indicator weights were established.

Derivation of HRF

Using the weights that were assigned by the participants at the two workshops, we established the following equation for assessing the hazardousness at various spot locations.

$$
\begin{aligned}
\text{HI} = {} & (0.145)(\text{IV of number of accidents}) \\
& + (0.199)(\text{IV of accident rate}) \\
& + (0.169)(\text{IV of accident severity}) \\
& + (0.073)(\text{IV of volume/capacity ratio}) \\
& + (0.066)(\text{IV of sight distance}) \\
& + (0.053)(\text{IV of traffic conflicts}) \\
& + (0.061)(\text{IV of erratic maneuvers}) \\
& + (0.132)(\text{IV of driver expectancy}) \\
& + (0.102)(\text{IV of information-system deficiencies}) \qquad (2)
\end{aligned}
$$

If all the indicators are not used at a particular site, the right side of the equation must be divided by the sum of the weights (coefficients) for the indicators used. For example, if erratic-maneuvers data are not available, the right side of the equation is divided by (1.000 - 0.061) or 0.939.

RESEARCH RESULTS

The primary product of the research effort is a users manual (2). This manual spells out the procedures to be followed in applying HRF to assess the relative hazardousness of spot locations of interest. The scaling charts and the computation forms necessary for implementing the procedure are also included in the manual.

Sixteen traffic-safety-program personnel from 14 states assisted in developing the final form of HRF and the scaling charts. The inputs of the traffic-safety personnel were derived from their participation in two workshops that were conducted near the conclusion of the project. The participants reviewed a draft of the draft users manual, provided weights for the nine indicators in HRF, visited 12 sites and assessed driver expectancy and information-system deficiencies through the use of the forms developed within the project, and provided an estimate of the relative hazardousness of each of the 12 sites on a scale 0 to 100. These estimates, or ratings, of site hazardousness were to be based on information generally available to safety-program officials (and furnished to the participants) but were to be independent of the specific procedures developed within the project for combining the various raw-data inputs. In fact, the site ratings were made after a field visit to the sites but before the participants were provided data on the subjective indicator ratings of their colleagues or the weights assigned to each indicator.

Because the accident-indicator data and objective non-accident-indicator data (volume/capacity ratio and sight distance, in this case, because collecting traffic conflicts and erratic-maneuver data was not feasible) are available and average ratings for the subjective non-accident indicators were obtained in the workshop, a number of comparisons of consistency among the indicators and the independent site ratings are possible.

Table 1 gives the IVs for each of the indicators, the HI values, and the group site ratings for each of the 12 study sites. Each IV was derived by transforming the indicator raw-score value to an IV through use of the appropriate scaling chart. Data for traffic conflicts and erratic maneuvers were not obtained. The HI values were computed by multiplying each IV by its respective weight, summing these products, and then dividing by the sum of the weights of the indicators used. Group site ratings were derived by averaging the individual ratings assigned by the 16 workshop participants. The weights assigned to the indicators are as follows:

Indicator	Weight	Indicator	Weight
Number of accidents per year	14.5	Volume/capacity ratio	7.3
Accident rate	19.9	Sight distance	6.6
Accident severity	16.9	Driver expectancy	13.2
		Information deficiency	10.2

The correlation coefficients for all pairs of indicators are given in Table 2. For example, the correlation of accident rate to driver expectancy is 0.458.

FINDINGS AND CONCLUSIONS

The findings are based on analyses of data at 12 study sites. Because this is a relatively small sample size for the complexity of the problem, the results of the statistical analysis should be interpreted with caution, and the researcher should exercise caution in generalizing the results to other situations. Furthermore, for a given site the HI value is a weighted average of the individual IVs, and the significance of the correlation coefficient between the HI values and a particular indicator, or

Figure 1. Chart for converting number of accidents to indicator values.

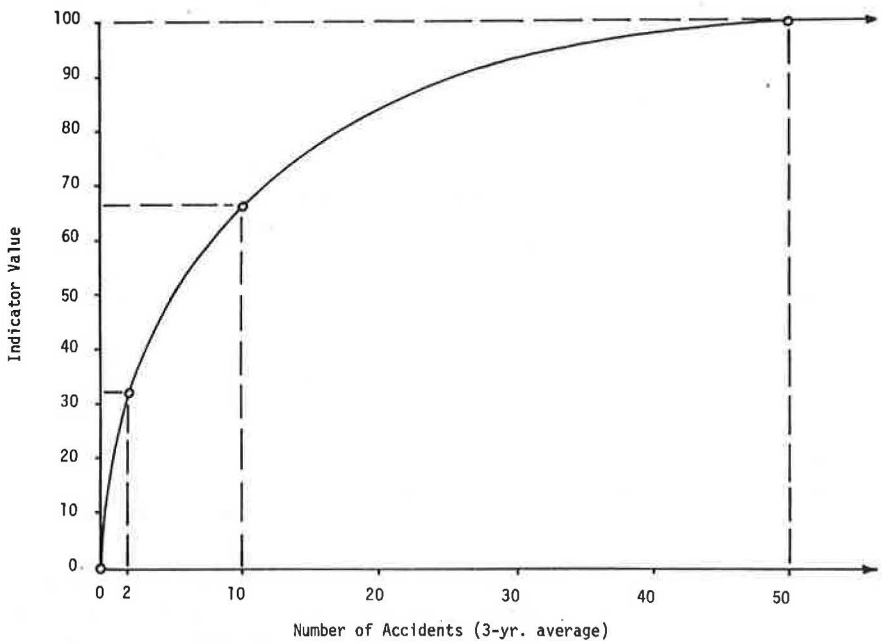

Table 1. Indicator values, HI values, and group site ratings.

Site Number	Indicator Value: Accidents per Year	Accident Rate	Accident Severity	Volume/ Capacity Ratio	Sight Distance	Driver Expectancy	Information Deficiency	HI Values	Group Site Ratings
22 to 48	39	11	42	45	0	25	30	28	22
22 to 97	48	63	66	38	33	86	79	61	80
22 to 98	46	55	68	47	42	41	45	51	46
22 to 99	68	22	44	38	0	51	55	42	38
36 to 4	59	49	70	22	0	37	47	47	42
36 to 6	61	48	63	32	3	38	36	45	43
36 to 26	39	20	69	32	9	41	55	40	33
38 to 18	42	14	43	41	9	44	53	35	22
38 to 31	50	34	60	31	33	60	52	47	57
38 to 32	52	26	41	36	2	21	26	31	32
38 to 37	72	57	44	37	15	39	52	49	56
50 to 1	52	51	61	67	0	56	63	53	58

Table 2. Correlation coefficients.

Indicator Value	Accidents	Accident Rate	Accident Severity	Volume/ Capacity Ratio	Sight Distance	Driver Expectancy	Information Deficiency	HI Values
Accident rate	0.424	–	–	–	–	–	–	–
Accident severity	-0.174	0.549	–	–	–	–	–	–
Volume/capacity ratio	-0.207	0.080	-0.146	–	–	–	–	–
Sight distance	-0.198	0.395	0.366	-0.017	–	–	–	–
Driver expectancy	0.001	0.458	0.406	0.137	0.405	–	–	–
Information deficiency	0.031	0.420	0.382	0.191	0.269	0.916	–	–
HI values	0.291	0.891	0.647	0.125	0.487	0.781	0.746	–
Group-site ratings	0.305	0.841	0.473	0.103	0.458	0.792	0.692	0.928

Figure 2. Rating form for driver expectancy problems.

Ratings:

0 -- Nothing unexpected or unusual at this location.

Actions required (if any) entirely consistent with driving strategy on approach.

Standard geometry, with pathway(s) for intended movement(s) clearly evident.

No interferences by other traffic likely.

1 --

2 --

3 -- Situation somewhat unexpected.

Driver must be alert, but should be able to respond adequately at "last minute" to most combinations of adverse circumstances.

Some initial confusion on intended path(s) or movement(s).

Interference from other traffic may create some degree of confusion or uncertainty for average driver.

4 --

5 --

6 --Very unusual situation; will "surprise" many unfamiliar drivers.

Driver required to make major change in driving tactics from those employed over past few miles.

At least a "near accident" almost expected if driver is even moderately inattentive; evasive actions likely to be required.

Intended pathway(s) confusing under fairly normal traffic or lighting conditions.

Other traffic, or lack of it, aggravates situation and misleads driver or deprives him of important cues.

Approach	Rating						
	0	1	2	3	4	5	6
A							
B							
C							
D							

group of indicators, must be interpreted in that light.

Findings

1. Correlation coefficients given in Table 2 indicate relatively low correlations between all pairs of individual indicators except for driver expectancy and information-system deficiencies. The low coefficients may be interpreted as an indication of the independence of the indicators. The higher correlation between driver expectancy and information-system deficiencies (0.916) indicates a strong relationship between the two subjective indicators. This relationship, in turn, means that reformulating the two indicator rating forms so that they better reflect two different aspects of hazardousness or perhaps combining the two forms into a single subjective indicator may be advisable; i.e., not much useful information is derived by including the second subjective indicator in the present form.

2. The high correlation between the accident rate and group site ratings (0.841) indicates that the safety experts place considerable emphasis on accident rate in estimating overall site hazardousness. (This emphasis is confirmed by their assigning the highest weight to the accident-rate indicator.)

3. Although the number-of-accidents indicator carries a higher weight in determining HI (as assigned by the workshop participants), the two subjective indicators correlate better with the group site ratings. This result may indicate that safety experts place a higher value on their subjective opinions, based on field examinations of the sites, than they express under the formalism of written relative weight assignments.

4. The group site ratings correlate highly with HI values (0.928). This result can be interpreted in at least two ways.

a. The two values are largely independent but, because they are both measures of true hazardousness, a high correlation coefficient is to be expected. In effect, the HRF procedure breaks down the assessment of hazardousness to a series of complementary value judgments. First the indicators were selected, then an appropriate format for the raw-data inputs was devised, the scaling charts were developed, and finally weights were assigned to each indicator. On the other hand, site rating is a single-value judgment and involves informal integration of raw data inputs by the individual. If this interpretation is accepted, one has the choice of employing HRF or the collective judgment of 16 safety experts. A secondary analysis indicates that the mean of the correlation coefficients between each individual's site ratings and HIs was 0.784; the range was from 0.560 to 0.874; therefore, an individual is not likely to assess the true hazardousness nearly as well as HRF.

b. The two values were not really arrived at independently; i.e., even though efforts were made to discourage the participants from employing the HRF concept in their assessment of the site hazardousness, they did use the workshop techniques in integrating the raw-data inputs. (Enough control was exerted to ensure that they did not use the scaling charts and numerical forms directly.)

5. The objective non-accident indicators did not correlate nearly so well with the group site ratings as the other classifications of indicators. This result may mean that appropriate data formats and scaling charts have not been formulated for the sight distance and volume/capacity ratio indicators, which have considerable intuitive appeal. In fact, these two indicators were selected from a large number of potential indicators during the early stages of this project.

Conclusions

1. The concept of HRF to assess relative hazardousness at spot locations appears to be valid, based on the results of the workshops and limited statistical analyses.

2. The concept was highly acceptable to the safety-program personnel who participated in the workshops where the procedures were discussed in detail and employed in the field. In fact, more than 90 percent of those attending the workshop rated the concept as very worthwhile and deserving of further development and testing. (This conclusion was derived from an end-of-workshop questionnaire administered by the Federal Highway Administration.)

3. Development of an effective, workable rating form for quantifying subjective, nonaccident indicators was accomplished by the project (Figure 2). In testing the rating forms for the 12 study sites, we observed that consistency among participants increased with familiarity, which indicates that the subjective indicators might be consistently quantified through the rating forms provided. Further, comparison of indicator weightings assigned at the beginning of the workshop with those assigned at the end of the workshop show that the weights for the subjective indicators were increased considerably after the participants had used the forms and procedures.

4. A users manual (2) developed within this project is a workable document. The workshop participants reviewed and used draft copies of the manual; only minor revisions were suggested, and these have been incorporated in the final draft.

PROGRAMMATIC APPLICATION

Collecting all the indicator data at all spot locations within a particular jurisdiction is not practical. Some of the indicators (particularly traffic conflicts and erratic maneuvers) require extensive data-collection efforts; use of any non-accident-related indicators requires a visit to the site, at a minimum.

Therefore, using the HRF methodology as a screening process is not feasible; the value of HRF lies in comparative assessment of hazardousness of sites of varying characteristics and with differences in the assessment data available or collectible. The methodology is particularly advantageous if one desires to include sites with and without accident histories in a single, comprehensive evaluation scheme.

A possible procedure for identifying hazardous locations and assessing their relative hazardousness in a specific jurisdiction is as follows:

1. Select the top 20 sites (an arbitrary number but perhaps twice the number for which treatment funds are likely to be available) on the basis of the accident-records system alone (this screening process can be accomplished by developing a computer program and format to provide partial hazardousness indexes on the basis of the first three terms of HRF),

2. Add 5 sites for which a number of citizen complaints have been registered,

3. Add 5 sites that the safety officials know to be hazardous even though few or no accidents have occurred (perhaps because of chance, new construction, or major change in operational characteristics),

4. Collect the non-accident-indicator data for these 30 sites, and

5. Compute the relative hazardousness of the 30 sites on the basis of the comprehensive HRF.

RECOMMENDED FUTURE RESEARCH

This research effort, although limited in scope and sample size, indicates that the concept of an HI scheme is valid and acceptable to the highway safety community. More than 90 percent of the participants introduced to the concepts and procedures at the two workshops indicated that they felt further development is warranted. The following specific areas are suggested for future research efforts.

1. Large-scale, long-range validation is needed. A possible procedure would be to rank a large number of sites in a given state by the HI and the priority-ranking scheme currently employed by that state. Analysis of the accident experience at those sites in the following 3 years should give an indication of which method most accurately defines future accident potential.

2. The scaling charts should be refined. Compilation and analysis of the distributions of raw-data scores to be encountered in the various indicators would permit development of scaling charts with more consistent meanings among indicators for given IVs. For example, a value of 67 could be assigned to the raw score, which is exceeded in only 1 percent of all cases encountered.

3. The traffic-conflict and erratic-maneuvers indicators should be developed. Giving adequate attention to traffic conflicts and erratic maneuvers was not possible within the constraints of this project. As a result, the IV curves derived for these indicators are the most suspect and are not backed by any use within the workshop.

4. HRF should be incorporated into safety-improvement programs. Although incorporation would call for the opening of a wider area of research than that of identification of hazardous locations, a methodology to assess the benefits of potential remedial treatments (in terms of reductions in HI) must be developed before the techniques developed within this project can be fully effective in the allocation of funds for safety-improvement programs.

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**Mr. Taylor and Mr. Thompson were with Pennsylvania State University when this research was performed.*

Evaluation of Freeway-Merging Safety as Influenced by Ramp-Metering Control

Bartholomew T. Cima,* Peat, Marwick, Mitchell and Company

The traffic-conflict technique was modified to evaluate the relative safety of freeway merging with and without the use of entrance ramp-metering control. Six types of traffic conflicts were defined for the entrance ramp and acceleration lane: braking on ramp, braking for lead vehicle, weaving around lead vehicle, entering second lane, entering side by side, and entering late. Five conflicts were specified for the freeway lane (merge lane) adjacent to the acceleration lane: weaving around entering vehicle, braking for entering vehicle, weaving around lead and entering vehicles, braking for lead and entering vehicles, and avoiding encroaching vehicle. A three-level severity rating (routine, moderate, and serious) was also developed to assess the seriousness of each conflict. An existing ramp-metering control installation was investigated during freeway levels of service C and D. A two-way analysis of variance was performed on the traffic-conflict data by using, as the independent variables, ramp-control condition (on and off) and freeway level of service (C and D). The study revealed a significant reduction of 11.6 percent in all traffic conflicts when ramp control was activated. Analysis results indicate that acceleration-lane conflicts significantly decreased when ramp-metering control was used. Merge-lane conflicts were found to be related more to freeway level of service than to ramp control. However, merge-lane, multiple-vehicle conflicts and their severity decreased when ramp control was in effect. An analysis of accident records supported these conclusions.

The use of ramp-metering control has been shown to reduce both freeway congestion and accidents (1). Most of these control-system evaluations concerned the benefits obtained for the entire affected freeway section and especially the relationship between the operational safety of freeway merging and the effect of ramp metering. To measure safety, previous studies generally relied on accident records that can require a considerable length of time to accumulate. To quickly capture the localized safety effects of ramp-metering control, a measure is needed that reflects the relative safety of driver behavior during the merging maneuver.

The following study was designed to evaluate freeway-merging safety as influenced by ramp-metering control. A modified traffic-conflict technique (TCT) was developed for the rapid assessment of the safety contribution of metering, separate from the broader aim of reducing congestion. This technique was used to appraise merging safety with and without the use of control at an existing metered entrance ramp.

FREEWAY-MERGING TCT

The TCT involves the systematic surveillance and recording of defined driver behavior at a highway location. The traffic-conflict data are collected by a team that is either observing at the site or viewing the visual recordings.

The TCT employed in this study is a combination of the methods developed at the U.K. Transport and Road Research Laboratory (2, 3) and by Perkins and Harris (4). The U.K. procedure defines a traffic conflict as a situation in which a driver takes evasive action to avoid a collision. Each conflict is ranked according to a five-point severity scale that ranges from a precautionary maneuver to "emergency action followed by a collision" (2). The technique developed by Perkins and Harris defined certain driver behaviors to be traffic conflicts for over 20 intersection-accident patterns (4).

For the freeway-merging application, 11 traffic conflicts were identified based on freeway-accident patterns. These conflicts were divided into acceleration-lane conflicts that occur on the freeway-entrance ramp and the connecting acceleration lane and merge-lane conflicts that occur on the mainline freeway lane adjacent to the entrance ramp. The conflicts were then classified by severity. The six acceleration-lane conflicts are defined below.

1. Braking on ramp occurs when the speed of a single merging vehicle must be reduced on the acceleration lane because no acceptable gap appears in the freeway traffic stream (Figure 1). A brake light signals this situation. A slowing vehicle is a routine conflict, but a stopped vehicle is a moderate conflict.
2. Braking for lead vehicle occurs when the lead vehicle in a platoon of vehicles entering the freeway causes any of the following vehicles to be braked or stopped (Figure 2). A brake light indicates this situation. A stopped vehicle is a moderate conflict.
3. Weaving around lead vehicle occurs when a following vehicle is merged into the mainstream flow ahead of a lead vehicle (Figure 3). The degree to which the lead vehicle is affected by the following vehicle determines the severity of the conflict.
4. Entering second lane occurs when a merging vehicle enters the freeway and crosses immediately to lane 2 or the center lane of the mainline (Figure 4). The severity of this maneuver is determined by its smoothness, speed, and angle of entry. A high-angle, fast, fishtail entrance is considered more dangerous than a small-angle, controlled merge.
5. Entering side by side occurs when two entering vehicles arrive at the acceleration lane at the same time and are positioned side by side (Figure 5). The danger created by these vehicles as drivers accommodate each other's movements determines severity.
6. Entering late occurs when an entering vehicle reaches the end of the acceleration lane and traverses the shoulder before merging (Figure 6). Severity can be judged by the driver's control during the maneuver and by the nearness of collision between the entering vehicle and any mainline freeway vehicles.

The five merge-lane conflicts are defined as follows:

1. Weaving around entering vehicle occurs when a mainline vehicle in the merge lane must change lanes to avoid a merging vehicle (Figure 7). The severity of this conflict is assessed by the smoothness of the maneuver and the danger of collision between the mainline and entering vehicles.
2. Braking for entering vehicle occurs when a mainline vehicle must reduce its speed because of an entering vehicle (Figure 8). The conflict is signaled by a brake light. A conflict in which a freeway vehicle slows quickly and comes close to the entering vehicle is ranked more severely than a conflict in which a freeway vehicle slows only

slightly and does not come near the merging vehicle.

3. Weaving around lead and entering vehicles occurs when a mainline vehicle slows for an entering vehicle and causes a following mainline vehicle to change lanes (Figure 9). The lead mainline vehicle need not be involved for this type of conflict to occur, but the presence of an entering vehicle is required. Severity is determined by the smoothness of the lane-changing maneuver and the chance of contact between the mainline lead and following vehicles.

4. Braking for lead and entering vehicles occurs when a mainline vehicle is braked for an entering vehicle and causes the following mainline vehicle to slow (Figure 10). A brake light indicates this situation. The lead vehicle need not be involved in this conflict. Severity is based on the degree of braking of the following vehicle and the danger of collision between the two mainline vehicles.

5. Avoiding encroaching vehicles occurs when a mainline vehicle moves into the adjacent lane to avoid an entering vehicle but does not change lanes (Figure 11). Severity is determined by the degree of swerving involved during the maneuver.

Each conflict is also classified according to the three-point severity scale given below.

1. A routine conflict involves precautionary braking or lane changing when the risk of collision is small. For example, a freeway driver might feel threatened by a merging vehicle and change speed or position although the chance of contact is slight.

2. A moderate conflict involves controlled braking or lane changing to avoid a situation with high collision potential. This maneuver clearly requires controlled evasive action.

3. A serious conflict involves rapid deceleration, swerving to change lanes, or stopping to avoid a collision. The driver has no time for a controlled maneuver. Often termed "a very near miss," this maneuver involves fish tailing and causes forward lurching of a vehicle being quickly stopped. This conflict is similar to the one used by the Washington Department of Highways for intersection-conflict counts (5).

The traffic-conflict data were obtained in time intervals of 5 min to be consistent with the calculation method for determining freeway peak-hour factors. Additional information concerning ramp and freeway volumes and environmental conditions was collected during the course of the study.

Figure 1. Braking-on-ramp conflict.

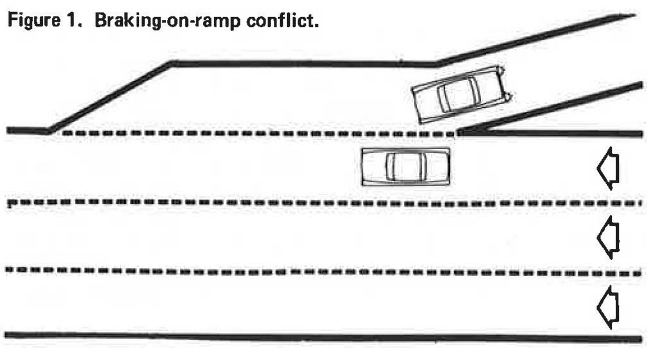


Figure 2. Braking-for-lead-vehicle conflict.

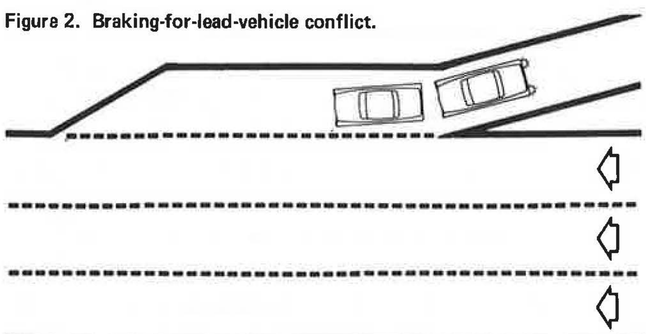


Figure 3. Weaving-around-lead-vehicle conflict.

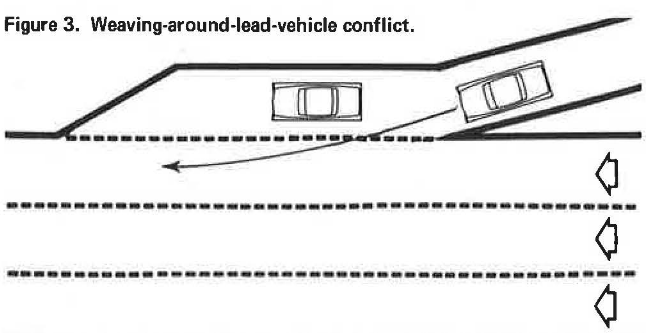


Figure 4. Entering-second-lane conflict.

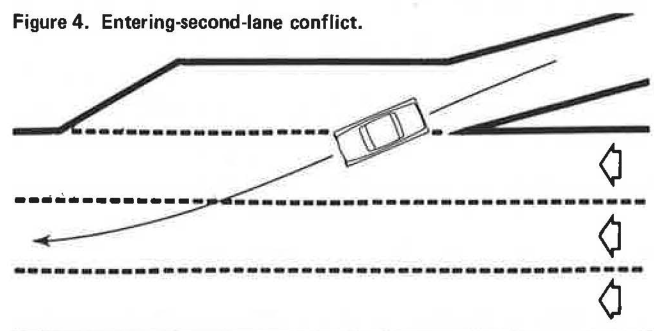


Figure 5. Entering-side-by-side conflict.

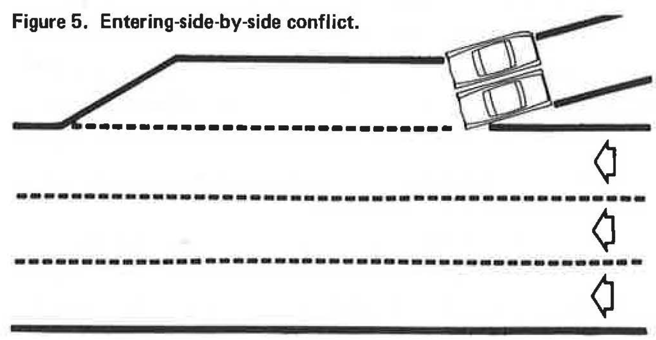
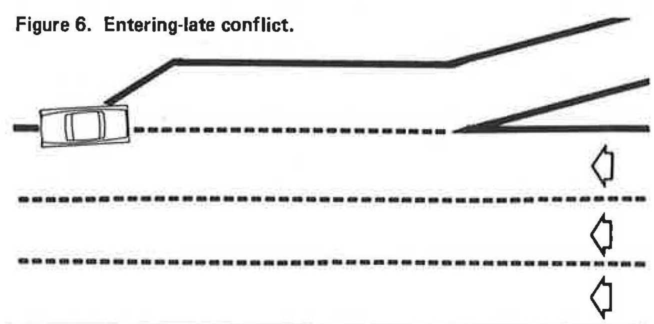


Figure 6. Entering-late conflict.



STUDY DESIGN

The goal of traffic-control devices is to modify driver performance and thus promote the efficient and safe flow of traffic. Focusing on the safety aspect of traffic-control devices, the engineer seeks to direct the behavior of the driver to reduce driving hazards. The control devices guide, warn, and regulate traffic movements. Assuming traffic conflicts reflect the safety of a highway site, we measure driver behavior to obtain an objective measurement of roadway safety. Evaluating the effects of controlled stimuli on human behavior is a major concern within the field of psychology, which has provided a study design for evaluating the safety effects of ramp-metering control.

The purpose of a psychological experiment is to investigate the relationship between the stimulus, in this case ramp-metering control, and the target behavior, i.e., traffic conflicts. The reversal design provides an appropriate way to explore this relationship.

In this study design, the number of occurrences of target behaviors under original environmental conditions is determined. Then the stimulus is introduced into the environment, and again the target behavior occurrences are counted. A change may be evident at this point, but the cause of the change is unclear. The next step is to remove the stimulus and survey the behaviors. The reversal design receives its name from this step. The environment is reversed to the state in which the stimulus was absent. Finally, the stimulus is reintroduced and the result is recorded. Evidence that the stimulus is responsible for the change is provided if the occurrence of target behaviors in the stimulus phase changes relative to the occurrence under the original conditions, returns to the occurrence under the original conditions when the stimulus is removed, and again changes in the final stimulus phase (6).

For this study, users of the selected entrance ramp were first presented with the metering signal operating in a one-vehicle-at-a-time mode (ramp control on), because this was the existing condition. No data were collected on days when the road was wet. Under this control condition, data were collected for 5 dry, nonholiday weekdays (period 1). On the following weekday, the metering signal was set to present a constant green indication (ramp control off). These signals rest in this state whenever control is not applied. Again, data were collected for 5 dry, nonholiday weekdays (period 2). At this point, control was turned on again, and data were collected for 3 d (period 3). Finally, control was turned off, and data were collected for 3 d (period 4). Thus, data were collected for 8 d under each control condition.

STUDY SITE

The metering installation selected for study is located at the Wilson Avenue entrance ramp to northbound Interstate 94. The site is just north of the junction between Ill-194 and I-94. The interchange, a half-diamond design typical of urban areas, joins the three-lane freeway on a level segment at the end of a 3°curve. The acceleration-lane taper is 198 m (650 ft) from the ramp nose, which yields a usable length of approximately 183 m (600 ft). The ramp-metering signal is 91 m (300 ft) from the ramp nose.

Data were collected between 3:30 and 5:30 p.m. just

Figure 7. Weaving-around-entering-vehicle conflict.

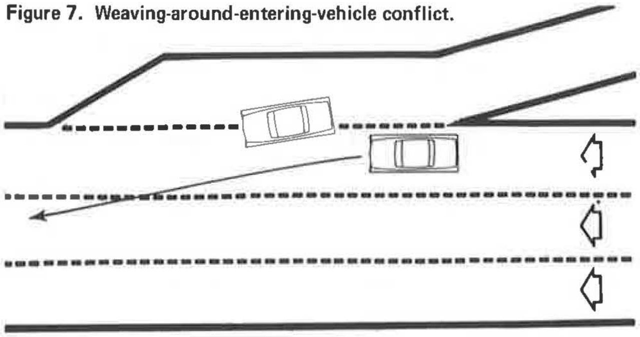


Figure 8. Braking-for-entering-vehicle conflict.

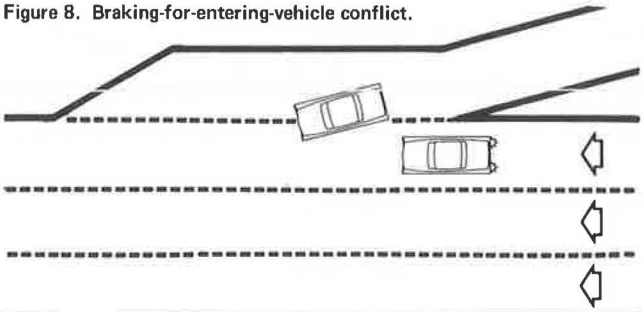


Figure 9. Weaving-around-lead-and-entering-vehicles conflict.

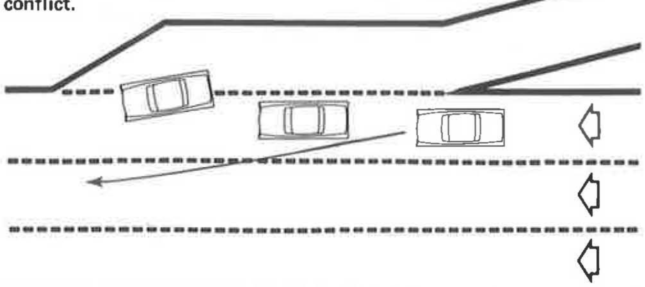


Figure 10. Braking-for-lead-and-entering-vehicles conflict.

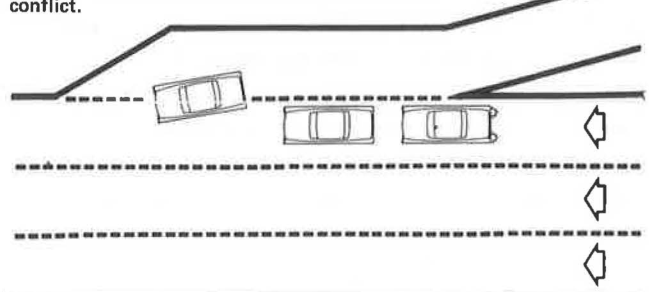
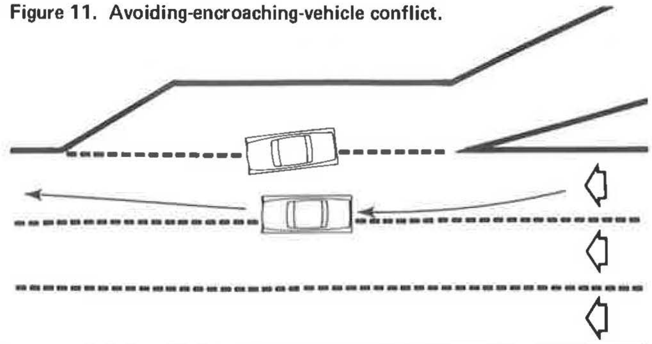


Figure 11. Avoiding-encroaching-vehicle conflict.



before the onset of stop-and-go traffic conditions. Average freeway volume during this 2-h period is 4300 vehicles/h upstream of the ramp, and average ramp volume is 700 vehicles/h. When on, the ramp-metering control was kept at a constant rate of 13 vehicles/min throughout the study.

DATA ANALYSIS

The 5-min interval for traffic-conflict counts was used as the basic unit of analysis. All results relate to the number of conflicts per 5 min. The study design planned for a total of 336 count intervals. But because of poor visibility, bad weather, and equipment problems, the number of usable intervals was reduced to 242. This reduction results in 85 count intervals for the first 5 d of the study when ramp control was on (period 1), 81 count intervals for the next 5 d when control was off (period 2), 38 count intervals for the next 3 d when control was on (period 3), and 38 count intevals for the last 3 d when control was off (period 4).

The investigation of the relationship between ramp-metering control and traffic conflicts requires an accounting for confounding factors. Environmental conditions have been dealt with by not considering count intervals that occurred during bad weather or poor-visibility conditions. However, there is evidence that traffic conflicts increase with volume (3). Therefore, variations in volume from day to day could affect the traffic-conflict data. One way to determine the possible effect of these daily variations would be to perform an analysis of variance (ANOVA) by using traffic flows as the dependent variable and the individual study days as the independent variable.

The two traffic characteristics chosen were the entrance-ramp and merge-lane volumes expressed in 5-min flow rates. Traffic conflicts measure the interaction of these two traffic streams. Based on the ANOVA results, the hypothesis that there is no difference among the study days for entrance-ramp volumes could not be rejected at the 5 percent significance level. The F-ratio (ratio of the larger mean square to the smaller mean square) is 1.008, which indicates near equality of within-day and between-day variances.

For merge-lane flows, the hypothesis that there is no difference among study days can be rejected at the 5 percent significance level. Thus, merge-lane conflicts can possibly be affected by traffic-flow variations.

A reversal design was selected for use in this study. This type of design attempts to demonstrate a causal link between an experimental condition and a target behavior by alternating the presence and absence of the condition. If ramp-metering control can increase the safety of freeway merging, traffic conflicts can be expected to decrease when ramp signals are on and to increase when ramp signals are off. The expected pattern according to the experimental design would be that the level of conflicts would be low during period 1, increase during period 2, decrease during period 3, and increase during period 4. One could reasonably expect that the difference between the two periods when ramp control was on and the difference between the two periods when ramp control was off would be significant.

Three aggregated conflicts were used to investigate the trend as the ramp-metering control was turned on and off according to the experimental design. The 5-min-interval count summations of all conflicts, all acceleration-lane conflicts, and all merge-lane conflicts were chosen as representative measures of the underlying process.

A heuristic analysis was made of the conflict trends by plotting the daily averages of the aggregated conflicts by study day (Figures 12, 13, and 14). The vertical line separates experimental periods, and the dashed horizontal line indicates the period mean. The plot of all conflicts (Figure 12) shows that the basic predicted trend holds; however, some daily averages overlap. The measure is the sum of the acceleration-lane and merge-lane conflicts and reflects driver behavior. Acceleration-lane conflicts conform to the predicted pattern as shown by Figure 13. However, the merge-lane conflicts do not match the predicted pattern (Figure 14). Factors such as traffic volume possibly affect the results. The plot also helps to account for driver behavior in all conflicts, especially in period 4.

The statistical significance of these observed trends can be found by testing for the differences between the means of the aggregated traffic conflicts for each possible pair of experimental periods. There are six possible combinations of these periods: four that compare ramp-control-on conditions with ramp-control-off conditions and two that compare like conditions. The Student's t-test was employed to test the differences in the means at the 5 percent level. A one-tailed test was used to compare the four different experimental condition pairs. This test was chosen because the major interest of this study is to demonstrate that more conflicts occur when ramp control is off than when ramp control is on. A one-tailed test is more restrictive in rejecting the null hypothesis when there is no difference and has a lesser risk of making a type II error. For the two like-condition pairs, the two-tailed test was chosen because there is no interest in the direction of the relationship. The results of the tests for the means are given in Table 1. The results for all traffic conflicts show a difference for period 1 comparisons and no difference for period 3 comparisons. This disparity could be due to two reasons. First, the merge-lane conflicts that are part of this summary do not conform in period 4 and are higher than hoped for in period 3. Second, the smaller sample size may have caused periods 3 and 4 to be less representative.

The analysis of test results for acceleration-lane conflicts shows that the conflicts follow the expected pattern. Having controlled for other rival factors, we can state that the use of ramp-metering control helps reduce the occurrence of acceleration-lane conflicts and thus eases driver tension and increases merging safety for the vehicle entering the freeway.

Figure 14 shows that merge-lane conflicts have no relationship to ramp-metering control. Traffic characteristics appear to have a significant effect on these types of conflicts.

The relationship between selected traffic conflicts and ramp control is investigated later in this paper. Because no statistical differences were found between the periods with the same experimental conditions, further analysis will be concerned only with comparing the data for the 8 d for which ramp control was on with data for the 8 d for which control was off.

A further division is made by determining freeway level of service (LOS) during each count interval (7). All traffic conditions during the study were classified as LOS C or D. This classification allows for the further investigation of the relationship between traffic conflicts and traffic characteristics. Two-way ANOVA was used to determine statistical significance; ramp control and freeway LOS were independent variables.

The ANOVA results for the aggregation of all traffic conflicts showed a significant effect at the 5 percent level for both ramp control and freeway LOS. Ramp metering reduces traffic conflicts 11.6 percent. The significance of LOS supports the concept that some conflicts are related to traffic characteristics. That the number of

Figure 12. All conflicts by study day.

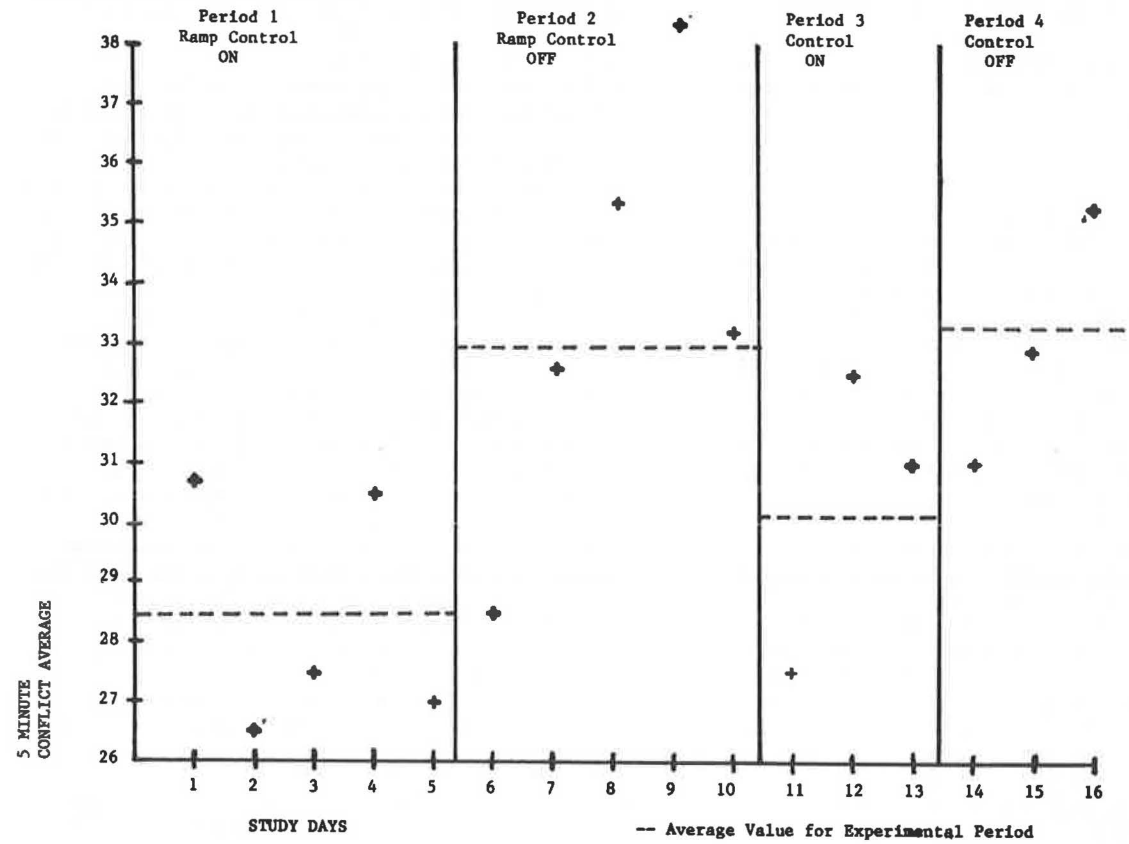
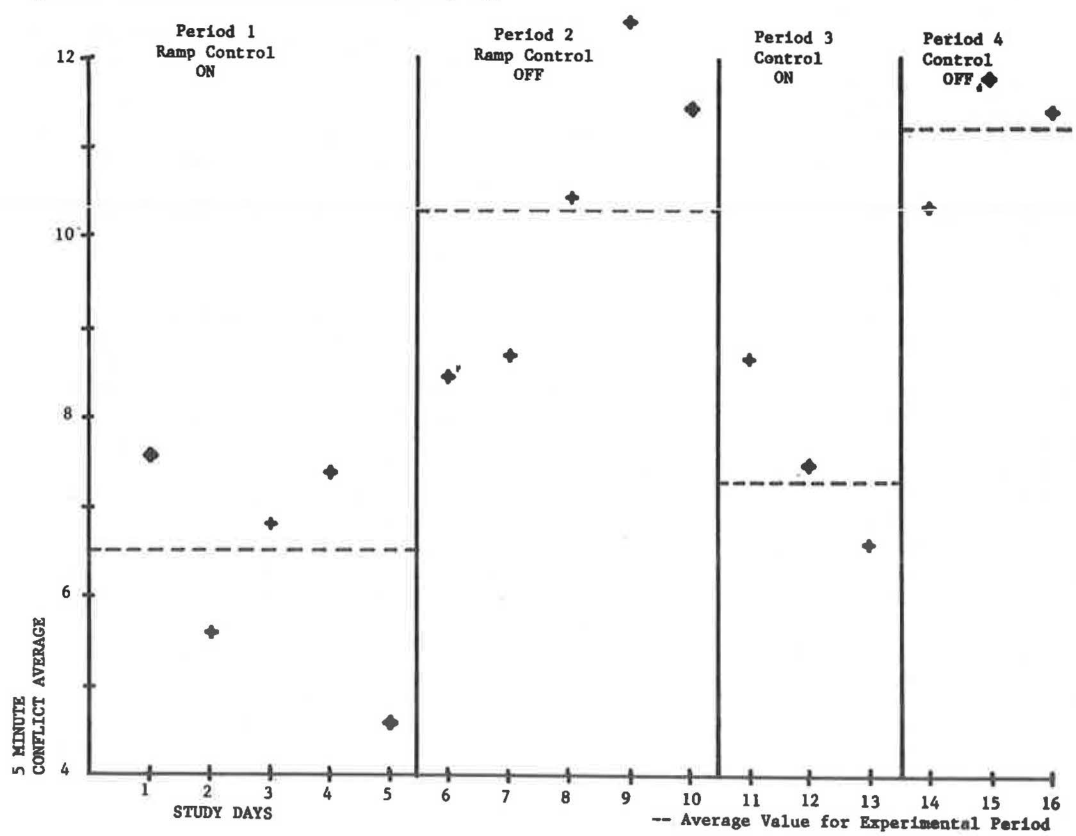


Figure 13. All acceleration-lane conflicts by study day.



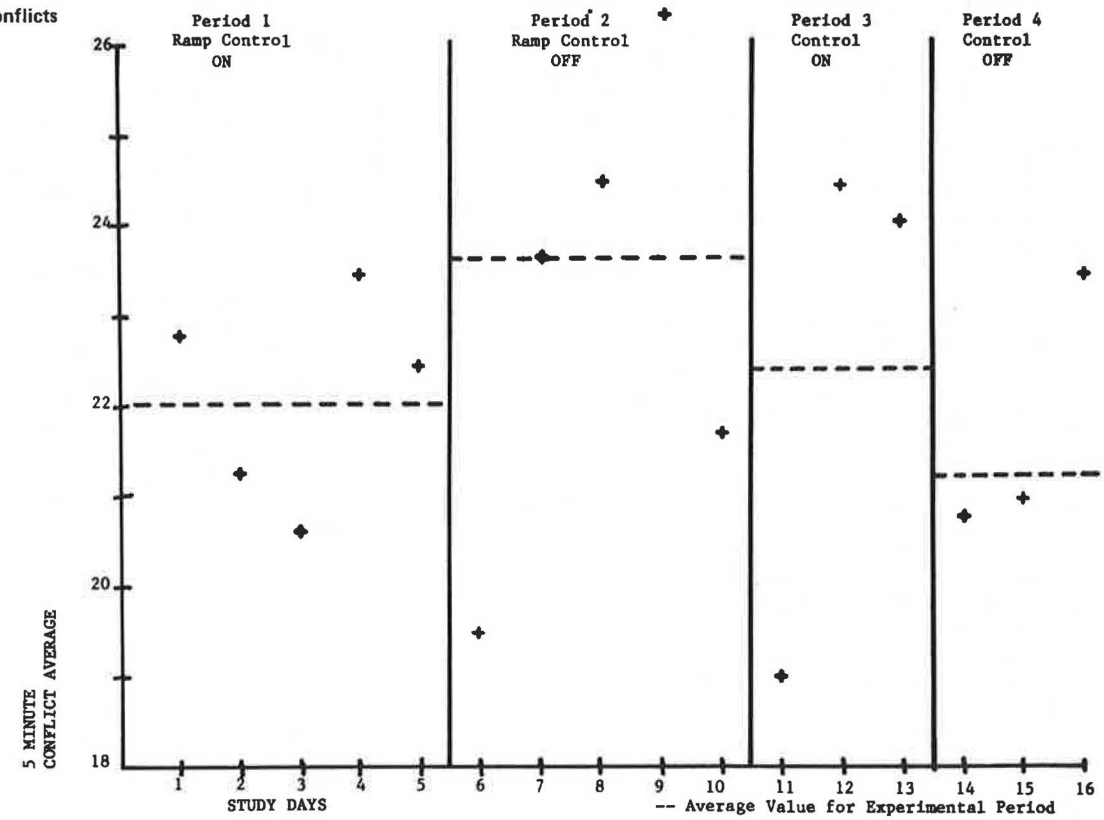


Figure 14. All merge-lane conflicts by study day.

Table 1. Results of tests of statistical differences between means of aggregated conflicts.

Test	Ramp Control	Test Periods	Traffic Conflicts	Acceleration-Lane Conflicts	Merge-Lane Conflicts
One-tailed	On and off	1 and 2	Yes	Yes	No
		1 and 4	Yes	Yes	No
		3 and 2	No	Yes	No
		3 and 4	No	Yes	No
Two-tailed	On	1 and 3	No	No	No
	Off	2 and 4	No	No	No

Note: Yes indicates that the hypothesis that there is no difference between the means can be rejected, and no indicates that the hypothesis cannot be rejected.

conflicts increases as freeway LOS changes from C to D under both control condition offers additional evidence.

For all merge-lane conflicts, the ANOVA indicates, as expected, no effect for metering but an effect due to LOS at the 5 percent level of significance. There is an average increase of 11.3 percent in all merge-lane conflicts when traffic moves from LOS C to D. This result confirms the concept that these types of conflicts are related to variation in traffic flow.

The ANOVA results for the five individual kinds of merge-lane traffic conflicts provide additional insight into the interaction between metering conflicts and traffic characteristics. The ANOVA for all rear-end conflicts indicates no effect for ramp control but a significant effect for LOS at the 5 percent level. This result is in agreement with early results. As freeway level of service decreases, the number of speed adjustments required to maintain a safe spacing between mainline vehicles increases. This relationship is subsequently reflected in the number of brake-light indications. The same results were found for all braking-for-lead-and-entering-vehicle conflicts.

An examination of the ANOVA for all weaving-around-lead-and-entering-vehicle conflicts shows a significant effect at the 5 percent level for both ramp control and LOS. This conflict partially measures the magnitude of multiple-vehicle involvements in conflict situations. This outcome suggests that more mainline vehicles are adversely affected by merging vehicles when ramp control is off than when ramp control is on.

The ANOVA results for all weaving-around-entering-vehicle conflicts show no significant effect. The analysis of all avoiding-encroaching-vehicle conflicts could not be made because of a small sample size. Thus, for braking-for-entering-vehicle conflicts, the major link is with freeway level of service. However, the weaving-around-lead-and-entering-vehicle results indicate that multiple-vehicle merges, which only occur when ramp control is not in use, cause more mainline vehicles to be involved in the same conflict situation. The ANOVA results for all acceleration-lane conflicts show a significant effect for ramp control and none for level of service, as expected.

During data collection, the following sequence of events was observed. The driver of a lead vehicle in the platoon of entering vehicles would merge at a speed that was adequate for his lead vehicle to match its gap. However, the driver of a following vehicle apparently would perceive that the speed of the lead vehicle would not allow his vehicle to merge. Thus, the following-vehicle driver would speed up and enter directly into lane 2 or decrease his vehicle's speed. The first case resulted in an entering-second-lane conflict, and the second case resulted in a braking-for-lead-vehicle conflict.

The analysis of all entering-second-lane conflicts showed a significant effect for control and no effect for LOS. When ramp control is on, the occurrence of those conflicts decreases for both levels of service. When ramp control is off, the opportunity for this type of maneuver decreases with level of service.

An examination of the ANOVA for braking-for-lead-vehicle conflicts shows a significant effect for control and none for level of service. Therefore, use of ramp control seems to reduce occurrence of braking by following vehicles during freeway merging.

The two-way ANOVA results for all braking-on-ramp conflicts indicate a significant effect for level of service and none for control. This result is not unexpected. This conflict specifically records the number of times that drivers of entering vehicles are unable to find an acceptable gap within the freeway traffic stream and are forced to reduce speed. As the level of service and subsequently the number of gaps decrease, the occurrence of this conflict increases.

For all entering-late conflicts and all weaving-around-lead-vehicle conflicts, the analysis is inconclusive because of small sample sizes.

As demonstrated above, when ramp control is on the number of acceleration-lane conflicts decreases and merging safety increases. This gain is mainly accomplished by reducing the interference between entering vehicles as manifested by the reduction in entering-second-lane and braking-for-lead-vehicle conflicts. The merging driver is able to give more attention to finding a suitable gap in the freeway traffic stream.

Each traffic conflict, in addition to being classified as one of the 11 individual conflicts, was also categorized into one of three severity ratings (routine, moderate, or serious). No serious conflicts were observed during the study.

The ANOVA results for the aggregation of all routine traffic conflicts show a significant effect for both metering and freeway LOS at the 5 percent level of significance. This outcome is in agreement with earlier analyses. However, the ANOVA for all moderate conflicts reveals only an effect for ramp control. The implication is that when metering is on the severity of conflicts is affected.

Further evidence is found in the ANOVA for all routine and all moderate merge-lane, rear-end conflicts. This type of conflict is the aggregation of braking-for-entering-vehicle and braking-for-lead-and-entering-vehicle conflicts. The ANOVA for the routine conflicts shows a significant effect only for LOS, as seen in previous results. The analysis for the moderate occurrences indicates a significant effect only for ramp control.

ACCIDENT ANALYSIS

Information was obtained from accident records of all reported freeway accidents occurring on northbound I-94 on a 0.603-km (0.375-mile) segment including the Wilson Avenue entrance ramp. The data spanned 8 years; data were collected 4 years before ramp metering was installed in March 1971 and 4 years after the installation. For the before period, only accidents occurring between the probable hours of ramp metering, 3:00 to 7:00 p.m., are considered for the analysis. For the after period, only those collisions that happened when ramp metering was on are used. Because ramp metering is manually activated according to traffic conditions, which results in daily time variations and the long time period for the accident data collection, a strict before-and-after comparison is difficult to make.

The list of property-damage accidents showed that the number of collisions in the study section decreased from 32 accidents before the installation of metering to 21 accidents after the installation, a 35 percent reduction. Based on a Poisson distribution for the accidents and an analysis technique developed by Michaels (8), this reduction is statistically significant at the 5 percent level, indicating that there is an improvement in accident experience when ramp metering is used.

The accident analysis supports the finding of increased safety reported elsewhere (1). But the analysis also serves to illustrate some of the problems encountered when accident records are used to evaluate a time-dependent traffic improvement. The modified traffic-conflict technique established a definite link between increased freeway-merging safety and ramp-metering control. Accident records are the prime measure of highway safety, but in the evaluation of certain traffic improvements need to be supplemented to provide a comprehensive analysis.

CONCLUSIONS

Based on the results of the data analysis, the use of ramp-metering control set at 13 vehicles/min helps to increase the safety of freeway merging during levels of service C and D. During this study an 11.6 percent reduction in all traffic conflicts occurred when metering was on.

Acceleration-lane conflicts exhibited the strongest link with the use of ramp control. The aggregation of all acceleration-lane conflicts followed exactly the pattern predicted by the reversal design. Ramp-metering control appears to reduce the interference between merging vehicles. This observation is demonstrated by the reduction in braking-on-ramp and entering-second-lane conflicts when metering is on.

Merge-lane conflicts are more strongly related to freeway level of service than to ramp control. However, fewer multiple-vehicle involvements occur when ramp control is on, and the severity of the merge-lane involvements decreases with the use of control.

This application of the TCT has demonstrated its usefulness in a freeway-merging setting. The accident analysis illustrated the ability of the TCT to provide a rapid and valid evaluation. The method furnished useful and reliable data that provide insights into the merging process.

The further testing of this modified technique at other entrance ramps is warranted. Testing would provide additional information concerning merging safety and insight into the operation of the technique. Applicability of the traffic-conflict technique at different entrance-ramp configurations, at other metering rates, and during various freeway traffic conditions (especially under congested conditions) still needs to be investigated.

ACKNOWLEDGMENTS

This study was conducted as part of the Illinois Department of Transportation research program on expressway surveillance and control. The study served as my thesis submitted in partial completion of the requirements for a master of science degree in civil engineering at Northwestern University. The paper reflects my views and I am responsible for the facts and accuracy of the data presented.

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