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Freeways, Automatic Vehicle Identification, and Effects of Geometrics

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Contents

Cost Effectiveness of Freeway Courtesy Patrols in Houston

Daniel B. Fambro, Conrad L. Dudek, and Carroll J. Messer, Texas Transportation Institute, Texas A&M University

Traffic incidents, whether due to accidents or stalled vehicles, are a major cause of congestion on urban freeways. Besides causing inconvenience to motorists, incidents also create safety hazards on the freeway. To enhance safety and to provide assistance and security to motorists, District 12 of the Texas State Department of Highways and Public Transportation has been operating a motorist courtesy patrol on some Houston freeways. A questionnaire study and a cost-effectiveness analysis were used to evaluate the operation of the patrol. The questionnaire study indicated that motorists aided by the patrol were overwhelmingly in favor of continuation of the program. The cost-effectiveness analysis showed the patrol to have a benefit-cost ratio of 2 to 1. Several additional qualitative benefits are discussed.

The need to provide assistance to stranded motorists arrived with the invention of the automobile. The expected number of motorists that need assistance, both on and off the freeway, increases as either average daily traffic increases or average trip length decreases (1). Large volumes of traffic and short trips are typical on freeways in large urban areas. Motorists who are forced to stop on the roadway because of either a stalled vehicle or an accident are one of the major causes of congestion on these freeways. Motorists involved in an incident may require one or any combination of the following needs for aid (2):

1. Service, i.e., for flat tires, mechanical and electrical problems, fuel, oil, or water, and towing;
2. Police;
3. Ambulance;
4. Fire; and
5. Information, either general information or emergency traffic routing.

An individual who is confronted with a stop because of an incident is generally unprepared to immediately cope with even the simplest of situations. Usually the only problem that the average motorist is capable of dealing with by himself is that of changing a flat tire, but some motorists may be incapable of doing even that. A few motorists might be able to take care of some of their other needs if they carried appropriate items or material for dealing with these problems. Clearly, the typical disabled motorist needs assistance.

Safety problems also arise as a result of stops on the freeway (2). These include motorists

1. Crossing operating lanes,
2. Wandering on highway shoulders,
3. Hitchhiking to seek help,
4. Leaving abandoned vehicles in or partially in operating lanes,
5. Climbing roadway protection fences, and
6. Attempting self-help (improper use of jack, touching hot engine components).

A motorist aid system does not eliminate all safety problems but should reduce their severity and frequency of occurrence. The major cause of these safety problems is the concern the motorist experiences when confronted with an unexpected breakdown in an unfamiliar environment. With the passage of time, presence of darkness, or remoteness of setting, this concern may turn to fear and cause the motorist to behave in an irrational manner. To reduce or eliminate this feeling, the motorist must have confidence that aid will come. District 12 of the Texas State Department of Highways and Public Transportation (TSDHPT) has implemented a courtesy patrol on selected freeways in the city of Houston to deal with the emergency needs of motorists and the problems that arise as a result of these needs.

COURTESY PATROL PROGRAM

Objectives

The primary objectives of the courtesy patrol program in the Houston area are to provide safety, assistance, and security for motorists using the freeways. These objectives are accomplished by

1. Assisting the stranded motorist in restoring the

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disabled vehicle to an operable condition,

2. Summoning additional aid for problems the patrol cannot correct,
3. Removing hazardous objects from the roadway,
4. Performing minor maintenance operations on roadside signs and lights,
5. Directing traffic in a safe and expedient manner in emergency situations, and
6. Operating in a prompt and dependable manner so as to instill a feeling of security in motorists.

If these tasks are carried out, benefits accrue to motorists, the TSDHPT, and the police department, and safety is improved. The courtesy patrol (a) saves motorists the expense of calling a private service, (b) reduces waiting time of stranded motorists, (c) provides a sense of security to motorists, and (d) reduces delay time to those involved in incidents as well as to those not directly involved by removing incidents and directing traffic through or around incident areas. The TSDHPT benefits through public relations and through time savings to other TSDHPT employees because the courtesy patrol performs functions that are normally done by other TSDHPT employees.

The police department benefits because of reductions in (a) police patrol time spent on nonpolice functions and (b) requests for aid that require no police function.

Safety is improved by (a) reducing accidents through early removal of debris and incidents, (b) reducing pedestrian movement on freeways, and (c) protecting stranded motorists while repairs are being made.

Figure 1. Routes of the courtesy patrol vehicles.

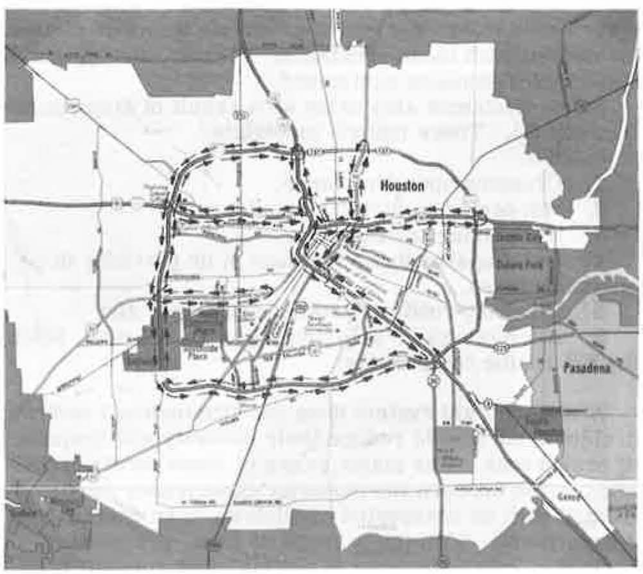

Figure 2. Courtesy patrol vehicle.

Description

Operation and Equipment

Originally, the courtesy patrol consisted of one vehicle operating on a 24-hour basis, 7 days a week. The patrol worked in three 8-hour shifts: 8 a.m. to 4 p.m., 4 p.m. to midnight, and midnight to 8 a.m. One man was on duty during each of these three shifts, and a fourth man was employed as an extra operator. In July 1972, the patrol was expanded to two pickup trucks because of the increasing demands on the services that the patrol was providing. A supervisor's pickup was used as an extra vehicle until a backup truck could be added. Currently, two men ride in each truck, thus requiring a 12-man crew to operate the patrol. The 8 a.m. to 4 p.m. shift was discontinued on weekdays on December 12, 1973, because of the energy crisis.

Emergency vehicle service is provided on 103 km (64 miles) of Houston freeways. Areas that the patrol covers include parts of loop I-610 and, inside the loop, freeways I-10, I-45, and US-59. Figure 1 shows the routes of the patrol vehicles. The 1972 ADT on each freeway was more than 91 000 vehicles/day (3). Patrol vehicles carry the following equipment:

1. One two-way radio,
2. Two flashing and one revolving amber lights per vehicle,
3. Eight flares and one case of fuses,
4. Nineteen liters (5 gal) of gasoline,
5. Nineteen liters (5 gal) of water,
6. One bumper jack,
7. One 1.1-Mg (1¼-ton) floor jack,
8. One 1.1-kg (2½-lb) and one 2.3-kg (5-lb) CO_2 fire extinguisher,
9. Two red flags,
10. One cross lug wrench,
11. One battery charger,
12. Miscellaneous mechanic's tools,
13. One shovel and one broom,
14. Six traffic cones, and
15. Absorb-all.

The vehicles are also equipped with push bumpers to move disabled vehicles from the main lanes to the shoulder. Figure 2 shows one of the patrol vehicles currently being used in Houston.

Services Provided

The patrol provides services that directly benefit motorists in need of aid, the State Department of Highways and Public Transportation, the Houston Police Department, and motorists who may not need aid themselves. Table 1 gives totals, percentages, and averages of the services rendered by the Houston courtesy patrol for different time periods during 1973. These data were taken from logbooks that were kept by the vehicle operators.

Method of Study

This paper presents an estimate of the cost effectiveness of the Houston courtesy patrol during 1973. The evaluation was done in three parts. First, responses to a questionnaire given motorists who were helped by the patrol were evaluated. The second phase of the analysis compared the benefits resulting from operation of the patrol to costs necessary to provide them. Finally, the intangible benefits resulting from operation of the courtesy patrol were addressed.

QUESTIONNAIRE STUDY

After the courtesy patrol had been in operation for a short time, a questionnaire was distributed to all motorists who were helped by the patrol during the 8-month period from March to October during 1973. The questionnaire was designed and administered by members of the Texas State Department of Highways and Public Transportation. A total of 1429 motorists filled out the questionnaire and returned it to the department. Responses to the questionnaire are necessarily biased because all respondents were assisted by the patrol. Only those questionnaires that were returned are considered in this analysis. Responses to the questionnaire are not comparable to data in Table 1 because the patrol performs many functions other than assisting disabled motorists. The five questions that appeared on the questionnaire and a brief summary of the responses to each are as follows:

1. About how long had you waited before the courtesy patrol arrived? Of the motorists who responded, 47 percent replied they had to wait less than 5 min for service, 74 percent less than 15 min, 90 percent less than 30 min, and 96 percent less than an hour.
2. What caused your problem, flat tire, out of gas, mechanical, other? The purpose of this question was to determine what type of aid the motorist needed. Replies indicated that 24 percent of the people helped had flat tires, 28 percent were out of gas, 30 percent had mechanical difficulty, and the remaining 18 percent had other problems.
3. Did this service help you? Of the motorists that returned the questionnaire, 94 percent replied that it did. The four people whom the patrol did not help commented that either they were not in need of aid or that help was already on the way. Six percent of the respondents did not answer this question.
4. This service is paid for out of the taxes you pay. Do you recommend that it be continued? The responses to this question were very similar to those of question three. Ninety-four percent of the motorists answered yes, and only three individuals answered no. Six percent of the respondents did not answer this question.
5. Comments. Because of the many different responses to this question, the replies were categorized into very favorable, favorable, unfavorable, and no comment. Very favorable comments included "excellent service," "very good service, should be continued," "the best program ever," "there should be more programs of this sort," and "this is a great service for women traveling by themselves." Some also gave detailed accounts of exactly what their problems were and how the State Department of Highways and Public Transportation in Texas should be commended for providing the service. Favorable comments were typically "thank you," "good program," and "your men were very helpful and courteous." Unfavorable comments were those that contained any negative response to the patrol. Thirty-five percent of the motorists responded very favorably, 26 percent were favorable, and 39 percent offered no comments. None of the motorists listed any unfavorable comments.

COST EFFECTIVENESS OF 24-HOUR OPERATION

All costs associated with the operation of the courtesy patrol were relatively easy to determine, but some of the benefits were rather difficult to quantify. The approach selected was to quantify those benefits that could readily be evaluated and to describe the additional nonpriceable benefits that make the service more effective. The benefits that could be priced were used in a benefit-cost analysis to determine the effectiveness of the courtesy patrol.

Costs

The cost to operate the patrol in 1973 on a 24-h basis was computed by using data supplied by District 12 of the Texas State Department of Highways and Public Transportation. The annual cost to operate the patrol was found to be \$229 400. A breakdown of these costs is given in Table 2.

Benefits

The quantifiable benefits of the Houston courtesy patrol were stratified by whether they benefited the motorist, the TSDHPT, or safety.

Motorist Related

Motorist-related benefits are those that save the motorist the expense of calling a service facility. The courtesy patrol provides several services that a motorist would normally obtain from a service facility. Data given in Table 3, from the courtesy patrol's logbooks, show the total services of this type that were performed by the patrol in 1973.

The savings in expenses to a stranded motorist were assumed to be the cost of obtaining aid from service facilities. The Houston office of the American Automobile Association (AAA) was contacted and, in turn, furnished the following cost information typically incurred by motorists for aid requests in the Houston area.

1. The maximum allowable charge for towing service by law inside Houston's city limits is \$27.50. AAA receipts examined indicated that the maximum amount was generally charged.
2. There is a standard \$5 charge by service facilities for going to the aid of a motorist on the freeway. If the service vehicle has to travel more than 1.6 km (1 mile), this price may increase to \$10, which is the charge to go from a location on the Loop to downtown Houston. For analysis purposes, the minimum \$5 charge was assumed. This \$5 charge is additional to the cost of the service provided.
3. The average price to fix a flat tire is \$2.50.
4. The average price for regular gasoline in Houston (July 1974) is \$0.133/liter (\$0.507/gal).

Based on those costs for services in Houston and the authors' experience of service rates in other cities, the following additional costs were estimated:

1. The charge to start a car or charge a battery is at least \$1.50,
2. On the average, minor repairs to vehicles are at least \$5,
3. Loaning tools or issuing water might not require an additional charge, and
4. Pushing a car from traffic requires wrecker service.

Based on these data, the savings to the motorists serviced by the courtesy patrol in Houston during 1973 were computed and tabulated (Table 3). The results indicate that operation of the courtesy patrol saved stranded motorists $40 161.

The courtesy patrol also reduces delay time of stranded motorists. The operation of the courtesy patrol enables the stranded motorist to receive aid faster than if no patrol vehicles were available. The savings in time to the motorist is a benefit of the courtesy patrol.

The average stopping times for disabled freeway motorists determined in a previous study conducted in Houston (4) are given in Table 4. Data from Table 4 were used to estimate an average stopped time per disabled vehicle of 49 min. Data given in Table 5, taken from the questionnaire evaluation, were used to estimate the average waiting time before a patrol vehicle arrived (12 min for each disabled motorist the patrol assisted). Previous studies in Houston (4, 5, 6, 7) indicate that 10 min is an acceptable estimate of the time required for aid service to be performed. Therefore, if courtesy patrol aid were obtained, the estimate of the average stopped time per disabled vehicle aided becomes 22 min. Thus, on the average, each motorist-related service the patrol performed saved a disabled motorist 27 min.

In 1973, the patrol performed 4568 motorist-related services. The total time savings to the vehicles involved is estimated as follows:

$$\text{Time savings} = (4568 \text{ services}) \times (27 \text{ min saved/service}) \times (1 \text{ h}/60 \text{ min}) = 2056 \text{ vehicle-h} \quad (1)$$

Based on a 1969 economic study of the Gulf Freeway and the conservative estimate of 1.0 person/passenger vehicle, the cost per person-hour of travel based on 1967 data was determined to be $2.92 (8). Assuming a conservative compound increase of 5 percent per year for 6 years and a more realistic value of 1.2 persons/passenger vehicle, the value of one vehicle-hour in 1973 would be $4.69. By using this amount, the value of time savings to the disabled motorists helped by the patrol becomes

$$\text{Time savings} = (2056 \text{ vehicle-h}) \times (\$4.69/\text{vehicle-h}) = \$9643 \quad (2)$$

Through early removal of incidents from traffic lanes during the peak periods, the courtesy patrol reduces motorist delay time at certain incidents. Incidents, whether stalled vehicles or accidents, are a major cause of congestion on urban freeways. Incidents reduce the capacity of the roadway, and if the reduction in capacity reaches a point where the demand on the facility is greater than the available capacity, motorists experience considerable delay. During peak periods, Houston freeways operate at or near capacity; therefore, any incident that occurs then causes motorists to experience greater travel times. A previous study on the Gulf Freeway (7) indicated that an incident that blocked one lane of a three-lane freeway for 15 min during the peak period caused 690 vehicle-hours of delay for motorists. During 1973, the patrol pushed 119 vehicles from the traffic stream during the peak period. Using 690 vehicle-hours as a reasonable estimate of the savings per incident and using the value of time previously shown as $4.69 per vehicle-hour yield the following estimate of monetary benefit resulting from the courtesy patrols assisting stranded motorists off the freeway main lanes during peak periods:

$$\text{Savings} = (119 \text{ services/year}) \times (690 \text{ vehicle-h/service}) \times (\$4.69/\text{vehicle-h}) = \$385\,096/\text{year} \quad (3)$$

The courtesy patrol benefits the Texas State Department of Highways and Public Transportation by saving other employees' time. In the absence of the courtesy patrol, the TSDHPT would have to use other personnel to perform some of the services the patrol currently provides. Before the patrol began operation, the maintenance sections in District 12 had to deal with requests for aid or repair work made at night. Since the operation of the patrol was initiated, each of the four maintenance sections in the city feels that they save an average of $400 per month in time alone. Thus, the annual savings can be conservatively estimated as

$$\text{Savings} = (\$400/\text{month/section}) \times (4 \text{ sections}) \times (12 \text{ months/year}) = \$19\,200/\text{year} \quad (4)$$

The safety-related benefits of the courtesy patrol include a reduction in the number of accidents due to early removal of debris and incidents. The services provided by the courtesy patrol make Houston freeways a safer place to drive. Kuprijanow (1) estimated that 10 stops/km (16 stops/mile) per day could be expected on a freeway with an ADT of 75 000 and an average trip length of 16 km (10 miles). Of these, 42 percent of the total are emergency stops (1), which require services of the highway patrol, private operators of tow services, ambulance services, or local fire departments. Because the ADT on the Houston freeways serviced by the patrol was between 90 000 and 160 000, 10 stops/km (16 stops/mile) is considered to be a conservative estimate of the number of stops per day in the patrol area. Because of the lack of data, 16 km (10 miles) was assumed to be a conservative estimate of the average trip length on the freeways serviced by the patrol. Based on these assumptions, the number of emergency stops that would be expected in the 103-km (64-mile) section of Houston freeways covered by the courtesy patrol is

$$\text{Number of emergency stops} = (10 \text{ stops/km/day}) \times 0.42 \times (103 \text{ km patrolled}) \times (365 \text{ days/year}) = 157\,899 \text{ emergency stops/year} \quad (5)$$

Goolsby (4) observed 27 000 emergency stops in a 17.7-km (11-mile) section of freeway in Houston during 1 year. This stoppage rate would result in 157 120 emergency stops per year in a 103-km (64-mile) section. Because of the favorable comparison of the results of the two references, 157 000 is considered a good estimate of emergency stops in the patrol section during 1973.

Each emergency stop has the possibility of causing a secondary accident, i.e., an accident involving a stopped, parked, or disabled vehicle. Data supplied by the Gulf Freeway Surveillance and Control Center indicated that, during 1973, there were 144 accidents of this type in the patrol section. Data taken from logbooks show that the courtesy patrol assisted more than 8000 disabled motorists during this same time period. Because of the safety aspect of courtesy patrol service (flashing lights, quicker service, experienced operators), no secondary accidents were reported when the patrol assisted disabled motorists. In contrast, a statistical analysis conducted by the authors showed that in a random sample of 8000 unaided emergency stops some secondary accidents would have been expected to occur. Because all secondary accidents in the patrol section occurred when courtesy patrol aid was not provided, the estimated number of secondary accidents per unserviced emergency stop can be computed as follows:

Table 1. Services rendered by the courtesy patrol during 1973.

Service Rendered	1973 Total	Percentage of Total	Average Daily	Average Weekday	Average Weekend	Total Midnight to 7 a.m.	Total 7 to 9 a.m.	Total 9 a.m. to 4 p.m.	Total 4 to 6 p.m.	Total 6 p.m. to midnight	Peak Period Total	Peak Period Percent
Removed debris or hazard	3 261	26.2	8.9	9.2	8.2	1321	164	979	255	542	237	7.2
Issued gas	1 217	9.7	3.3	3.4	3.2	301	72	356	155	333	178	14.1
Controlled traffic	1 119	9.0	3.1	3.1	3.0	271	62	345	133	308	149	13.3
Pushed from traffic	572	4.6	1.3	1.2	1.4	100	46	185	96	145	119	20.8
Changed tire	546	4.4	1.5	1.1	2.5	130	35	196	50	135	57	10.4
Lent tools	696	5.6	1.9	1.8	2.1	174	31	210	81	200	83	11.9
Issued water	405	3.3	1.1	1.1	1.1	64	24	136	71	110	67	16.5
Took to phone	201	1.6	0.6	0.6	0.5	32	17	73	33	46	45	22.3
Took to service station	357	2.9	1.0	1.0	0.9	82	23	129	38	85	44	12.3
Charged battery	205	1.7	0.6	0.6	0.5	46	0	84	7	68	6	2.9
Made call for motorist	365	3.0	1.0	0.9	1.3	80	15	114	45	111	47	12.8
Made minor repair to vehicle	220	1.8	0.6	0.6	0.7	30	13	94	25	58	37	16.8
Started vehicle	707	5.7	1.9	2.0	1.8	131	67	221	98	190	134	18.9
Reported stall	196	1.6	0.5	0.5	0.7	126	1	26	12	31	8	4.0
Reported accident to police	452	3.6	1.2	1.2	1.3	91	29	121	86	125	92	20.3
Reported debris	81	0.7	0.2	0.2	0.3	23	2	31	5	20	5	6.1
Reported abandoned vehicle	222	1.8	0.6	0.6	0.6	76	16	60	25	45	35	15.7
Called wrecker	258	2.1	0.7	0.7	0.8	46	12	67	53	80	46	17.8
Reported damage to facilities	560	4.5	1.5	1.3	2.1	243	16	126	24	151	17	3.0
Repaired facilities	370	3.0	1.0	0.9	1.2	202	13	79	17	59	19	5.1
Gave directions	140	1.1	0.4	0.3	0.5	42	3	49	14	32	7	5.0
Put fire out	21	0.1	0.1	0.1	0.1	4	1	5	7	4	5	23.8
Other	257	2.1	0.7	0.7	0.8	48	25	92	36	56	41	15.9
Totals	12 428	100	33.7	33.1	35.6	3663	687	3778	1366	2934	1468	

Table 2. Cost to operate courtesy patrol in 1973.

Item	Monthly Cost ($)	Annual Cost ($)
Administration	1 500	18 000
Labor (12 man-years)	14 500	174 000
Vehicle operating expense and depreciation	2 400	28 800
Materials and supplies	720	8 600
Total	19 120	229 400

Table 3. Motorist savings gained by not having to request aid from a private business in 1973.

Service	No. of Services	Cost per Service ($)	Annual Savings ($)
Issued gas	1217	0.507 + 5	6 702
Pushed from traffic	572	27.50	15 730
Changed tire	546	2.50 + 5	4 095
Lent tools	696	5	3 480
Issued water	405	5	2 025
Charged battery	205	1.50 + 5	1 333
Made minor repair to vehicle	220	5.00 + 5	2 200
Started vehicle	707	1.50 + 5	4 596
Total	4568		40 161

Table 4. Average stopped times before the courtesy patrol began operation.

Reason for Stop	No. of Stops	Average Stopped Time (min)	Total Stopped Time (min)
Gas	131	30.9	4 047.9
Tire	207	41.4	8 569.8
Mechanical	299	82.3	24 607.7
Accident	50	72.6	3 630.0
Other	194	14.6	2 832.4
Total	881		43 687.8

Table 5. Average waiting time for courtesy patrol aid based on estimates by stranded motorists the patrol assisted during 1973.

Waiting Time (min)	Number (N)	Midpoint (X_i)	$X_i \times N$ (min)
<5	600	2.5	1 500
5 to 15	347	10.0	3 470
15 to 30	214	22.5	4 815
30 to 60	71	45.0	3 195
>60	50	60.0	3 000
Total	1282		15 980

Table 6. Benefits of the courtesy patrol in 1973.

Benefit	Estimated Annual Saving ($)
Saves motorist expense of calling private service	40 161
Reduces delay time to motorists in need of aid	9 643
Reduces delay time to motorists on freeway	385 096
Saving to Texas State Department of Highways and Public Transportation	19 200
Reduction in accidents	5 152
Total	459 252

Rate = (144 secondary accidents in the patrol section)
$\div$ (157 000 emergency stops - 8000 serviced emergency stops)
= 0.000 97 secondary accident/unserviced emergency stop (6)

If we assume that the accident rate given in equation 6 is a reasonable estimate, then the number of secondary accidents that would have occurred if the courtesy patrol had not provided aid for the 8000 disabled motorists it helped during 1973 can be calculated as follows:

Secondary accidents = (0.000 97 secondary accident/unserviced emergency stop) x (8000 serviced stops) = 8 secondary accidents that did not occur (7)

Burke (9) determined accident costs for three types of accidents. It was assumed that in the eight secondary accidents only two cars would have been involved and that only property damage would have occurred. Using Burke's figure of \$307/vehicle for a property damage accident in 1972 and assuming a 5 percent inflation rate per year yield the following cost for eight secondary accidents in 1973:

(\$322/vehicle) x (16 vehicles) = \$5152 (8)

Annual savings due to the reduction of eight secondary accidents is \$5152.

Services of the courtesy patrol also reduce the number of pedestrian accidents. Because the courtesy patrol provided aid to more than 8000 disabled motorists, the number of would-be pedestrians walking for aid services in the patrol area decreased. A California study (10) concluded that 43 percent of all the pedestrians struck on freeways were on the facility because their vehicle was either disabled or involved in a prior accident. Data from the Gulf Freeway Surveillance and Control Center showed that there were 34 pedestrian accidents in the patrol section during 1973. No pedestrian accidents were reported when courtesy patrol service was provided for disabled motorists; therefore, all pedestrian accidents that occurred were assumed to be the result of unserviced stops. The estimated pedestrian accident rate is calculated as follows:

Rate = [(34 total pedestrian accidents) x 43 percent] ÷ (157 000 emergency stops - 8000 serviced stops) = 0.0001 pedestrian accient/unserviced emergency stop (9)

Based on the estimated accident rate, the number of pedestrian accidents that would have occurred in the patrol section if the patrol had not serviced the 8000 disabled motorists would have been less than one. Although the number of would-be pedestrians decreases, no reduction in number of pedestrian accidents occurs because of the low pedestrian accident rate. An estimated reduction in the number of accidents would occur if the patrol were able to assist more stranded motorists.

Comparison of Benefits and Costs

The cost to operate the patrol in 1973 was \$229 400. Monetary benefits of the patrol are given in Table 6 as \$459 252. The resulting benefit-cost ratio is

b/c = \$459 252/\$229 400 = 2 (10)

This means that, for every dollar spent to provide courtesy patrol service on the Houston freeways during 1973, an estimated \$2 worth of benefits were gained by motorists or the Texas State Department of Highways and Public Transportation.

Additional Benefits

In addition to the quantifiable benefits that have already been discussed, the following nonpriceable benefits add to the effectiveness of the patrol.

Benefits to Motorists

The courtesy patrol provides a sense of security to motorists. Prompt, dependable service by the courtesy patrol creates a sense of security for stranded motorists. Knowing the patrol is on duty, motorists feel safer when their vehicles become disabled. This feeling of safety is intensified when trouble occurs late at night or when the vehicle operator is alone and in an unfamiliar area. Assigning a monetary value to this feeling would at best be arbitrary and is not considered in this report as such; however, it is recognized as a benefit that the patrol provides.

Benefits to Texas State Department of Highways and Public Transportation

Operation of the courtesy patrol improves public relations. The questionnaire survey indicated that nearly all of the people that the patrol helped thought that it was a worthwhile service and that it should be continued. No one interviewed made any negative comments about the operation of the patrol. This indicates that the courtesy patrol has helped to establish a favorable public image. American Oil Company operates a similar patrol in San Diego (6, 7) and feels that increased patronage of its dealers' fuel and service facilities as a result of the image they are creating justifies operation of the patrol.

Benefits to Houston Police Department

Operation of the courtesy patrol reduces the time spent by police patrols on nonpolice functions. Operation of the courtesy patrol reduces the demand for aid on the police patrols on the freeways. Ideally, the police department should be able to decrease the number of patrols because of decreased demand for their services; however, in Houston this has not been the case. Houston's freeways were patrolled by two motorcycle policemen, both before and after initiation of the courtesy patrol, but before the courtesy patrol began operation there were just that many more needs that the police patrols could not take care of. There are probably enough police-related needs on the freeways to keep two patrolmen busy. Because the number of police patrols was not reduced, no monetary savings can be attributed to the operation of the courtesy patrol; however, the courtesy patrol allowed the police patrols to spend more of their time on police-related work.

Another benefit to the police department is that the courtesy patrol reduces requests for aid that require no police function. A previous study indicated that between 55 and 85 percent of the requests for aid require no police function (2). It would be expected that the police department would be able to devote that much more time to problems for which it is needed. But, as the courtesy patrol is able to deal with so few of the total needs for assistance, a reduction in requests for police aid would be very difficult to determine.

SUMMARY

District 12 of the Texas State Department of Highways and Public Transportation provides courtesy patrol service for motorists on some of the freeways in Houston. The objectives of this patrol are to provide safety, assistance, and security for motorists using the freeways. To accomplish these objectives, the patrol provides services that directly benefit motorists in need of aid, motorists who are indirectly affected by incidents, the Houston Police Department, and other members of the Texas State Department of Highways and Public Transportation.

In 1973 the courtesy patrol in Houston performed 12 428 services. Of these, 4568 services benefited motorists in need of aid, 2017 services benefited motorists using the freeway who were indirectly affected by incidents, 1571 services benefited the Houston Police Department, and 4272 services benefited other members of the State Department of Highways and Public Transportation. A total of 1429 questionnaires were returned by disabled motorists who the patrol assisted during March through October 1973. The questionnaires were evaluated, and the results indicate that motorists who were aided by the patrol overwhelmingly favored continuation of the program.

This report estimates the cost effectiveness of the courtesy patrol in Houston. To do this required computing the costs to operate the patrol in 1973 based on data supplied by District 12 of the Texas State Department of Highways and Public Transportation. Estimated monetary benefits to motorists using Houston's freeways in 1973 were $40 161 saved because the motorists did not have to request aid from a private service facility, $9643 saved by the stranded motorists because of reduced waiting time for aid to arrive, and $385 096 worth of time saved by other motorists due to early removal of incidents from traffic lanes during the peak periods. The Texas State Department of Highways and Public Transportation was able to save $19 200 because maintenance personnel did not have to respond to aid calls at night. A $5152 savings was attributed to the patrol as a result of a decreased number of secondary accidents. By comparing these estimated benefits to the cost necessary to provide them, a benefit-cost ratio of 2 to 1 was computed. In addition, the provision of a feeling of security to motorists and the creation of a favorable public image were considered intangible benefits of the patrol.

ACKNOWLEDGMENTS

This report discussed one phase of a research project on development of urban traffic management and control systems conducted by the Texas Transportation Institute and the Texas State Department of Highways and Public Transportation in cooperation with the Federal Highway Administration.

The contents of this report reflect the view of the authors who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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Urban Freeway Corridor Control Model

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In recent years, on-ramp control and freeway redesign to improve freeway traffic operations have received considerable attention. Concurrently, work has been under way to develop coordinated control and design systems to improve street operations. However, only a few studies have used an integrated approach to control and design both a freeway and an adjacent street network (i.e., an urban freeway cooridor). Inasmuch as corridor control and design systems have already been implemented in some locations and are planned in others, it would be most appropriate to have a methodology available that simultaneously considered all pertinent corridor control and design variables, such as ramp-metering rates, traffic signal timings, quantity of traffic to be diverted, and corridor geometry. The objectives of this study are (a) to develop such methodology and (b) to apply the methodology to corridor control optimization.

A fixed-time methodology is developed to provide insight into the interactions among various corridor de-

Figure 1. Freeway corridor research overview.

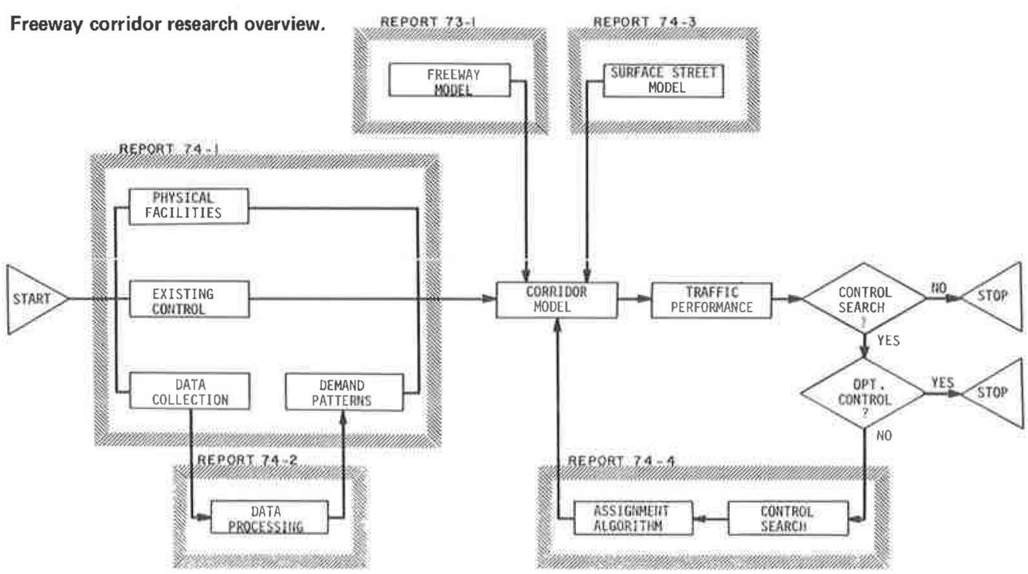

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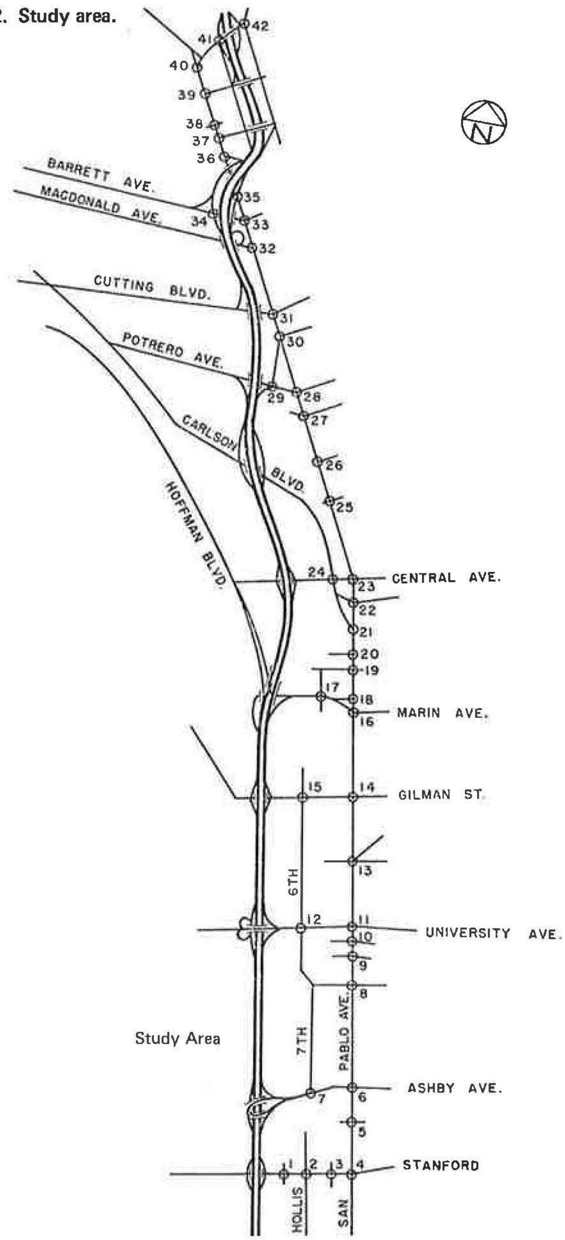
Figure 2. Study area.



sign, demand, and control variables. This methodology, after computerization, is used to evaluate possible corridor control strategies and to select the optimum. The resulting computer model, corridor model with queuing analysis, version 1, including control CORQ1C, combines two existing simulation models with a decision model. The decision model, based on a linear programming technique, selects an optimum corridor control strategy and predicts the resulting traffic diversion (Figure 1).

This methodology is then applied to the determination of optimum fixed-time corridor control strategies by using data from a section of the northbound Eastshore Freeway corridor (I-80) in the San Francisco Bay area (Figure 2).

Such research is expected to provide optimum corridor control strategies and to simulate improved use of existing facilities by balancing traffic against available capacity throughout a corridor system.

Single Point Diversion of Freeway Traffic

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This paper discusses the development and application of diversion control policies for the routing of freeway traffic at a single point. Specifically, this class of policies can be used to allocate traffic flows over a two-roadway system during periods when traffic demands exceed some roadway capacities. Evaluation of these policies reveals a consistent reduction in roadway congestion, a reduction in motorist delay, and an improvement in overall system operation. The principal tool used for system evaluation was the Sperry traffic analysis and research simulation program. This macroscopic freeway simulation is based on a hydrodynamic traffic flow model. The resulting optimized policies have been incorporated into the design for a practical real-time alternate routing system by Sperry Systems Management.

During peak periods, intercity freeway systems experience oversaturation due to increased traffic demands. These demands cause recurrent congestion that is intensified when accidents or other unpredictable or temporary disturbances occur. This problem can be solved by constructing additional facilities, a prospect that may not be possible. The program documentation for the federally coordinated diversion and control project identifies an alternative solution: to distribute traffic over alternative highway facilities, which may offer advantages to drivers when the primary facility is saturated, although the routes may be less direct. The advantages to individual drivers can be increased if the choice of alternate routes can be related to their destinations. The situations can range from the simplest case of a single alternate route to that of multiple routes and choice points within and in advance of a traffic corridor between urban areas.

The Office of Research of the Federal Highway Administration initiated a research program aimed at developing a system to distribute traffic over existing facilities by supplying real-time routing information to intercity and local motorists. This would enable motorists to avoid (a) congestion on primary routes between and around cities and (b) the mixture of through traffic and local peak-hour traffic. With the cooperation of the Maryland State Highway Administration, Interstate 95, the Baltimore Harbor Tunnel Thruway (HTT), and the Baltimore Beltway (I-695) were selected as the location for the design, installation, and evaluation of a system for diverting traffic at a single point onto a single alternative (Figure 1).

Sperry Systems Management was selected as the FHWA contractor for this research problem. The contract included the task of designing a real-time system for optimal distribution of southbound traffic on the highway system shown in Figure 1. Under this task was the requirement for developing and testing candidate diversion strategies and algorithms for routing traffic at a single point.

This paper addresses the class of diversion control policies that have application for the management of traffic flows at a single point of a roadway system during those periods when the traffic demands downstream exceed the roadway capacities. The freeway configuration and the control concept are discussed, and the control algorithms are evaluated by using the Sperry traffic analysis and research (STAR) simulation program. The impact of design considerations is discussed in relation to the strategy implemented.

TRAFFIC APPLICATION

Every year more freeway systems are experiencing peak-period traffic demands that cause recurrent congestion and oversaturated flow conditions at multiple locations throughout a roadway network. In addition to the recurrent congestion at these bottleneck locations, nonrecurrent congestion can occur at any network location as a result of a major incident or other randomly occurring, capacity-reducing event. In many instances, because of the specific location of the traffic congestion and the configuration of the roadway, it is possible at a single interchange to divert a fraction of the traffic from the roadway experiencing congestion to another roadway operating with excess capacity, which improves overall system operation.

As shown in Figure 1, HTT is the shortest and quickest route, in the absence of congestion, for intercity

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traffic originating north of Baltimore and destined for Washington, D.C., and points south. However, because of its limited capacity (1450 vehicles/lane/h), the combined demands of intercity and local traffic frequently produce extensive roadway congestion. During these daily peak periods, weekends, and holidays, I-695 provides a viable alternative for the diversion of the intercity motorist. By measuring the precongestion flows and determining the regions of congestion and incidents, control strategies can be used to divert a fraction of the intercity traffic to the alternate route.

Conversely, congestion at the tunnel notwithstanding, traffic conditions on I-695 may be such that the HTT remains the preferred route. Consequently, if some measure can be derived to permit a real-time evaluation of the more advantageous route, traffic can be rerouted such that the intercity motorist benefits and congestion on the more critical route is relieved. The essential consideration in the design of diversion strategies, therefore, is the choice of criteria with which to perform the route diversion.

TRAFFIC DYNAMICS AND SIMULATION

Hydrodynamic Flow Model

The movement of traffic over the network (i.e., the traffic dynamics) must be described in terms of a quantitative model. In general, traffic research has established that macroscopic flow models are suitable for freeway modeling when several kilometers of roadway are of interest for several hours at a time. The hydrodynamic flow model as developed by Lighthill and Whitham (1) is the most widely recognized of this class. The stream variables that characterize the flow of freeway traffic are volume q, speed u, and concentration or density k. Under uniform flow conditions (i.e., all vehicles traveling at the same speed and uniform spacing), the equation relating these three quantities is

$$q = uk \qquad (1)$$

The function defining the fundamental nature of vehicular traffic flow (sometimes called the traffic equation of state) is shown in Figure 2 as an idealized relation between the volume and the concentration of traffic at a specific roadway location. From equation 1 the corresponding speed on the roadway is defined. In Figure 2, the two distinct regions of traffic flow are an uncongested, stable, free-flow region on the roadway for a concentration ranging from 0 to the critical concentration (kC) and a congested, unstable flow region for a concentration ranging from kC to the jam concentration (kJ). The boundary separating these two regions defines the roadway operating point (P0) at which maximum flow (the roadway capacity) is achieved.

Based on accepted traffic operational practices, a desirable operating state of roadway section is a steady-state operating point (P1) on the q-k curve slightly to the left of the maximum capacity point. This steady-state operating point for a roadway results in a margin of safety so that minor flow disturbances will not force the roadway operation into the congested flow region (P2).

On a typical freeway there exist sections of roadway operating at distinct (q, k, u) operating points within homogeneous zones and separated by zone boundaries or wave fronts. The wave fronts in general move over the roadway, propagating the changes in traffic flow. Lighthill and Whitham (1) determined that the speed at which the wave fronts move with respect to the roadway is

$$S_w = \Delta q/\Delta k = (q_1 - q_2)/(k_1 - k_2) \qquad (2)$$

where S_w = wave front speed in kilometers per hour.

When the sign of the volume difference across the wave front Δq is the opposite of the sign of the density difference Δk, the wave front moves upstream with respect to the roadway. These backward moving wave fronts result either when demand exceeds capacity, in which case congestion builds, or after a bottleneck has been removed, in which case congestion dissipates. In all other cases, the wave fronts move downstream with respect to the roadway. The moving wave front is an indicator of the changing traffic conditions along the roadway.

STAR Simulation

The use of traffic simulation as the principal design tool for developing the single point diversion control policies was dictated by the complex geometric and dynamic nature of the I-95/HTT/I-695 freeway system. In addition, a tool was needed to provide the level of confidence required by traffic operations personnel before system implementation would be attempted. Therefore, Sperry developed an event-scanning traffic simulation based on the hydrodynamic flow model with the capability of simulating multiroadway freeway systems (2). Its use provided the ability to economically study the system's responses to a variety of input specifications and traffic scenarios.

In the hydrodynamic flow model, traffic is propagated along each major roadway in homogeneous zones separated by wave fronts. Permanent stationary wave fronts are established at entrance and exit ramps, at detector stations, and at roadway locations where the traffic equation of state changes because of variations in roadway geometry, e.g., at lane drops, grades, and tunnels. Stationary wave fronts are also introduced and removed at roadway locations as required to simulate the occurrence and removal of capacity-reducing incidents and bottlenecks. Moving wave fronts with speed S_w (equation 2) are introduced at each entrance ramp each time the entrance volume changes, at incident sites, at roadway bottlenecks when demand exceeds capacity, and at the diversion point interchange when traffic is shifted between roadways.

Because the effect of the single point control algorithm is to divert the intercity motorist, the STAR simulation and the hydrodynamic flow model were provided with the capability of differentiating between intercity and local traffic. This differentiation was implemented by defining a second type of moving wave front called a vehicle boundary, which defines the boundary between sections of flow that have different ratios of intercity volume to total traffic volume. All other traffic parameters remain constant across the boundary. The speed of this vehicle boundary with respect to the roadway is the mean speed u of the vehicles in both sections.

The vehicle boundaries are generated only at the diversion interchange of I-95 and I-695, i.e., the interchange where the intercity traffic enters the roadway. All other entrances generate only local traffic. The sum of local and intercity traffic gives the total traffic at any point along the roadways. By using these vehicle boundaries, floating car travel times from the I-95/I-695 diversion point to any point along either roadway can be provided. In particular, we are interested in the southbound travel times along both the I-695 and I-95/HTT routes to the Baltimore-Washington Parkway and to the interchange of I-95 south of Baltimore.

As the wave fronts and the vehicle boundaries move over the roadways, intersections or collisions between them occur. These intersections are the most important of the five types of state events that occur at various times during the simulation and that represent a change in the traffic state of the simulation model.

The types of state events are

1. Intersections between wave fronts, vehicle boundaries, or both,
2. Queues at entrance ramps,
3. Volume changes at entrance ramps,
4. Incident occurrence or clearance, and
5. Changes in diversion control.

In addition, there is another class of events related to the system update of surveillance data. These surveillance events occur periodically to observe and record the roadway state without changing it. The occurrence of any of these events steps the simulation in time. A detailed description of each of these classes of events is presented elsewhere (2).

The logic that ties together the several segments of the program is the simulation clock. The clock controls the cataloging and processing of the events, including surveillance events, as they occur during a run. Present time is used as the datum, and the events are sorted and cataloged in sequence based on the time interval to event occurrence. This set of event times is then searched for the minimum time to the next event. The simulation is stepped ahead by this minimum event time interval and the event dynamics are processed. Upon completion of the processing, the clock is reentered and the steps repeated. The use of an event-scanning clock with a variable time step results in an extremely efficient simulation with respect to computer running time.

CONTROL CONCEPT

By applying the traffic dynamics to the I-95/HTT/I-695 roadway system, several traffic scenarios can be postulated for which the application of diversion control concepts would be appropriate and result in improved system operation. The scenarios have the following stages: The roadways are operating under peak-period conditions for which demand volume flows are approaching roadway capacities. All roadway sections are operating under free-flow conditions. During this peak period an incident occurs on the HTT that restricts the flow of traffic. The severity of the incident can be quantified by measuring the residual flow of traffic moving past the incident location. This flow can be termed the bottleneck capacity of the incident.

Upstream of the incident a region of congested flow (density > kC) propagates at a speed S_w determined by equation 2.

The capacity-reducing incident remains on the roadway for a period of time (generally, 15 to 30 min), after which it is removed. The traffic congestion built up behind the incident is released and moves down the roadway at the roadway capacity (P0 in Figure 2), and the roadway operation returns to preincident conditions.

The application of diversion control modifies the scenario at step 3 by shifting a portion of the traffic demand volume upstream of the incident to I-695. At a later time, after the incident has ended, the traffic diversion is terminated and the traffic demand is shifted back to the I-95/HTT roadway.

The essential consideration in the design of diversion strategies is the choice of a performance criterion with which to perform the real-time evaluation of each roadway so that the intercity motorist can be routed. Three criteria incorporated into candidate algorithms for evaluation were

1. Delay difference, control based on difference in travel delay experienced by the intercity motorist on each roadway;
2. Delay rate difference, control based on an equalization of the rate of increase of motorist travel delay on each roadway; and
3. Total roadway delay difference, control based on the difference in system delay of all motorists on each roadway.

Figure 3 shows a basic mechanization of the diversion control algorithm. The algorithm uses a switching plane to compare the relative performance of each roadway as defined by each of the above criteria. The comparison is made by defining the appropriate switching boundaries. In principle, the boundaries divide the plane into two regions, each of which corresponds to a control action. When the system is operating in region 1, the control action is to direct the intercity motorist to the I-95/HTT roadway. For the system operating in region 2, the control action is to divert the intercity motorist to the I-695 roadway. The control algorithms are therefore on-off types of controllers (3). The principle control action available is the time at which the diversion is turned on and the time at which the diversion is turned off.

A mechanization of each of the above candidate algorithms requires, in addition to the switching plane, the specification of a surveillance subsystem to measure the fundamental traffic parameters (volume, speed, concentration) at a suitable set of roadway locations (detector stations) and to calculate the performance parameters for each roadway. Table 1 gives a set of equations that can be used to calculate each of the performance parameters.

Figure 4 shows a block diagram of the I-95/HTT/I-695 single point diversion system with control and the overall signal flow paths and interfaces between the roadway system and the control algorithm. The linearization of the roadway system as implied by the block diagram, although a simplification of actual system operation, clearly shows the essential system dynamic relationships that determine the major algorithm characteristics. These relationships define vehicle delay TD as the integral of delay rate $\dot{T}D$ and total roadway delay TDI as the integral of vehicle delay TD.

The dynamics for each roadway are represented by the forward paths as shown within each dashed box. The traffic flows on each roadway (q_i, $i = 1, 2$) are approaching the diversion point A and are modified by the divertible traffic at the control points B. Under the assumption that the resulting total traffic demand, whether normal or incident, is greater than the roadway capacity (q_{MAXi}), the delay rate TD_i is calculated C. The other parameters are obtained by integration. The feedback path incorporates the diversion algorithm, which shifts the divertible traffic q′ between the roadways D. The control algorithms generated from each of the specified criteria represent rate feedback, position feedback, and position integral controllers respectively.

During algorithm evaluation, the control characteristics represented by these controller types (4) were found to play a significant role in final algorithm selection. As will be discussed, the best diversion strategy was found to be the strategy that incorporated the delay difference criterion.

Whereas the basic algorithm is a two-level, on-off controller, an additional level of control authority is possible depending on the capability of the roadside equipment used to communicate with the motorist. Research in motorist communication techniques (5) has shown that motorist compliance (number of motorists who will divert) with displayed sign messages varies. At the present time, it is questionable whether this level of control can be used in a consistent positive manner. The algorithms discussed here do not address this level of control; i.e., they do not use the ability to effect variation in motorist diversion compliance through the use of variation in sign messages.

In general, the level of control authority available for diversion can be modeled as

$$qd = \alpha q' \tag{3}$$

where

q' = volume of intercity traffic approaching the diversion point,
α = fraction of intercity traffic that will divert in response to the message displayed, and
qd = volume of intercity traffic diverted (i.e., the level of control).

Values for q' and α were selected for testing based on the volume and origin-destination data collected at the study site. Several values of these parameters were tested in the simulation studies. The sensitivity of α is discussed later.

EVALUATION AND OPTIMIZATION OF THE CONTROL

To evaluate and optimize the control algorithm require that a set of performance criteria or measures of effectiveness (MOEs) be specified. These MOEs must quantify the following operational objectives:

1. The total roadway system must show improved operational characteristics when the diversion control system is activated;
2. Intercity motorists must benefit from system operation; and
3. All motorists, both intercity and local, must benefit from system operation.

The specification of the second objective is required because it is the intercity motorist who provides the control authority for the operation of the system. If a typical motorist in that group does not perceive a benefit from the operation of the diversion system, then a negative reinforcement will result such that he or she will tend not to believe the information provided. In equation 3, this condition will manifest itself by the diversion fraction α approaching zero and hence the divertible traffic volume also nearing zero. This observation represents one example of a unique characteristic of traffic systems and their control: the independence and, in a sense, the uncontrollability of the driver.

Two MOEs that collectively have been used to quantify the above objectives are mean roadway speed in kilometers per hour

$$MS = \left[\sum_{j=1}^{N} \int_{t_1}^{t_2} L_j \Delta X_j k_j(t) u_j(t) dt \right] \Bigg/ \left[\sum_{j=1}^{N} \int_{t_1}^{t_2} L_j \Delta X_j k_j(t) dt \right] \tag{4}$$

and total system delay in vehicle-hours

$$TSD = \sum_{j=1}^{N} \int_{t_1}^{t_2} L_j \Delta X_j k_j(t) [1 - u_j(t)/uFF_j] \, dt \tag{5}$$

where

L_j = number of lanes of roadway section,
ΔX_j = length of roadway section,
$k_j(t)$ = concentration over roadway section during time period t,
uFF_j = free-flow speed of roadway section,
$u_j(t)$ = speed over roadway section during time period t,
N = number of roadway sections for each roadway, and
t_1, t_2 = upper and lower limits defining the time period of observation (t).

Equations for these MOEs can also be written in discrete time for individual sections and for all roadways over an entire system.

A second equation for the delay MOE is appropriate when major diversion of the traffic flow occurs within the system:

$$TSDD = \sum_{j=1}^{N} \int_{t_1}^{t_2} L_j \Delta X_j k(t)_j [1 - u(t)_j/uFF_j] + [(1 - TTFF1/TTFF2) qd_j/q_j] \, dt \tag{6}$$

where

TTFF = free-flow travel time over the routes included in the diversion,
1 = route from which vehicles are diverted (I-95/HTT), and
2 = route to which vehicles are diverted (I-695).

This equation represents the total system delay plus the delay to the diverted vehicles based on the change in free-flow travel time of the route to which diversion is made. This latter term is $(1 - TTFF1/TTFF2)qd_j/q_j$, which takes into account the cost based on the change. The volume qd_j is defined in equation 3. This form of equation 6 is a quantification of objective 3, i.e., that all motorists must benefit from system operation.

To quantify objective 2 requires that the total system delay MOE be modified by including factors to apportion that fraction of delay incurred by the intercity motorist. Specifically, the factor (q_j'/q_j) is inserted in the integral for equations 5 and 6 thus modifying all terms. In addition, the factor (qd_j/q_j) in equation 6 is replaced by (qd_j/q_j').

SIMULATION RESULTS

Extensive testing and evaluation of the control algorithm were performed by using STAR, a dynamic traffic simulation of the I-95/HTT/I-695 roadway system, based on the hydrodynamic flow model. A limited validation of the simulation was performed before the algorithm was tested and evaluated (6). The evaluation was based on the following set of simulation runs:

1. A baseline run that established roadway operations under no incident and no control conditions,
2. An incident run that established roadway operations under uncontrolled incident conditions, and
3. A control run that established for each algorithm to be tested roadway operations under controlled incident conditions.

Sets of these runs were made for various combinations of the following parameters: roadway demand volume, incident location, incident severity, volume of intercity traffic, and diversion fraction. A typical set of these simulation runs for each control algorithm is shown in Figures 5 and 6. The set of input parameters

Figure 1. Study network.

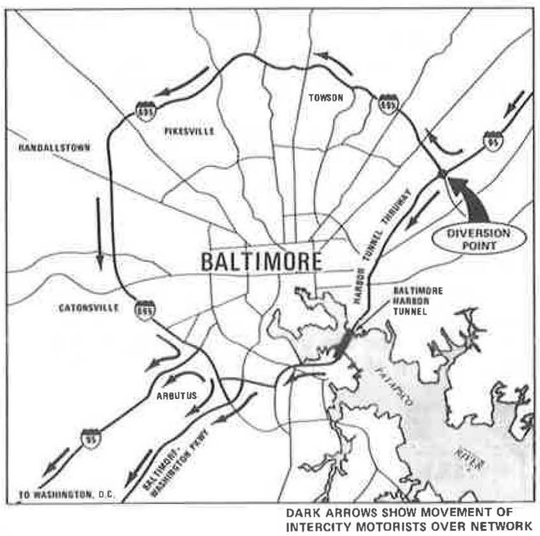


Figure 2. Flow-concentration relation.

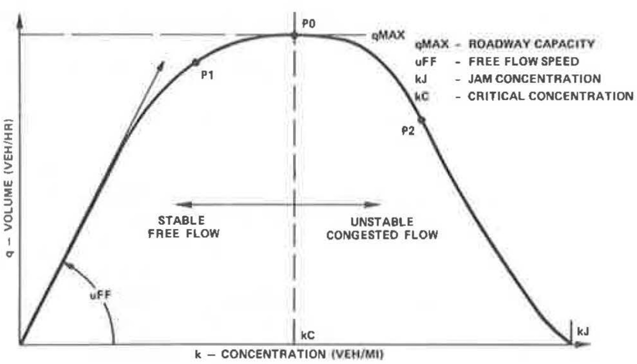


Figure 3. Diversion control algorithm switching plane.

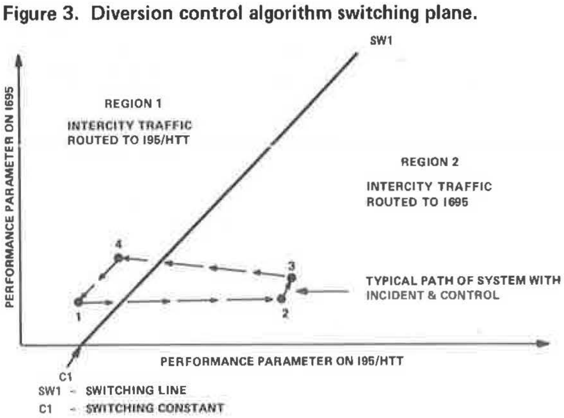


Figure 4. Single point diversion with control.

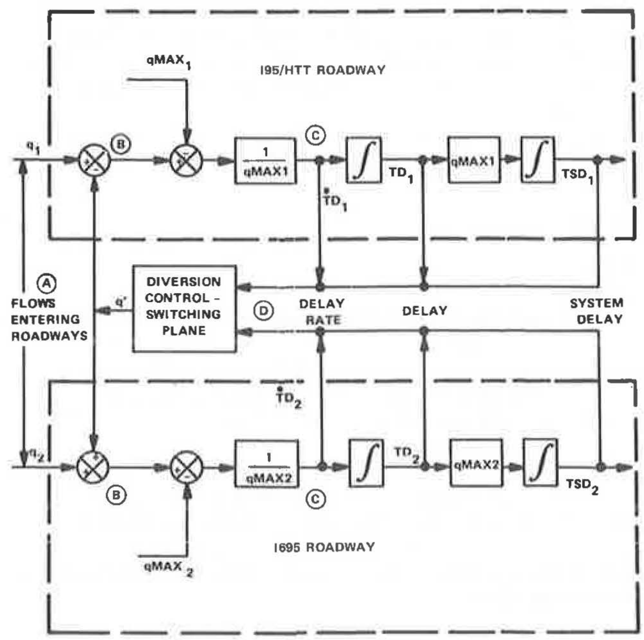


Figure 5. Mean roadway speed on (a) I-95/HTT and (b) I-695.

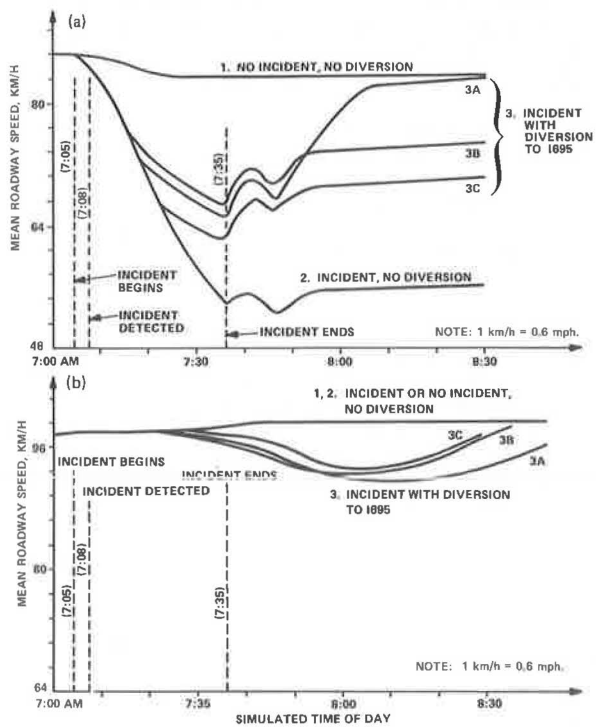


Table 1. Candidate control criteria and equations.

Parameter	Equation	Definition of Terms
Delay rate, TD	$TD = (q_1 - q_{MAX})/q_{MAX}$	Equation applicable during roadway incident or congestion conditions; q_1 = roadway demand flow and q_{MAX} = roadway capacity
Delay, TD	$TD = \sum_{j=1}^{N} (\Delta X_j/u_j)[1 - (u_j/uFF)]$	uFF = free-flow speed of roadway, u_j = speed of roadway section, ΔX_j = length of roadway section, and N = number of roadway sections
System delay, TDI	$TDI = \sum_{j=1}^{N} [1 - (u_j/uFF)]K_j \Delta X_j L_j \Delta T$	K_j = density of roadway section, L_j = number of lanes, and ΔT = time interval over which parameter is calculated

Table 2. Control algorithm switching times.

Scenario Event	Scenario Event Time (min from datum)[a]	Algorithm Switching Times (min from datum)[a] A	B	C
Incident begins	5			
Incident detected	8			
Diversion begins		10	8	14
Incident ends	35			
Diversion ends		54	42	42

[a]Datum = 7:00 a.m.

Figure 6. Total system delay for (a) all motorists and (b) intercity motorists.

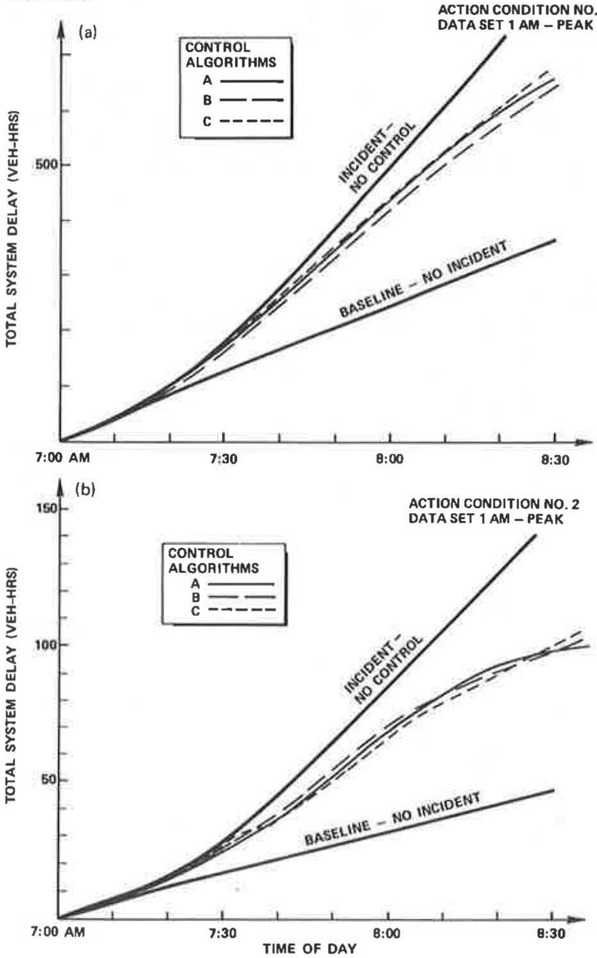
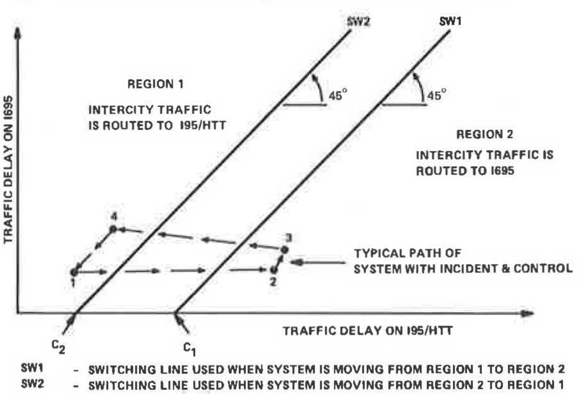


Figure 7. Diversion control algorithm difference criteria.



specified for these runs were a.m. peak roadway demand volume, incident located 1.6 km (1 mile) north of the Harbor Tunnel, incident severity sufficient to reduce capacity 50 percent on the two-lane roadway, and intercity traffic volume equal to 40 percent of traffic volume at the diversion point.

The simulation runs with control implemented require the specification of the control algorithm under evaluation. This is implemented in the STAR simulation by defining the diversion fraction α in equation 3 for each message presented to the motorist.

Based on the results of an origin-destination license plate survey and a motorist message survey (6), the fraction of intercity traffic α to use I-695 during the a.m. peak period was selected as 0.2 when route I-95/HTT is the superior route and 0.85 when route I-95/HTT is the inferior route. For other time periods, the value of α changed accordingly. It is interesting to note that theoretical analysis using queuing theory (7) determined that the design of the control algorithm is insensitive to the fraction of traffic diverting in response to a sign message. This analysis implies that the successful implementation of the control algorithm will not be severely compromised by errors in the assumed values for the fraction of intercity traffic that diverts.

Figure 5 shows the mean speed over each roadway for each of the three simulation runs. The dashed vertical lines indicate the time evolution of the scenario. Curve 1 on both figures shows the performance of each roadway under no incident, no control conditions. Curve 2 shows performance under uncontrolled incident conditions. On I-95/HTT, the mean speed degrades badly in the presence of an incident. Even after the incident ends, the roadway does not recover its preincident condition because of the heavy peak traffic demand. On I-695, since no traffic is diverted to it, the performance of the roadway remains constant. Curve 3A shows performance of the roadways with delay difference control implemented. As would be expected, there is a small degradation in performance on I-695; on I-95/HTT the improvement is significant. The control action not only improves roadway performance during the period the incident is on the road but also returns the I-95/HTT roadway to its preincident condition.

Tests of the other diversion algorithms (3B, delay difference rate, and 3C, system delay difference) for which similar runs were developed show congestion remaining throughout the peak period. This residual congestion phenomenon is caused by the premature termination of the diversion command as illustrated by the algorithm switching times given in Table 2. Based on the complete set of simulation runs, only the delay difference algorithm consistently held the diversion until all congestion resulting from the incident dissipated.

The algorithm based on the delay difference rate criteria performed erratically with an underdamped response that resulted in undesirable system oscillations. The system delay algorithm had an overdamped response. This response resulted in a system that performed sluggishly with relatively long time intervals between sign changes. These characteristics obtained from the simulation runs are based on the dynamic relationships of the three criteria shown in Figure 4. These relationships clearly show that the delay difference algorithm is superior to the other algorithms because its response is matched to the dynamics of the roadway system. The importance of these curves themselves is that they allow judgment of the operational characteristics of the roadway system (objective 1).

Figure 6 shows total system hours of delay for all motorists and all intercity motorists in the system. The lower curve in both figures represents the no incident, no control baseline case. The upper curve represents the uncontrolled incident case. The three curves in the middle represent the performance of each of three control algorithms. In all figures the curves labeled A, B, C represent the delay difference, delay difference rate, and total roadway delay algorithms. The figure taken together illustrates the success in obtaining a control algorithm that satisfies all operational objectives.

DISCUSSION OF RESULTS

The simulated performance of the I-95/HTT/I-695 roadway system was significantly improved when the single point delay difference algorithm was implemented. The intercity motorist's acceptance of his or her role as the control function will allow the actual realization of this improvement. This role will only be accepted when the intercity motorist receives a positive benefit from the operation of the system, which indicates the importance of the performance curves shown in Figure 6b. As was shown by Berger and Shaw (7), optimal diversion control policies can penalize some drivers for the good of the system. This conflict between individual and social optimization appears in many service system problems and was resolved in the single point delay difference algorithm by the introduction of the double switching lines (Figure 7). The explanation of this hysteresis is that, for vehicles that will experience small delays at the onset of an incident (trajectory segment 1, 2 in Figure 7), a high threshold (switching line SW1) causes only minor increases in total delay. For vehicles that will experience large delays at the latter stages of an incident (trajectory segment 3, 4), a low threshold (switching line SW2) is required if nearly minimal total delay is to be achieved. The low threshold more closely corresponds to the optimal threshold obtained by Berger and Shaw (7). The switching coefficients are C1, the amount of delay a motorist will tolerate before diverting to the alternate route, and C2, the amount of delay a motorist normally encounters on the primary roadway during nonincident, noncongested flow conditions over a specific time period (peak or off peak). The coefficients can be calibrated to a specific single point roadway system by setting C1 equal to the difference in free-flow travel times between the two roadways

$$C1 = TTFF2 - TTFF1 \tag{7}$$

and by setting C2 equal to the difference of the normal travel time and the free-flow travel time on the primary route

$$C2 = TT - TTFF1 \tag{8}$$

For the I-95/HTT/I-695 roadway system, these coefficients were set to 12 and 2.5 min respectively.

CONCLUSIONS

The objective was to advance the state of the art in the development of diversion control algorithms suitable for a multiroadway freeway system. This objective has been realized through the resolution of a real-world traffic control problem requiring the real-time diversion of traffic flows over a freeway system and through the development of a class of diversion control algorithms that can be used to implement the single point diversion concept for control of freeway traffic.

The diversion algorithms examined in this paper, specifically the delay difference algorithm, have been specified for further testing and implementation on the I-95/HTT/I-695 roadway system whenever the system is installed. It is important to note that this algorithm satisfies the three operational objectives discussed. It satisfies the first objective on freeway operation by quickly returning the roadways, after an incident has occurred, to their preincident, noncongested condition. Satisfying this objective essentially meets the requirements of the traffic system operator (traffic engineer) inasmuch as the roadways are being used most efficiently to maintain uncongested free-flow conditions.

By satisfying the second objective, a 30 percent reduction in system delay experienced by the intercity motorists, the algorithm provides the positive benefit to the group of motorists who provide the control authority necessary for actual system operation. Finally, the third objective is satisfied by realizing a 20 percent reduction in total system delay over the uncontrolled operation of the system during an incident. By meeting this objective, the algorithm provides a proven benefit to the system users.

In the future, the concept of single point traffic diversion to a single alternate route can be expanded by investigating and developing single point diversion control algorithms for multiple alternate routes.

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Discussion

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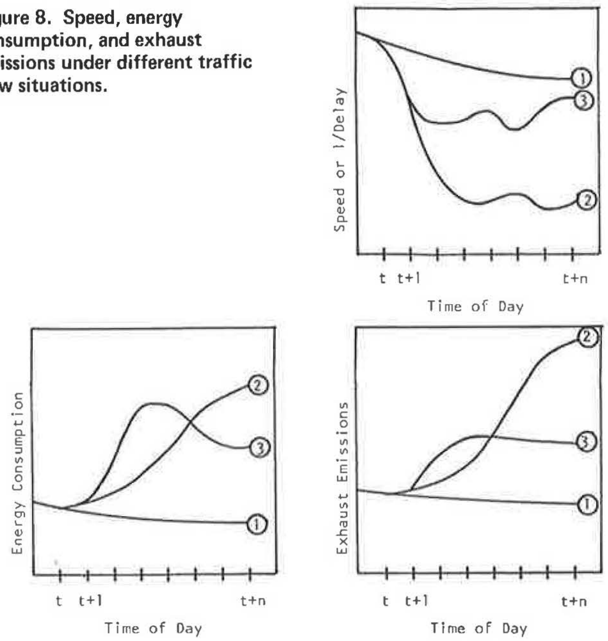
The paper presents a good technique for diverting traffic to minimize system delays in a freeway network when one or more alternative paths are available for traveling from one node to another. The algorithm substantially reduces delay and improves traffic flows. However, this is done without considering other criteria such as energy consumption, exhaust emissions, and so on. Such an approach is likely to lead to development of traffic control systems that may be inadequate when considered in the light of multiple traffic management objectives. This is illustrated by an example.

Consider a set of three goals: reduce traffic delays, reduce exhaust emissions, and reduce energy consumption. When the single point diversion algorithm is applied, it optimizes the first criterion. However, as can be seen from the I-95/I-695 example, a reduction in traffic delays also leads to increased vehicle-kilometers of travel. This will obviously affect both the fuel consumption and exhaust emission levels.

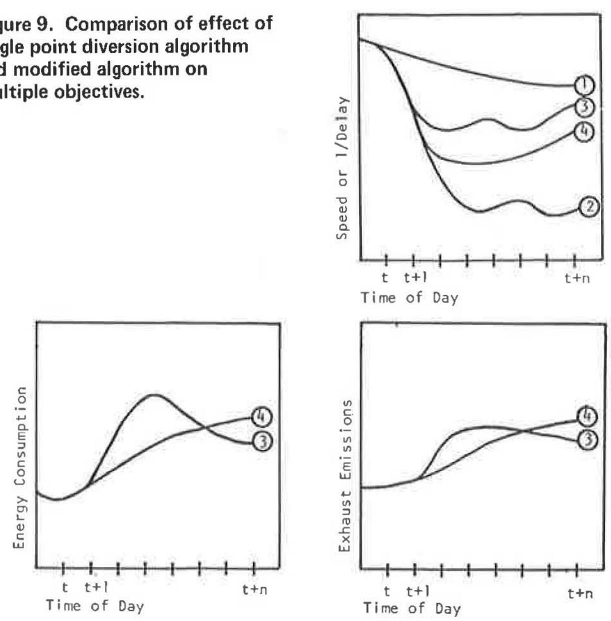
Because the algorithm improves traffic flows, the unit emission and energy consumption rates (per kilometer of operation) will be lower. The overall emissions and energy consumption levels for alternative

Figure 8. Speed, energy consumption, and exhaust emissions under different traffic flow situations.



1. No Incident
2. Incident - No Control
3. Incident - Single Point Diversion Using Algorithm

Figure 9. Comparison of effect of single point diversion algorithm and modified algorithm on multiple objectives.



1. No Incident
2. Incident - No Control
3. Incident - Single Point Diversion Using Algorithm
4. Incident - Using Modified Algorithm

traffic flow situations (no incident, incident with no control, and incident with single point diversion control) will depend on

1. The nature of the incident—minor, major, or critical,
2. Vehicle-kilometers of travel, and
3. Unit emission and energy consumption factors.

If the incident is minor and causes some traffic flow interruption, the algorithm will reduce traffic delays. However, this is likely to lead to higher emission and energy consumption levels due to the increased vehicle-kilometers of travel.

On the other hand, when the incident is critical and causes long traffic delays (many stops, extensive idling), the algorithm will substantially reduce delays. The change in emissions will depend on the relative magnitude of the increase in travel distance and traffic interruptions. A small to moderate diversion resulting in, say, a 50 percent increase in vehicle-kilometers of travel is likely to have lower emissions because of favorable unit emission rates. However, if diversion is large, the emissions may be higher when the algorithm is applied to divert traffic.

This may argue for a control algorithm that delays diversion during minor incidents. The actual strategy will of course have to be derived on the basis of the interactions among the above three factors.

Representative curves for the three traffic management goals are shown in Figure 8. The incident is seen to occur at time t, causing a sharp drop in speed or increase in delay. The algorithm causes diversions, and the effect takes place at time t + 1. This substantially improves speed but may cause a transient increase in energy consumption and emissions. Consequently, the optimum strategy may be to divert traffic at some time, t + Δt, when the overall impact is beneficial. The effect of such a modified algorithm is shown compared with the single point diversion algorithm in Figure 9.

Optimizing over a multidimensional space will no doubt increase the complexity of the algorithm. However, simple combinatorial techniques (e.g., weighted worths) can be used as a compromise. I feel that consideration of multiple objectives, even in a simplistic manner, is necessary to ensure that the traffic control strategies developed are beneficial from the total system standpoint.

Authors' Closure

The authors would like to thank Budhraja for taking the time to comment on our paper. The point that he raises is of course well taken in the context of freeway traffic management. We would like to raise several points in response to Budhraja's discussion.

First, it is important to note that the operation of the single point algorithm is based on the relative performance of each roadway as defined with respect to the parameters of vehicle travel delay. Thus, the placement of the switching lines in the control plane (Figure 7) determines the operation of the algorithm for a given traffic scenario. For the results presented in the paper, the location of the switching lines was based explicitly on minimizing system delay for all motorists and for the intercity motorist. Maximizing mean roadway speed was also explicitly examined. The question to be answered is then, Where will the switching lines be located (in the switching plane) if the minimization of exhaust emissions and energy consumption is explicitly considered? Whichever measure of effectiveness or combination is examined, the complexity of the algorithm is not increased; control is still defined within the context of a switching plane.

Also, it is not at all clear that a control policy explicitly designed to minimize delay would not also minimize exhaust emissions and fuel consumption (at least in a suboptimal sense). Our first reaction is to say that the policy would not, however, as Budhraja points out, because vehicle-kilometers of travel would increase and the unit emission and energy consumption factors are dependent on speed. The final word is not yet in.

It is thus appropriate to suggest the need for additional research to examine explicitly the influence of the objectives of minimizing fuel consumption and exhaust emissions on the operation and control of multiroadway freeway systems.

Operating Parameters for Main-Line Sensors in Freeway Surveillance Systems

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Freeway surveillance and control systems obtain most of their input information from discrete vehicle presence sensors. Performance measures, for both on-line control and off-line evaluation, must be determined from the presence data. The most significant measures include speed, flow rate, and detector occupancy. To obtain accurate estimates of the measures requires that several parameters be established for the sensor configuration and for the computational algorithms. These include size of detection zone, detector scanning interval, vehicle lengths, speed distribution characteristics, averaging period, and degree of feedback of the average speed into the calculation of individual speeds. The relationship between these parameters is complex and does not lend itself to a simple analytical treatment. Therefore, a simulation model was developed to analyze the effect of the various parameters on the accuracy of the measurements obtained from discrete sensors. Through the simulation model, several relationships were investigated. Based on these relationships, some important conclusions and recommendations are offered for the design and operation of discrete sensor systems for freeway surveillance.

Discrete sensors used in traffic surveillance systems provide only binary information. In other words, they simply indicate whether or not a vehicle is passing through the zone of detection. These inputs must somehow be digested and converted to information that is more meaningful to the user.

There are two basic units of information available directly from discrete sensors:

1. The flow rate, i.e., the number of actuations recorded by the detector during a given time period; and
2. Time occupancy, i.e., the relative proportion of time that the detector was activated by a vehicle (this is a dimensionless number, usually expressed as a percentage).

The average speed at the point over a given time period may be estimated by dividing the flow rate by the detector occupancy. In this case, the detector occupancy is used as an estimator of equivalent density at the point. By definition, the speed is equal to the flow rate divided by the density. Space mean speed is total distance traveled divided by the total time required or

$$V_s = [Q(l + w)p/\theta p] \qquad (1)$$

where

V_s = space mean speed,
Q = flow rate,
l = average vehicle length,
w = sensor dimension in direction of traffic flow,
p = length of averaging period, and
θ = proportion of time the sensor is occupied by a vehicle during averaging period.

This reduces to

$$V_s = (Q/\theta)L \qquad (2)$$

where L is the combined length of the vehicle and sensor. Thus, L is a constant of proportionality that expresses speed as flow divided by occupancy.

There are two primary sources of error within this calculation.

1. The actual length of the vehicles in a given sample is unknown and must, therefore, be estimated by an average value.
2. The exact time period of occupancy cannot be determined economically, especially when the detector is located a significant distance from the surveillance center. Instead, a periodic scanning process must be used to estimate this time interval.

The accuracy of the speed estimate obtained from equation 2 depends on several factors including

1. Size of detection zone,
2. Scanning interval (resolution),
3. Vehicle length distribution,
4. Speed distribution,
5. Volume,

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6. Averaging period, and
7. Computational method.

This paper considers the effect of each of these factors.

SENSOR REQUIREMENTS FOR FLOW MEASUREMENTS

Volume counts obtained from a single discrete sensor in each lane can have errors caused by

1. Failure of the detector to recognize the presence of a vehicle,
2. Lane straddling by vehicles, which results in either missed or double counts,
3. Tailgating, which causes two successive vehicles to be recognized as a single unit, or
4. Too slow a detector sampling rate, which allows vehicles (or intervehicle gaps) to pass through the system unnoticed.

The first problem is one of proper design and installation of the detector unit. The use of state-of-the-art equipment and techniques should eliminate difficulties in this area.

The second problem may be minimized by placing sensors in locations where lane changing maneuvers are not excessive. If a high degree of precision is required where frequent lane changes occur, it may be necessary to install an additional sensor between each lane. This technique requires a more complex processing algorithm and is therefore best suited to systems that are monitored by a central computer. It has been used successfully on the John C. Lodge Freeway in Detroit (1).

The solution to the third problem lies in keeping the longitudinal dimension of the sensor to a minimum. The minimum loop size recommended by many manufacturers is approximately 2 m (6 ft). Drivers seldom follow the vehicle in front of them by less than 2 m (6 ft) in moving traffic. However, as discussed later, increasing loop size somewhat does have advantages from the point of view of speed measurement accuracy.

The problem of sampling rates is of particular importance where a central computer is used for monitoring the field sensors, especially where long communication distances are involved. The sampling rate is a significant factor in communication costs; therefore, the minimum sampling rate that will provide the required degree of accuracy should be chosen.

To ensure that no traffic events (either vehicles or gaps) will be missed requires that the sampling interval be shorter than the minimum duration of an event. This requires a separate analysis for vehicle presence and gaps.

The minimum vehicle presence duration may be calculated as

$$D_p = (l + w)/V \qquad (3)$$

where

D_p = duration of presence of a vehicle on the detector in seconds and
V = vehicle speed.

By assuming reasonable values for the vehicle following distance (4.3 m or 14 ft minimum) and for the detector size (2 m or 6 ft), we can calculate the presence duration as a function of speed. This relationship is shown in Figure 1.

The gap duration is somewhat more elusive because it depends very heavily on driver behavior. The duration of a gap between vehicles depends on how closely drivers are willing to follow each other. From a worst case point of view, this figure may be expressed in terms of either a minimum time gap or a minimum distance gap. In both cases, the detected gap duration is a function of speed. If a minimum time gap is assumed, the detected gap duration is expressed as

$$D_G = (VT_G - w)/V \qquad (4)$$

where

D_G = detected duration of the gap,
V = speed (both vehicles are assumed to be traveling at the same speed), and
T_G = time gap between vehicles.

Note that the detected gap duration is smaller than the actual gap duration because of the width of the sensor, which must not be occupied by either vehicle during the gap detection interval.

The previously assumed sensor size of 2 m (6 ft) was used to plot the detected gap durations shown in Figure 1 for a minimum intervehicle gap of 0.3 s, which may be considered as a conservative value.

If a minimum distance gap is assumed, the detected gap duration is expressed as

$$D_G = (L_G - w)/V \qquad (5)$$

where L_G = the length of the minimum gap.

If the minimum following distance is assumed to be one car length, then the detected gap becomes

$$D_G = (l - w)/V \qquad (6)$$

The detected gap duration is shown in Figure 1 as a function of speed, based on the 4.3-m (14-ft) minimum value established previously.

The three relationships shown in Figure 1 may be used to establish the minimum scan interval for traffic counting. Each relationship governs the scan interval throughout a specific speed range. In the lower speed range, the minimum following distance establishes the maximum scan interval. This value reaches approximately 160 ms at 14.6 m/s (48 ft/s). If the minimum following distance were used at higher speeds, the minimum intervehicle time gap of 0.3 s would be violated. The minimum intervehicle gap continues to be the dominant factor until the speed reaches 26 m/s (85 ft/s), at which point the duration of vehicle presence time becomes more critical. Vehicle presence time continues to dominate for the rest of the speed range. At the upper limit of the speed range, the minimum required duration again reaches 160 ms.

The foregoing analysis may be considered valid for moving traffic when processing algorithms require only one scan interval to establish vehicle presence or a gap. It is common practice in freeway surveillance to require two successive scan intervals to establish either condition. In such cases, the scan rate must be doubled. However, refining the algorithms to eliminate the need for two successive intervals at very low speeds may improve the accuracy of volume counting inasmuch as the assumption of a minimum following distance is less valid in this speed range. Detected gap durations based on minimum gap times are also extremely short at low speeds as shown in Figure 1.

SPEED VERSUS OCCUPANCY

Flow rate values do not provide, by themselves, an adequate description of the operation of a freeway. For example, a low rate of flow at any particular time may be an indication of low demand or heavy congestion. Additional measures such as speed or occupancy must be used to resolve this ambiguity. Occupancy is used in several existing surveillance systems, partly because it is easier to measure than speed and partly because it is a more meaningful indicator of the degree of use of the facility under low-volume conditions. Speed on the other hand tends to be a more stable measure when demand approaches capacity. At this operating level, occupancy values tend to remain at the same level as the operation degenerates from an area of relatively stable flow to conditions of forced flow. The same is true for the reverse process, which occurs as congestion is dissipated. Therefore, both speed estimates and occupancy measurements from main-line freeway sensors can be used.

Several factors affect the accuracy of speed estimates made with a single discrete sensor. The complex interrelationships between these factors do not, unfortunately, lend themselves to simple analytical treatment. They do, however, provide the basis for a practical simulation model. Such a model was developed to analyze the accuracy of a set of conditions specified by assigning values to the various operating parameters.

The accuracy of the speed computations for any given configuration may be expressed in terms of either error or bias. Error is the discrepancy between the calculated and actual velocities for a given sampling period. It is represented by a root mean square (rms) value that is always positive regardless of the direction of error. Bias, on the other hand, is the algebraic mean error over a large number of samples and may be either positive or negative. It is possible, therefore, to have a configuration with a large error and a very small bias. It is not, on the other hand, possible for the bias to exceed the rms error. Bias and error are both examined in this analysis. However, error is the preferred measure because correction factors can be applied to eliminate a known bias condition.

DESCRIPTION OF SIMULATION MODEL

The simulation model developed for this analysis is actually a combination of two submodels:

1. A stochastic submodel that generates a set of vehicles and determines a speed, length, arrival time, and departure time from the presence sensor by using Monte Carlo techniques (a sample normally consists of 3000 vehicles) and
2. A deterministic submodel that scans the table of vehicles and examines the output of the sensor (present or absent) for predetermined scanning intervals.

The speeds of individual vehicles and average speeds for specified time periods are estimated by using the outputs of the deterministic submodel. Comparing these estimates with the known speeds obtained from the stochastic submodel allows the accuracy of a given configuration of parameters to be determined.

Three distributions are involved in the stochastic submodel. The first is the headway distribution, which is a shifted negative exponential distribution whose mean headway is determined by the traffic volume and is shifted to allow a minimum headway of 1.5 s between successive vehicles. Second, speed is considered to be normally distributed, and its mean and standard deviation are determined as a function of freeway volume by using the relationship given in the Highway Capacity Manual (2) and in Table 1. The third distribution determines the length of each vehicle. This is an empirical distribution derived from an analysis of data obtained on Interstate 75 in Tampa, Florida. This distribution (Figure 2) has a mean vehicle length of approximately 5.6 m (18.5 ft) with a standard deviation of approximately 2 m (6 ft).

Input Requirements

To establish a configuration for analysis requires the following inputs:

1. Traffic volume (0 to 2000 vehicles/lane/h),
2. Scanning interval (1 to 500 ms), and
3. Averaging period (constant time interval or constant number of vehicles in the averaging sample).

Model Outputs

For each case as specified by a flow rate, a scan interval, and an averaging period, the following values are calculated:

1. Percentage of error for individual vehicle speeds,
2. Percentage of error for time mean period average,
3. Percentage of error for space mean period average,
4. Percentage of bias for individual speeds,
5. Percentage of bias for time mean period average, and
6. Percentage of bias for space mean period average.

The time mean period averages are calculated as the arithmetic mean of the individual speed values. The space mean period averages are calculated as the harmonic mean of the individual speed values.

INDIVIDUAL SPEED CALCULATIONS

The most important factor in the calculation of individual speeds is the scan rate because it determines the accuracy of the time measurement on which the speed calculations are based. Figure 3 shows this relationship. Both error and bias are shown in this figure, which covers the full range of scan intervals to a maximum of 500 ms. The minimum expected error, corresponding to the shortest possible scanning interval, is approximately 19 percent.

This error may be expected to increase monotonically as the scanning interval increases; however, a resonant effect is observed in the vicinity of the 300-ms scanning interval. At scanning intervals of this magnitude a vehicle will tend to be recognized in the loop for one scanning period. The average speed for this sample was 22 m/s (73 ft/s). For an average combined vehicle plus sensor length of 7.5 m (24.5 ft), the average time spent in the detection area by each vehicle is 297 ms. The low error that occurs at this point simply represents a coincidence between the length of one scanning interval and the speed represented by that length. This resonant point will occur at different scanning intervals for different values of assumed average speed and vehicle length. It is therefore of no value to the speed computations. Accurate computations will in fact require that the scanning interval be kept below the range in which this resonant effect is observed. This limits the scanning interval to approximately 200 ms.

Figure 1. Event durations versus vehicle speed.

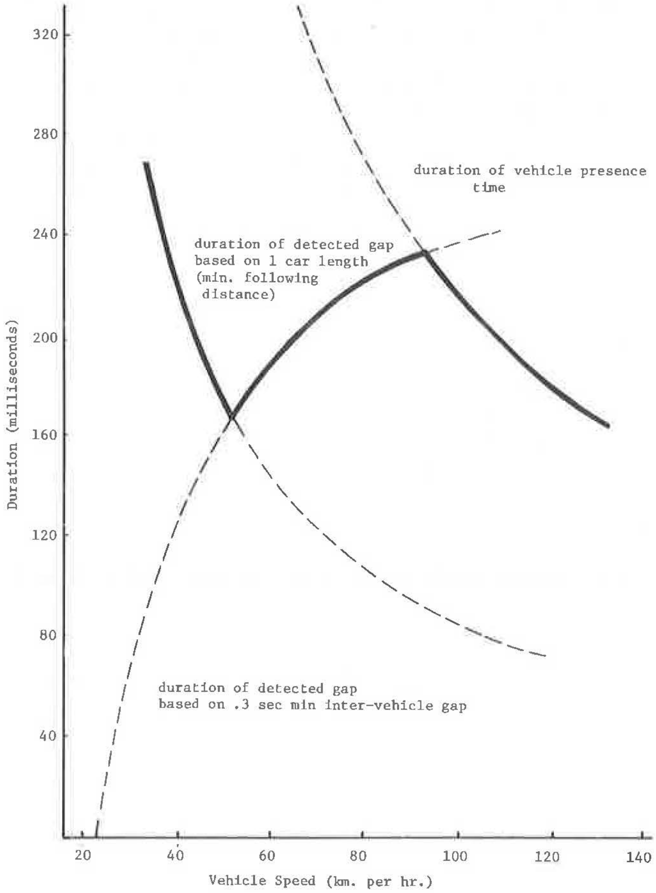


Table 1. Relationship between volume and speed.

Freeway Lane Volume (vehicles/h)	Mean Vehicle Speed (km/h)	Approximate Speed Standard Deviation (km/h)
200	95	12.1
700	89	8.8
1000	84	7.2
1500	76	7.2
1800	63	6.4
2000	51	3.2

Note: 1 km/h = 0.62 mph.

Figure 2. Cumulative distribution of vehicle lengths used in simulation analysis.

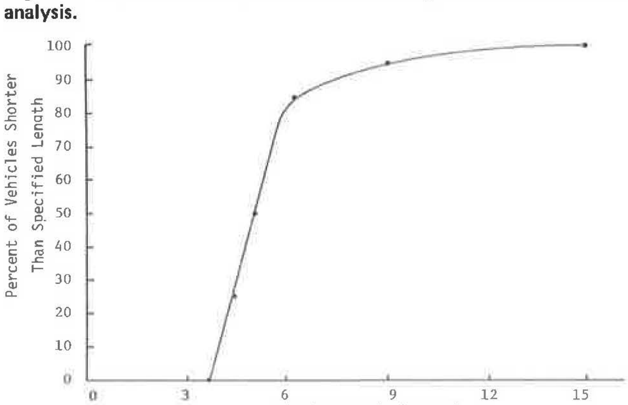


Figure 3. Effect of scan interval on error and bias for individual vehicle speed estimates based on a single presence detector.

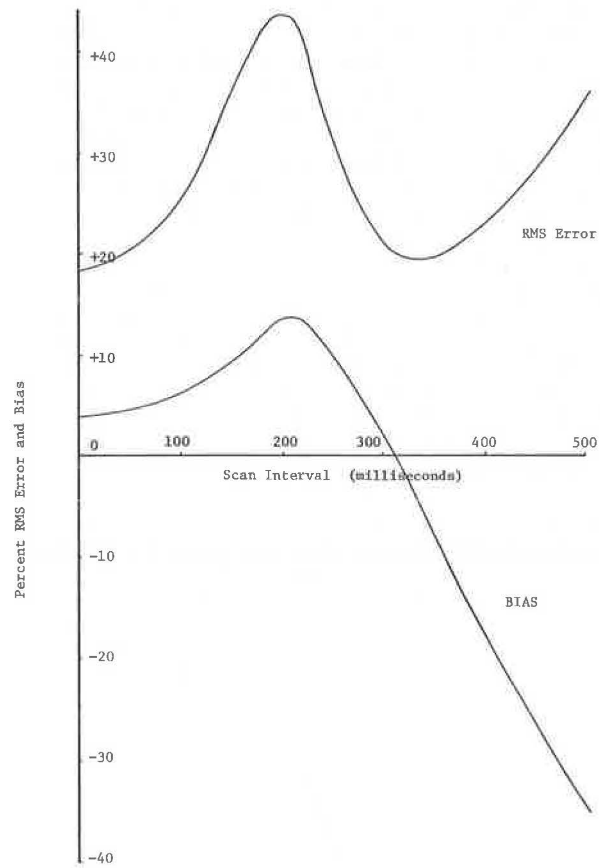
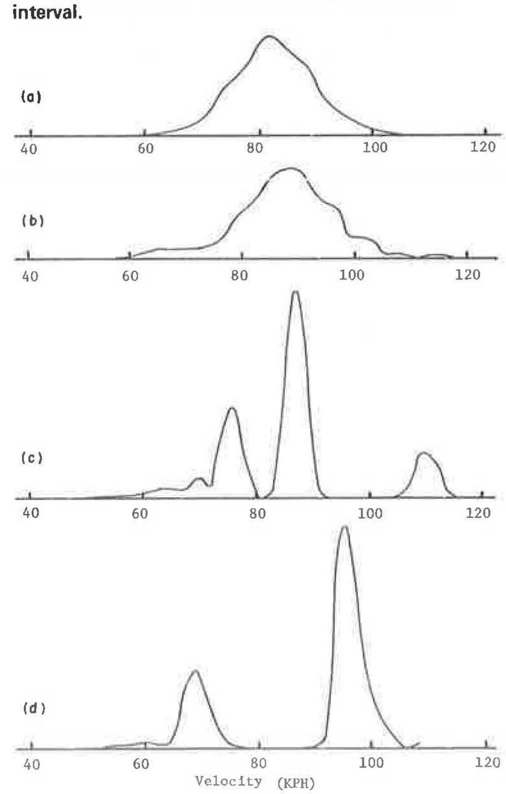


Figure 4. (a) Actual speed distribution and distortion due to (b) 20-ms, (c) 100-ms, and (d) 250-ms scan interval.



The effect of scanning interval on bias is also very interesting. No resonant effect is observed; however, the bias does cross the zero axis and shifts from positive (overestimate) to negative (underestimate) somewhere around the resonant point.

The distribution of computed speeds is also worth examining. A histogram of the actual speeds generated by the simulation model (Figure 4a) illustrates excellent conformance with the normal distribution. The distortion in this distribution that takes place as the scanning interval is increased from 20 to 500 ms is shown in Figure 4. The 20-ms case (Figure 4b) shows very little deviation from the actual distribution. Subsequent cases, however, show a marked grouping of the values into a bimodal and trimodal shape. Note, for instance, the two distinctly separated normal distributions in Figure 4d, corresponding to a 250-ms scanning period. As the scanning interval increases even further, the distribution tends to take on a single value corresponding to the speed calculated by using a vehicle of average length that occupies the loop for one scanning interval. This creates an increasing error accompanied by a strong negative bias.

The general conclusion to be drawn from Figure 4 is that rough estimates (approximately 20 percent error) may be obtained if the scan interval is kept very short. The validity of the individual calculations, however, begins to deteriorate rapidly above 20 ms and, above 50 ms, individual calculations have little or no significance.

AVERAGE SPEED CALCULATIONS

Individual speeds, fortunately, are not an important consideration in most traffic surveillance and control applications; the average speed of the traffic stream at a specific time period is of more interest. Several important questions arise in connection with the measurement of average speeds.

1. What is the effect of the scanning interval on the measurement accuracy?
2. Is time mean speed preferable to space mean speed from an accuracy point of view?
3. Is a constant time period preferable to a constant sample size for averaging purposes?
4. What is the effect of different lengths of averaging periods?
5. What is the effect of traffic volume on the accuracy of speed computations?
6. What is the effect of feedback of previous speeds in the speed computations on the error analysis?
7. What is the effect of the sensor length computational accuracy?

These questions have all been examined by using the simulation model and are discussed separately.

Scan Interval

The scan interval is limited to a maximum value of about 200 ms by two factors:

1. The requirement for accurate counting of vehicles and
2. The resonant effect observed above 200 ms.

The effect of scan interval on accuracy is shown in Figure 5, which assumes representative traffic conditions and surveillance system operating parameters. The effect of varying these assumptions is shown in subsequent figures. A relatively flat relationship is shown in Figure 5, and the error varies from approximately 5 to 7 percent (without feedback) throughout the full range of scan intervals. This suggests that relatively long scan intervals will be more cost effective in many surveillance applications.

Effect of Feedback of Previous Speeds

The detector occupancy time for an individual vehicle depends on both the length and the speed of the vehicle. If the length is assumed constant, as in equation 2, all of the occupancy time variation between vehicles will be erroneously associated with speed variation, and a larger error will result. At the other extreme, if the speed is assumed constant, the speed measurement capability will be destroyed. Between these two extremes lies a solution that reduces the speed measurement error due to the distribution of vehicle lengths without introducing other errors in the process.

The technique investigated involves feeding back the calculated speed from the previous averaging period into the calculations for the current averaging period to produce more accurate estimates.

$$V_i = \alpha S_x + (1 - \alpha)[(l + w)/T] \qquad (7)$$

where

V_i = corrected individual vehicle speed,
S_x = space mean speed computed in the last averaging period,
α = weighting coefficient ($0 \leq \alpha < 1$), and
T = presence duration of the individual vehicle when it passes over the sensor.

The open loop computations were made with $\alpha = 0$ so that no consideration was given to the previously computed speed. Length and occupancy calculations were updated only when a vehicle left the loop. This precludes the problem of a partial vehicle at the end of an averaging period. The speed estimates produced in this manner are equivalent to Mikhalkin's estimator three (3). Increasing α to 1.0 amounts to assuming that speed is, in fact, constant.

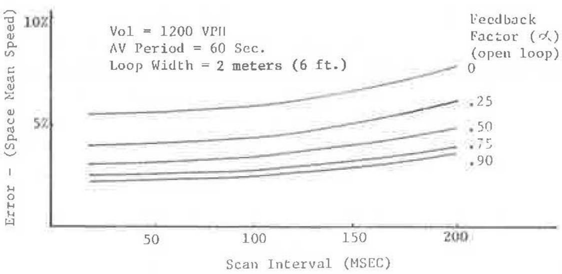
The effect of varying degrees of feedback is shown in Figure 5. Values of α shown in the figure range from 0 to 90 percent. For the given conditions of 1200 vehicles/h/lane and averaging period of 60 s, the accuracy was improved to a significant degree as α was increased. It must be recognized, however, that there is a tradeoff between accuracy of individual computations in the steady state and speed of response to a definite change in freeway speed as a function of time. The condition of fluctuating speed is extremely difficult to simulate realistically, and therefore as a conservative measure the feedback parameter was maintained at a value of 0.5 for all of the analyses discussed. Before a final value is established, tests should be conducted on an operational surveillance system.

Time Mean Speed Versus Space Mean Speed

Time mean speed (estimated by the arithmetic mean) will give a heavier weight to individual vehicle speeds that are excessively high. Space mean speed (estimated by the harmonic mean) on the other hand will weigh more heavily on vehicle speeds that are excessively low. Which method is preferable will therefore depend on whether the greatest potential for gross error is on the high side or the low side. Data shown in Figure 6 indicate that for each of the four averaging periods represented the space mean speed is clearly preferable to the time mean speed in terms of both error and bias. However, especially with longer averaging periods, the error and bias for the time mean speed are close to each other. This indicates that a bias correction may be applied to time mean speed, which may reduce the error substantially.

Figure 7 shows the results of a bias correction factor on the time mean speed. In this case, the errors for corrected time mean speeds are plotted along with space mean speed for various averaging periods as a function of scanning interval. The time mean speed shows a lower error than the space mean speed does in all cases. There may, therefore, be some advantage to using time mean speed depending on the objectives of the surveillance system. The advantage, however, is very slight, as shown in Figure 7. On the other hand, because the space mean speed is involved in the speed-flow-density relationship and the harmonic mean is simpler to measure from a computational point of view, the space mean speed is likely to emerge as the preferred choice in most applications.

Figure 5. Effect of feedback factor on error.



Constant Time Period Versus Constant Sample Size

There is, at least theoretically, an advantage in choosing the averaging period to contain a constant number of vehicles as opposed to a constant length of time. This question was investigated by using the simulation model

Figure 6. Error and bias in time mean speed and space mean speed estimates.

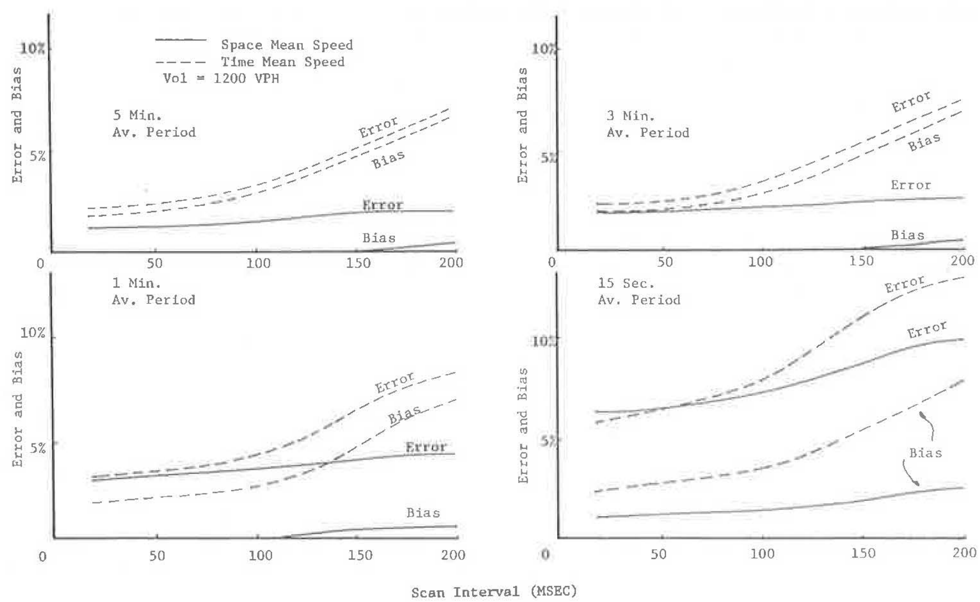
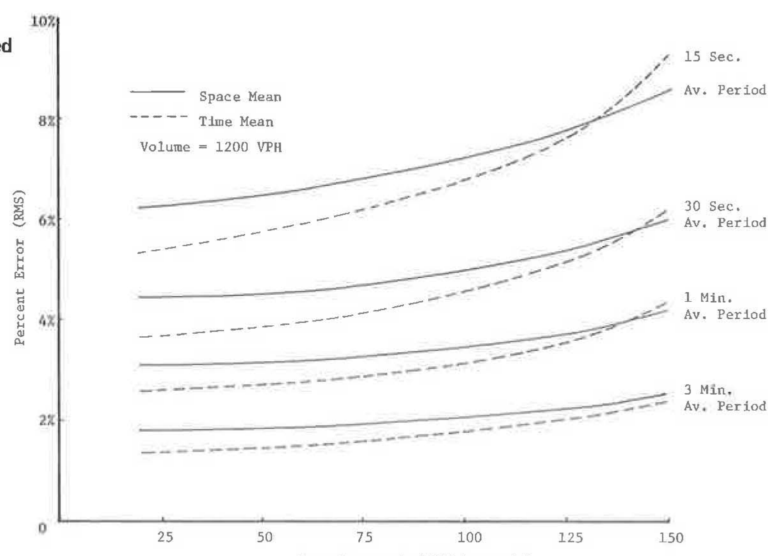


Figure 7. Accuracy of space mean speed and time mean speed (corrected for bias).



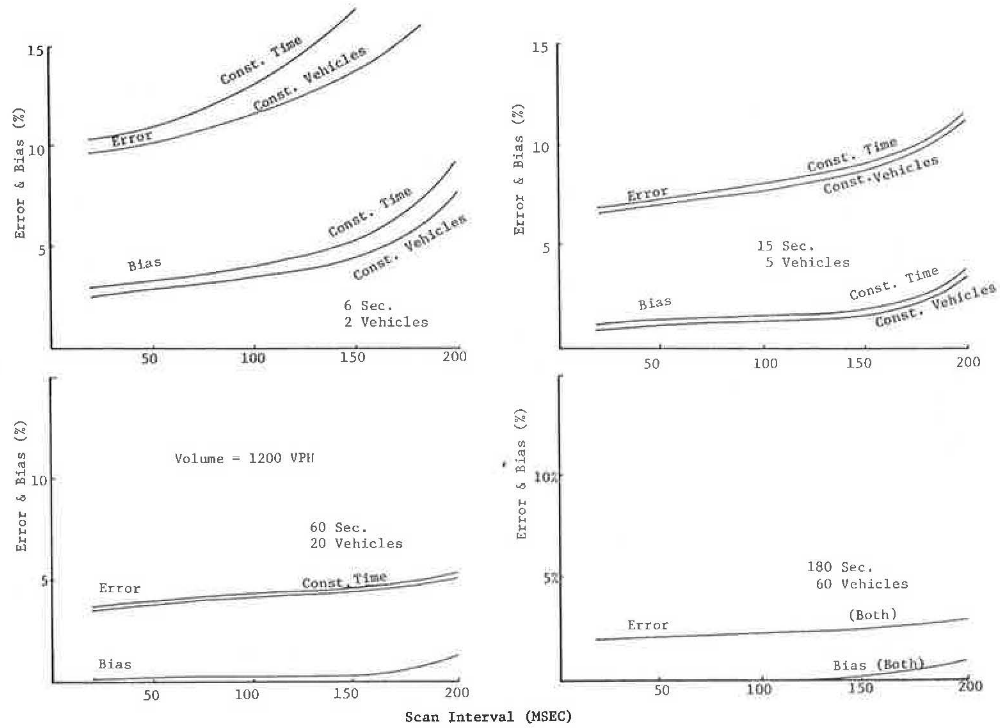


Figure 8. Accuracy of constant time period and constant vehicle sample averages.

Figure 9. Effect of averaging period on error.

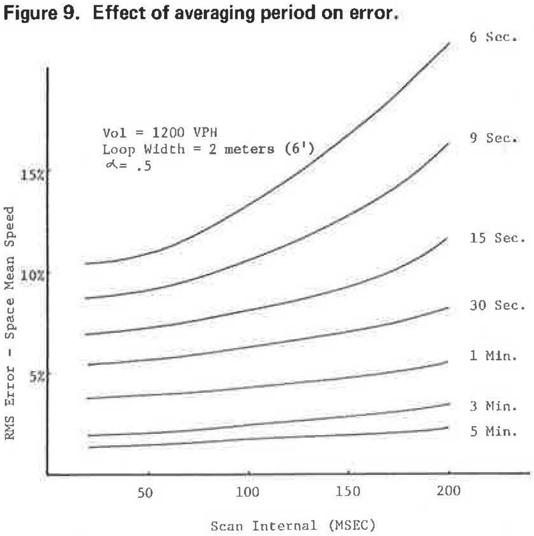


Figure 10. Effect of lane volume on error.

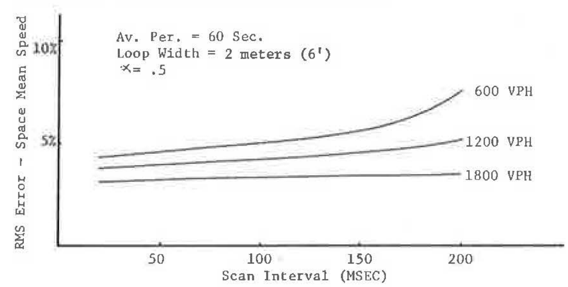
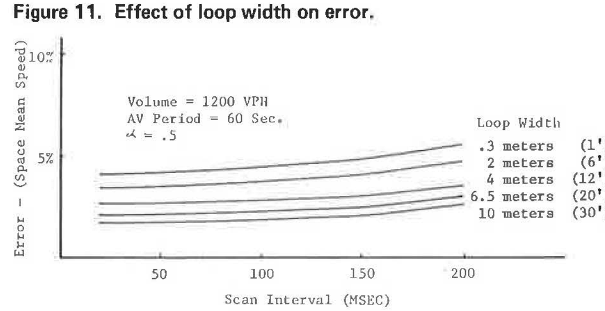


Figure 11. Effect of loop width on error.



(Figure 8). Figure 8 shows, for four assumed averaging periods, the error and bias associated with constant time averaging versus constant vehicle averaging. A noticeable difference exists only in the case of very short sampling periods. For the chosen flow value (1200 vehicles/h) only the 6-s, 2-vehicle sampling period exhibited a worthwhile improvement for constant vehicle averaging. This improvement subsided very quickly as the averaging period increased and became imperceptible under the 60-s, 20-vehicle case. Because of the computational difficulty of constant vehicle averaging, i.e., the asynchronous condition that would result if several ramps were treated in this manner simultaneously, it is suggested that constant time averaging should be used in operational surveillance systems.

Effect of Averaging Period

Whether a constant time or constant vehicle averaging period is chosen, the length of the period will have a substantial effect on the accuracy of computations, inasmuch as longer periods will tend to give larger sample sizes. The relationship between error and averaging period, derived from the simulation model, is shown in Figure 9. The errors below 10 percent may be expected with reasonable scanning intervals for averaging periods of 0.25 min or so. On the other hand, a 5 percent error requires that the averaging period be extended to about 1 min.

Effect of Traffic Volume

The data describing the effect of averaging period shown in Figure 1 were based on a traffic volume of 1200 vehicles/h. To examine the effect of volume on the error analysis, separate runs were made assuming an averaging period of 60 s for volumes of 600, 1200, and 1800 vehicles/h. The results of this analysis are shown in Figure 10. As anticipated, error tends to rise slightly as volume decreases. However, even under the fairly low volume of 600 vehicles/h, the error still remains in the 5 percent range for the 60-s sampling period. From a freeway surveillance point of view, main-line volumes of 600 vehicles/lane/h are considered extremely light. It would not generally be desirable to compromise the peak-period computation by designing the system for high accuracy in velocity measurements at volumes of less than 600 vehicles/h.

Effect of Sensor Length

Speed computations may be expected to improve as the length of the sensor (parallel to travel direction) is increased. The sensor itself contributes a speed trap effect to the measurement system, and increasing its length may therefore reduce measurement errors. This effect was simulated for sensor lengths between 0.3 and 9 m (1 and 30 ft), and a significant degree of improvement in speed computations was observed (Figure 11). It appears that, by using a sensor width of 6 m (20 ft), errors may be reduced by approximately 50 percent over the 1.8-m (6-ft) loop that is more or less a current standard. This improvement must of course be traded off against the increased cost of installing longer loops plus the increased error in counting due to very close vehicle headways. It is suggested, however, that 6-m (20-ft) loops could be used successfully for speed measurement in most freeway surveillance systems provided that computational algorithms did not require extremely high accuracy for counting individual vehicles under jam density conditions.

Another advantage to the longer loop that is not incorporated into the simulation model is the fact that the electrical length of the loop, which is known to change somewhat with ambient conditions, will vary by a smaller percentage of the loop length. Thus a further improvement in measurement accuracy may be possible.

CONCLUSIONS AND RECOMMENDATIONS

Based on the analysis, the following recommendations are offered for the design and operation of main-line traffic sensor systems for freeway surveillance.

1. Estimates of average speed should be made in addition to the measurements of detector occupancy.
2. The sampling process should be based on a constant time period rather than a constant number of vehicles. A 1-min interval is adequate for most purposes.
3. Space mean speed should be measured and not time mean speed.
4. A detector sampling rate of about 10 samples/s gives sufficient accuracy.
5. Computation of the individual vehicle speed should include feedback of the average vehicle speed.
6. There should be at least one speed trap on the freeway to measure average vehicle length, which, in turn, is used to compute individual vehicle speeds.
7. The accuracy of speed estimates may be improved by increasing the longitudinal dimension (dimension in the traffic flow direction) of the main-line detectors to a value larger than that in current installations. A maximum length of 4.6 to 6 m (15 to 20 ft) is recommended, provided that there is no need to accurately count individual vehicles under jam conditions. However, where accurate volume counts are required, the sensor dimensions should be kept to a minimum.

These recommendations apply particularly to systems involving centralized supervision by a digital computer. If the current trend toward local control by microprocessors continues, the problems of data communication over long distances will be reduced. This will also permit refinement of the data processing algorithms and may modify some of the recommendations, particularly those dealing with scanning intervals. It will also permit the use of more sensors at a given location, which, in turn, should improve the surveillance and control capability of a given system.

ACKNOWLEDGMENT

This research was supported by Stanford Research Institute and the National Cooperative Highway Research Program. The opinions, conclusions, and findings are those of the authors and not necessarily those of the supporting agencies.

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Discussion

John L. Barker, LFE Corporation

Courage, Bauer, and Ross are to be sincerely congratulated on a clear and concise paper dealing with the much needed evaluation of input sensor data to freeway and surveillance systems. Their approach is understandable; the information is usable and does not need a great deal of discussion. My one suggestion is that the authors should rewrite the paper one more time such that the result would be about halfway between the abstract and the details in their paper. There are a large number of rules, conclusions, and teachings that would be of immediate use to the operating engineer. The basic paper would still remain a reference.

To take at least some issue with the authors, I criticize the rather dogmatic recommendation made about the use of longer loops to secure added accuracy in the measurement of speed. Their suggested length of 4.6 to 6 m (15 to 20 ft) in the direction of travel, as compared to the more conventional 1.5 to 2 m (5 to 6 ft), brings up the possibility of more problems than those listed by the authors, including some degradation in volume counts due to a gap not being recognized between tailgating vehicles under jam traffic conditions and the increased costs of the larger loops.

A few other points are listed below.

1. Cross talk, i.e., one loop electrically interacting adversely with another loop in an adjacent lane, is almost directly proportional to the length of the adjacent sides. Increasing the length of the adjacent sides from 2 to 6 m (6 to 20 ft), added to the increased sensitivity used today to ensure accurate high-bodied truck and motorcycle detection, could well present a cross talk problem.

2. It is problematical whether the electrical stability of the larger loop, with respect to environmental changes, would be as good as that of the smaller loop. This would be a foregone conclusion if the larger loop had to be located such that it extended longitudinally across an expansion joint between two slabs of the roadway.

3. As the size of the loop increases, the problems of accurate detection under the conditions of lane changing become more severe, with regard to both count and occupancy. Good installations try to minimize this problem by judicious sensor location. Even with 2-m (6-ft) loops, lane changing vehicles moving diagonally across the detector give short presence pulses for occupancy and speed data; tailgaters do not provide suitable gaps between vehicles for proper count. It would be better to concentrate on slightly faster scanning rates or preprocessing of the data at the sensor before telemetry to the computer. Also, separate measurement of vehicle lengths on a sampling basis materially increases the accuracy of assumed average length of vehicles in the system and keeps it up to date with regard to changes in the mix of passenger and other vehicles.

4. The algorithm using feedback of previous speed data to compute present averages to produce more accurate estimates might just be an illusion. The improvement (Figure 5) in the accuracy versus the amount of feedback appears to be related to the nature of exponential smoothing—the scheme used to put the feedback factor into the calculation. As such, this could imply no advantage over simply taking an average over a correspondingly longer period. Further, if this is the case, this also means any apparent gain in response time is lost. I suggest that a rigid mathematical analysis or more computer runs be made to resolve whether this is an illusion or a real gain.

Robert Reiss, Sperry Rand Corporation

The authors have provided a long needed analysis of measurement errors in traffic surveillance systems. This information will be of great value to designers, particularly for choosing sampling frequencies, choosing averaging periods, and smoothing filter constants.

Although the authors are correct in pointing out the complexity of the interrelationships affecting traffic parameter estimation, certain simplified analytical approaches can often lend insight into the problem of measurement accuracy.

For example, Figure 12 shows a true detector pulse along with a pulse reconstructed from discrete samples. The error in the reconstructed pulse is the sum of the start and stop time errors t_1 - b and d - t_2 respectively. Because each of these errors has a uniform probability density, the density of the sum is triangular with characteristics as shown in Figure 13. This density has a mean of zero and a variance of $T^2/6$. Therefore, the expected percentage of rms error in a single occupancy measurement is

$$(T/\sqrt{6})(1/D_p)100 \qquad (8)$$

where D_p is the duration of vehicle presence.

For example, a 10-ms sampling interval and a pulse duration of 280 ms (approximated 96 km/h or 60 mph) result in a 1.5 percent error.

Because individual vehicle speed is proportional to the reciprocal of pulse width, the probability density function (pdf) of the speed measurement can be derived from the pulse width pdf by using the theory of transformation of random variables. For the range of nominal speeds and sampling frequencies of interest, the speed measurement pdf can be shown to be approximately triangular.

A useful follow-up to the work presented in the paper would be an error analysis for a speed trap detector configuration (two closely spaced detectors in a lane). Designers should be cognizant of the accuracy inherent in a speed trap versus the increased cost associated with its installation.

Standard error analysis techniques were used to develop the curve shown in Figure 14 illustrating the expected rms error in vehicle length measurement by using a speed trap configuration with the following parameters:

Parameter	Value	Variance
Distance between loop centerlines	4.6 m	93 cm^2
Length of loop field	2 m	465 cm^2
Speed	96 km/h	
Sampling period	$^1/_{32}$ s	

Analyses of the type indicated in this discussion usually cannot treat the entire measurement accuracy problem but can provide additional understanding of the phenomena involved.

ACKNOWLEDGMENT

I would like to acknowledge useful discussions with K. W. Dodge and H. Satz of Sperry Systems Management.

Figure 12. Detector pulse sampling.

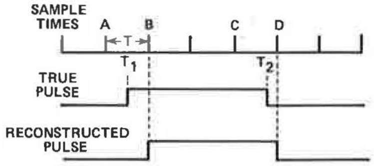


Figure 13. Probability density of pulse duration errors.

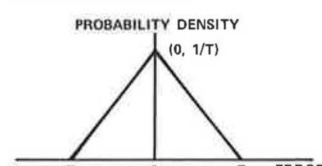
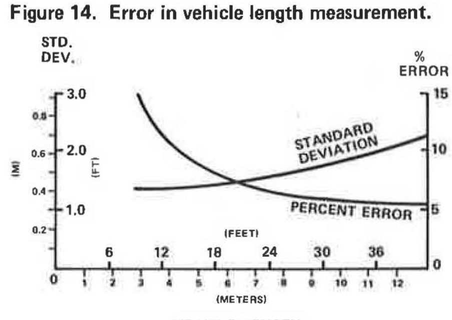


Figure 14. Error in vehicle length measurement.



Joseph Treiterer, Ohio State University

Any surveillance and control system can only be as good as the sensing system, and the most sophisticated computer will be useless if the input of traffic data is inadequate. I therefore consider this paper to be an important contribution to any surveillance and control system, i.e., existing ones that can be improved by applying the findings of the paper and new ones that can be developed by using the approach in this paper for an optimal design of the loop detector sensing system.

I very much agree with the conclusion and recommendation that an estimate of average speed should be made, and I would like to add that refining the process of speed estimation from a loop detector sensing system appears to be a promising approach for improving traffic conditions on urban freeways.

Measurements taken on Interstate 71 in Columbus, Ohio, show that maximum volume is obtained at a speed range of 38 to 64 km/h (30 to 40 mph). The same range, however, will only produce a throughput to 113 to 145 vehicle-km/h^2 (70 to 90 vehicle-miles/h^2), which is about 65 to 80 percent of the optimum traffic condition. That means that the productivity of urban freeways can be increased by 20 to 35 percent if the control system is based on proper speed control.

Furthermore, there is an important secondary aspect of reliable speed measurements. The most important phase in the process of recovering from a kinematic disturbance occurs at an almost constant traffic density of about 43 vehicles/km (70 vehicles/mile). If this recovery process is disturbed and cannot develop fully, then traffic flow conditions will recycle around the A-loop, thus resulting in a very inefficient stop-and-go operation. It is therefore of utmost importance that the recovery process be recognized and controlled properly to allow traffic to return to more efficient flow conditions. Because density is almost constant in this process, reliable speed determination data are the only computer input necessary for the control and dissipation of kinematic disturbances. In the recovery process speed increases steadily from about 32 to 64 km/h (20 to 40 mph) at about uniform density.

Attempts to obtain more accurate and reliable speed measurements for traffic surveillance lead to the multiple figure eight loop detector that was developed in a research project on the investigation of traffic dynamics by aerial photogrammetry techniques. Although the multiple figure eight detector, because it was designed to provide speed and density information, might have some advantages in comparison with the standard single loop detector, it is costly, the equipment is complicated, and the research carried out by Courage, Bauer, and Ross might very well cover the need for density and speed information in a much more economical and less complicated manner.

Authors' Closure

The authors appreciate both the complimentary and critical comments offered by the reviewers. These comments have been taken into account in making minor revisions to the paper and will be further considered in determining the future course of research on this subject.

Developments in Automatic Vehicle Identification During 1974 and 1975

Robert S. Foote, Port Authority of New York and New Jersey

Technology for automatically and uniquely identifying vehicles in motion has been under development and testing since the early 1960s and is receiving increasing attention. Systems using this technology make possible nonstop collection of tolls and other road user charges. Other potential applications include traffic control, law enforcement, and fleet management. This report summarizes recent developments of the technology. A coding format has been developed that is suitable for standard use. An intermediate generation of radio frequency equipment did not perform as expected, but a new generation is about to be tested. A system using microwaves is being developed and tested. An optical system is delivering good performance and reduced-rate cash toll collection. Studies of cost elements in a nonstop toll collection system, such as account maintenance, are being conducted. Market research is under way.

By the end of 1973 several reports had been published describing the growth since 1963 of technology for uniquely and automatically identifying vehicles in motion (1, 2, 3, 4, 5, 6, 7). Such a system would make possible fully automatic nonstop collection of tolls and other road user charges such as parking fees and assist traffic operations, vehicle security, law enforcement, vehicle maintenance, fleet management, motor vehicle administration, transportation planning, and other functions.

Since the end of 1973, there have been both progress and setbacks. Some of the major steps forward are as follows:

1. Initial testing by the Port Authority of New York and New Jersey of a new automatic vehicle identification (AVI) system,
2. Development of a new generation of low-power radio frequency AVI equipment,
3. Letting by the U.S. Department of Defense of a second-stage contract for a microwave AVI system,
4. Testing by the New Jersey Turnpike Authority of two additional AVI systems,
5. Testing by the Association of American Railroads of a new generation of AVI equipment, and
6. Use of an optical sticker in conjunction with coin or token collection on Delaware River Port Authority bridges.

On the other hand, AVI equipment tested at the Golden Gate Bridge and the bus terminal of the Port Authority of New York and New Jersey did not meet expectations. This equipment, whose design was later than that tested successfully by the Port Authority under contract for the U.S. Department of Transportation, failed because of breaks in a wire connecting a component to a printed circuit board, which were caused by different coefficients of thermal expansion. This made it impossible for Golden Gate Bridge officials to offer the AVI system to the general public, as had been planned. However, Golden Gate officials are now testing an AVI design developed to meet the Port Authority contract.

In summary, progress has been made. Interest in AVI systems for nonstop toll collection has continued, and the outlook is for field testing and refinement of several forms of AVI for toll collection in the immediate future.

Why should members of the toll road industry be particularly interested in AVI? By automatically and uniquely identifying moving vehicles, AVI, in conjunction with other system components that have already been proved, makes possible fully automatic collection of tolls, i.e., collection without requiring the transfer of currency and therefore without requiring any action on the part of either the motorist or the toll agency. This means toll collection without toll collectors and without toll plazas in its ultimate form. And that provides safety and convenience for patrons and lower costs and higher security for toll agencies. In addition, AVI can provide other benefits for road use and traffic control, vehicle administration, and road planning.

What is AVI? This technology was developed in the last 10 to 15 years and is now being applied in supermarkets, clothing stores, and other retail outlets as well as in military and transportation operations. Typically there are two elements involved: a sticker or other passive device carried on the vehicle or object to be identified and an active element mounted in or near the roadway and capable of reading electronically the identity of the device on the vehicle. Generally the vehicle-

Publication of this paper sponsored by Committee on Communications.

mounted device is referred to as a transponder; the road device is an interrogator. Depending on the type of system, information may be transferred between the transponder and the interrogator in the visible light spectrum, in the radio frequency spectrum, or in the microwave spectrum. Energy required for the transponder to emit its identity can be supplied by an internal power source such as batteries, by the electrical system of the host vehicle, or by reflectance or inductance from the interrogator power source.

How can this technology be used in a nonstop toll collection system? Because the only transfer at the time of vehicle passage is of electronic information, cash payment can be made either before or after the vehicle uses the toll facility. In either case, an account is maintained for each vehicle (or fleet operator) and is updated after each use and each payment. The amount of bookkeeping this entails is not feasible without computer technology. And, although such accounting is within the scope of today's computers, the cost may still be excessive in some toll applications. A key factor in reducing costs is to use standard transponders and interrogators and possibly a pooled accounting operation. With standardization, the motorist can use his or her transponder at the facilities of many toll and other agencies. And, with standardization, each toll operator can handle a larger proportion of toll collection through AVI. Through standardization, any interrogator can read any transponder, but the message format, coding, and AVI hardware must be standardized.

Who are the principal suppliers of AVI? At this point, only five companies are actively marketing AVI systems, and several others are moving toward the marketing stage with optical and microwave systems.

Where are AVI developments taking place? Both potential suppliers and potential users come from nearly all industrial countries. There has been significant development in France, the Netherlands, Great Britain, Germany, Canada, and Japan as well as the United States. The most significant field testing of radio frequency systems has been at the Golden Gate Bridge in San Francisco and by the Port Authority of New York and New Jersey. During 1972 and 1973 the Port Authority conducted a federally sponsored test (5) that confirmed that radio frequency AVI performs at the 98 percent plus level of accuracy needed for use in a fully automatic AVI-based toll system. Optical systems were tested briefly on the Quebec Autoroutes and by the New Jersey Highway Authority and are now being used in conjunction with a coin system at the Delaware River Port Authority bridges in Philadelphia.

When will AVI be in widespread use? The answer depends on many developments. However, it seems likely that the first fully automatic, nonstop AVI-based toll collection system will be offered to the public in the next 5 years. The Golden Gate Bridge had planned to offer such a system last year, but its introduction was delayed because of poor performance of the hardware. The New Jersey Turnpike may offer such a system on a test basis in the next year. And the Port Authority is working to implement an AVI system for buses in the next year. The rate of public acceptance depends first on the decision by toll agencies to offer such systems, second on the performance of the systems, and third on the actual costs and benefits, and it is increasingly likely that these factors will become well defined in the next 5 years.

PORT AUTHORITY TESTS

The Port Authority has extensively tested radio frequency AVI systems developed by four suppliers in a program supported by the Federal Highway Administration. By 1974, a system had been proposed for further testing. Special attention was given to the coding and message format to ensure that the system could serve as a standard for the toll industry.

The format adopted has an 80-bit transponder that can be coded in two main forms, one containing only fixed numbers (i.e., all bits in the transponder would be coded at time of manufacture and could not be altered subsequently without destroying the transponder) and one providing, in addition to some fixed numbers, five-digit number capacity that could be varied from on board the vehicle. Although the toll road industry is mainly interested in transponders that are coded permanently, some of the variable-unit systems may be particularly valuable for fleet operators. For example, buses could display route number and passenger loading to assist dispatchers to optimize fleet use.

The transponders and interrogators were delivered to the Port Authority in summer 1975. Loops for the interrogators had previously been installed in toll lanes 3 and 5 of the Lincoln Tunnel toll plaza in Weehawken, New Jersey. Also, cable had been run approximately 61 m (200 ft) to a room overlooking the toll plaza where the interrogator circuitry could be located. The functioning of both the interrogator and transponder units has been demonstrated satisfactorily. The Port Authority has tested most transponders to ensure proper operation, and initial units have been delivered to Transport of New Jersey, the bus company participating in the test program.

Information from both the new and old interrogators will be routed to a computer at the Lincoln Tunnel Administration Building. One of the main purposes of the test is to evaluate and demonstrate the feasibility of maintaining AVI accounts on-line.

Although a radio frequency system was chosen for this test, the Port Authority has not committed itself to this technology for the full system. Continuing interest is being maintained in alternate approaches, and additional testing may be scheduled.

NEW JERSEY TURNPIKE STUDIES

Early in 1974, the New Jersey Turnpike Authority decided to cease testing of AVIs, but, because of continuing inflation in the cost of conventional toll collection, the authority is still interested in new forms of automatic toll collection. After rethinking their requirements, turnpike staff solicited proposals for systems that would automatically identify patrons rather than vehicles. The status in September 1975 was summarized in a letter from Harry R. Loewengart, project engineer for the turnpike:

> About one year ago, we formulated a four year toll equipment modernization plan. This plan provides for development (where needed) and implementation of an overall system which includes, among other features, "permanent identification" of commuter vehicles (instead of single use toll tickets), a computer network and prepayment or charging of tolls by about 100 000 patrons. (Cars constitute 85 percent of our traffic.)
>
> . . . we recently solicited proposals for vehicle identification and detection equipment from 35 firms. From the six responses, we have selected two, . . . a microwave system and . . . an electro-optical system. A development and demonstration contract has been awarded [for] each; under these contracts both systems will be tested competitively in early 1976, with 100 participants (authority staff).
>
> Both systems provide for a transponder or label which can be hand held or attached to the vehicle. . . .

In addition to exploring hardware for this system, the turnpike staff is studying the costs of operating such an AVI system and also planning a survey of turnpike patrons to determine their interest in such new approaches to toll collection. The studies are still in progress but are developing important new data that will be essential in applying AVI technology in the toll road industry.

SYSTEM COST ANALYSIS

The cost of transponders is probably the most visible element in AVI system cost because the ultimate system will deal with millions of vehicles. By far the least expensive transponder is the optical sticker, which costs 10 cents to $8 depending on the required information content, reliability, and longevity. The cost of the radio frequency transponders is about $50, and this is a significant drawback to widespread application at first glance. However, the estimated life of these units, 15 years, along with the greater accuracy, reliability, and information capacity they offer, as compared with optical stickers, may make them competitive in price per use. The cost of microwave transponders has not yet been defined inasmuch as none of these units has yet reached the market stage, but present indications are that their cost would be between that of the optical and radio frequency units.

A preliminary analysis of total system costs was made by Port Authority staff in July 1974. Major capital cost elements include transponders, interrogators, local data recording, and central processing facilities. Tasks include installation and removal of transponders, preparation of monthly statements, and handling of accounts receivable and delinquent accounts. Many choices must be made in defining the exact system. For example, would transponders be purchased or leased by vehicle owners, or would both options be available? If leased, would a deposit be required? What agency would install or remove transponders? Who would pay? What information would be supplied on the periodic statements? Would this vary by class of user? Would all statements be issued monthly, or would variations of this period be desirable? What would be the procedure for issuing statements and receiving payment?

Answers to these questions will be developed in part through a greater understanding of the costs and benefits implicit in the choices and in part through marketing considerations. Much of the work under way to demonstrate a complete AVI-based toll system at the Port Authority is intended to define processing costs. The ball park estimate of total system costs is $10/year/user, of which roughly two-thirds is for the transponder and one-third for mailing and account processing. Other work to define AVI-based toll system costs is being undertaken by the New Jersey Turnpike Authority.

FURTHER AVI EQUIPMENT DEVELOPMENTS

The first routine operation of an AVI system by the public has been implemented by the Delaware River Port Authority, which uses an optical system (19). There are 14 lanes of equipment installed at the Walt Whitman Bridge, 13 lanes at the Ben Franklin Bridge, and 4 lanes at the Commodore Barry Bridge. The optical system is used in conjunction with automatic cash toll collection equipment. Stickers, mounted on the side windows of commuter vehicles, contain four digits: a classification number and three digits for an expiration code representing 30 days from the date of sale. If the date is valid, the patron passes through the lane after depositing a reduced cash toll in the automatic toll collection machine. At the Walt Whitman Bridge, all 14 lanes are equipped with a gate control. Bridge authorities report that they are pleased with the operation of the system.

In November 1974, a microwave AVI system developed under contract for the Army was completed. The microwave system uses a 7.6 by 11.4 by 0.6-cm (3 by 4.5 by 0.25-in) label or transponder affixed to the side of the container (or vehicle). The system is a line-of-sight operation, but, because the information is transmitted in the microwave band rather than the visible light band, the surface of the transponder can be made to appear as part of the vehicle to which it is affixed. As noted above, this system is to be tested by the New Jersey Turnpike Authority.

ROAD PRICING

Road pricing schemes differ from conventional toll road financing in which the primary aim is to recoup road construction and operation costs. Instead, the primary aim is to modify road use to promote public goals such as use of rapid transit. The widespread adoption of such schemes could create a significant market for AVI systems and stimulate the wide introduction of this technology. Thus the development of road pricing is of interest in AVI development as well.

The most significant progress in applying road pricing on a large scale appears to have been made in Singapore. A special charge, equivalent to $1.30, has been imposed on low-occupancy vehicles entering the central area during the morning peak hour. This scheme is being implemented by use of stickers rather than AVI. Peak-hour private car traffic has decreased to 25 percent of its previous level, and bus speeds have increased dramatically. The scheme is generally considered to be a success, and implementation during the evening peak is being considered.

In the United States, studies of road pricing are being made at the federal level, but prospects for implementation appear remote.

RAILROAD DEVELOPMENTS

As the major existing market for automatic identification technology, the railroad industry has been the primary factor in development of equipment suitable for AVI. The multicolored stickers identifying rail cars are ubiquitous, and optical scanners are in widespread use. However, the performance of this system has not met expectations. Identifications were being made only at a level of about 80 percent, even when the problem of dirty labels was attended to.

To evaluate the prospects for improved performance, the Southern Railway staff canvassed developments in identification technology in the United States and other countries and issued a comprehensive report in May 1974 (14). The report states in part:

> Substantial progress is evident in non-optical ACI development. This survey found prototype hardware whose performance exceeds that of the present optical system. All of these non-optical systems operate at wave lengths which penetrate label contaminants which cause the present system to fail.... The tests lead us to conclude that a microwave reflection system can meet crucial AAR specification requirements that are unobtainable by the optical system.... We believe a program could be completed in about 3½ years.

Responding in part to these recommendations, the Association of American Railroads is undertaking a search for a new generation of identification equipment. Progress in that effort will undoubtedly benefit highway applications as well.

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An Approach for Maximizing the Capacity of Self-Service Parking Facilities

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This paper develops a quick and effective method for determining the maximum storage capacity of a parcel of land designated for a self-service parking facility. Because of scattered and inadequate information on design guidelines, this study was authorized. The basic design unit in this analysis is the parking block, which comprises dual-stall units. The elements that constitute the block are analytically correlated. Two most widely used parking configurations are proposed for parking capacity analyses. The varying data on dimensional elements in a set of mathematical functions are entered into a developed computer program, and a series of derived graphs and application procedures that are necessary for solving a variety of parking capacity design problems is presented.

The growing use of automobiles and the decreasing availability of urban land have caused the scarcity of parking spaces. Trip generators in urban centers, which are moving toward higher and higher skyscrapers, attract more people per unit of land area than ever before, which results in higher parking needs per unit of land area. Street curb parking, which hinders traffic flow and consumes valuable driving area of street, is no longer considered feasible and desirable in areas of high traffic concentration. Hence, effective solutions—off-street facilities along with public transit—are urgently needed to accommodate the increasing parking demand in growing urban areas.

An intricate design for an off-street parking facility encompasses a complex process. To maximize the use of a facility, the design must be aimed at not only ensuring convenience and safety in and out of the facility but also keeping the cost low enough to attract users. In areas where land values are high, the problem of providing space for a parking facility becomes not only an economical one, but also one based on availability. If the parking land is obtainable, the facility should be designed to reduce the unit parking land cost. To this end, the facility designer's goal is to render the greatest efficiency of parking space use. However, specific guides to parking geometric design to maximize facility storage capacity are scattered and inadequate. Frequently conflicts arise in the design specifications when different sources are consulted. The designer, therefore, must use his or her judgment to arrive at the best solution. Obviously, this leaves room for judgment errors and thus signifies the need for standard design criteria for specific solutions in a variety of cases.

STUDY OBJECTIVES

As indicated previously, in consideration of parking facility layout, the use of parking space and the efficiency of parking operations are equally significant. Of primary concern to this study is the development of a solution that maximizes the parking capacity of self-service parking facilities and maintains a satisfactory operational efficiency. The rectangular parking facility was chosen as a design basis because it is most commonly used. This may appear to limit the usefulness of the solution obtained. However, parking facilities of many other shapes, particularly larger ones, can be divided into rectangular units so that the solution is still applicable.

BASIC GEOMETRIC DESIGN CONSIDERATIONS

Geometric design of the parking facility is the phase that requires designer creativity in making proper selection. Design elements such as stall width and length, aisle width, parking angle, and type of parking operation leave room for trade-offs to meet particular specifications.

The parking stall normally varies in width from 2.4 to 3.05 m (8 to 10 ft). The width should be such that it provides adequate space for the occupants to get in and out of the car from either side, which makes faster operation possible. Higher values should be used where drivers are likely to be inexperienced, and lower values should be used where attendant parking is available. Also, the width of the access aisles can be reduced as the stall width is increased. The stall length should be sufficient to accommodate the average car. A length of 5.5 to 6.1 m (18 to 20 ft) is dependent on conditions such as wheel stops and walls.

The access aisle must be wide enough to allow ease

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of maneuvering cars in and out of stalls. Aisle widths vary according to the angle of parking. Selecting widths for access aisles is of great importance in the design. In designating the aisle width, the designer must consider the width of the stalls, the efficiency of operation desired, specifications of the design vehicles, and designated parking angle.

Initially, the general parking configuration must be chosen. The choices are a single row of cars or a double row using the herringbone or interlocking pattern. Each has its advantages and disadvantages. The goal in the manipulation of all design elements is the most economical use of land area and efficient and safe operation of the facility.

NOTATION

The following notation is used in the analysis presented in this paper:

A = total area of lot,
a = area of block,
B = area per stall,
F_1 = number of stalls along one side of lot length,
F_2 = number of stalls along one side of lot width,
K = number of dual-stall units per block,
L = lot length,
n = total number of stalls per block,
S = number of blocks in lot,
t_1 = see Figures 1 and 2,
t_2 = see Figures 1 and 2,
U = see Figure 1,
W = lot width,
w = block width,
W/L = ratio of lot width to lot length,
x = stall length,
y = stall width,
z = aisle width,
α = parking angle, and
ℓ = block length.

Figure 1. Parking block geometrics.

$w = (2x + g)\sin\alpha$
$a = w \cdot \ell$
$n = 2(K + 1)$
$\ell = t_2 + U + K \cdot t_1$

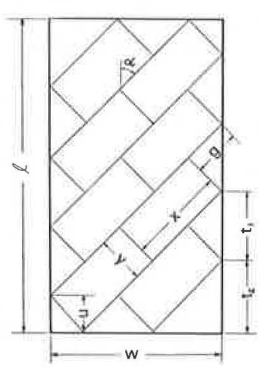


Figure 2. Parking configuration 1.

$L = S \cdot w + (S + 1)Z$
$W = \ell + 2Z$
$A = L \cdot W$
$B = A/N$
$R = W/L$
$N = S \cdot n$

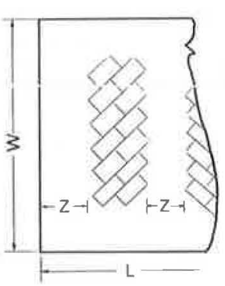
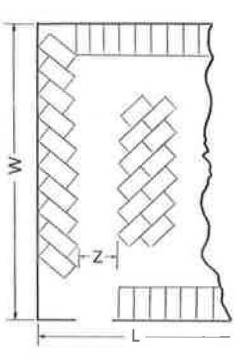


Figure 3. Parking configuration 2.

$L = S \cdot w + 2(x + g)\sin\alpha + Z(S + 1)$
$W = \ell + 2x + 48$
$A = W \cdot L$
$N = 2[F_1 + F_2 + S(K + 1)]$
$R = W/L$
$B = A/N$
$F_1 = [1 - 2(x + g)\sin\alpha - Z]/y$
$F_2 = [(w - x)/t_1] - 1$
$N = 2(F_1 + F_2)\ S \cdot n$

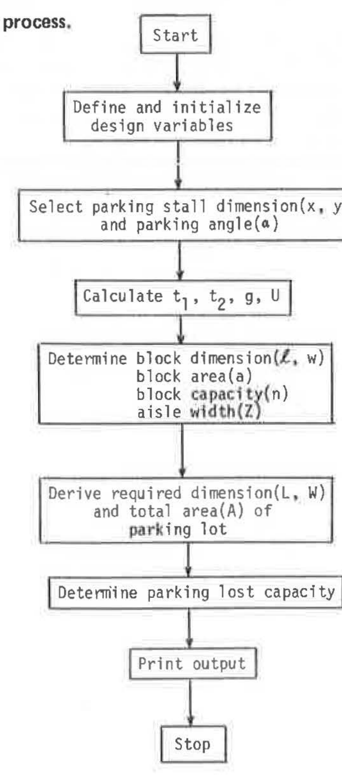


METHOD OF ANALYSIS

In this analysis, the parking block comprised of dual-stall units (Figure 1) is selected as the basic unit for capacity estimation. The geometric elements that constitute the block are analytically correlated. A two-part analysis of the relationship between parking angle and aisle width is made: one applied to a parking angle of 75 deg or less and the other for a 90-deg parking angle. The reason for separate analysis is that only one-way traffic is allowed for parking angles less than 75 deg, whereas a 90-deg parking angle can serve two-way traffic flow in the aisle. The mathematical relationships of the variables involved in the parking block are shown in Figure 1. To expand the equations for the total facility area requires that aisle width be taken into consideration. The computation can be made by using any, say, 15-deg increment of parking angles from 30 to 90 deg. The variation in angle parking and the corresponding changes in the aisle width are suggested in many published sources.

Two typical types of parking configurations (Figures 2 and 3) are proposed for this analysis. These configurations consist of parking rows in a herringbone parking

Figure 4. Computer process.



pattern. This specific parking pattern was selected because it is not only adaptable to changes in the angle of parking but also flexible in providing alternate traffic flow patterns in the parking facility. For the purpose of capacity analysis, various design parameters are correlated into a series of mathematical equations for the configurations shown in Figures 2 and 3. Based on those equations, a computer program has been developed to facilitate the calculations for specified data sets. A flow chart of the computer process is shown in Figure 4.

As shown in Figures 5 through 10, the graphical relationship between the total facility area and the required area per car is derived from changing the values of the block patterns and aisle widths for the selected herringbone configurations. The graphs shown in those figures can be used to obtain the maximum numbers of cars that can be stored in a given size of facility. The ratio of the facility width to its length should be known before

Figure 5. Parking configuration 1 for S = 2 blocks.

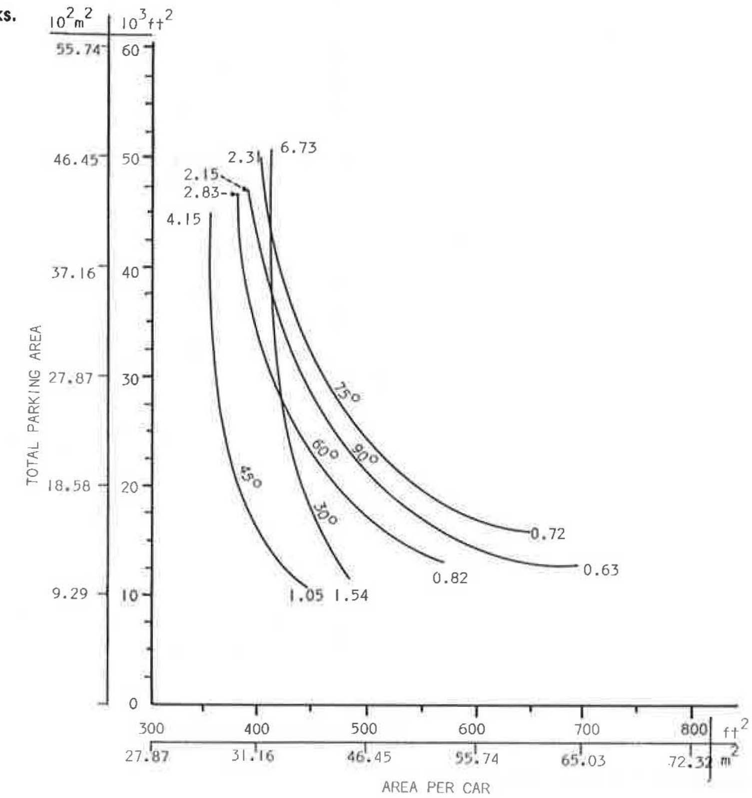
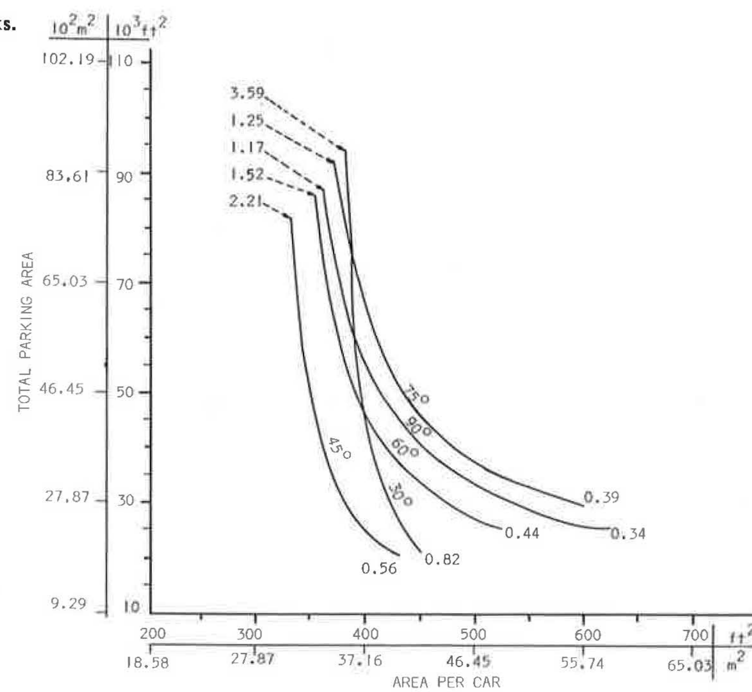


Figure 6. Parking configuration 1 for S = 4 blocks.



the optimum space use is obtained. The ratios of widths to lengths are labeled at the end points of the curves. To obtain the ratio for any point on the curve, a linear interpolation is used between the indicated ratios at the end points of each curve.

APPLICATION PROCEDURES

The graphs shown in Figures 5 through 10 can be used to quickly solve the geometric design problem of maximizing parking capacity of rectangular facilities. The following is a typical design example. Find the parking

Figure 7. Parking configuration 1 for S = 6 blocks.

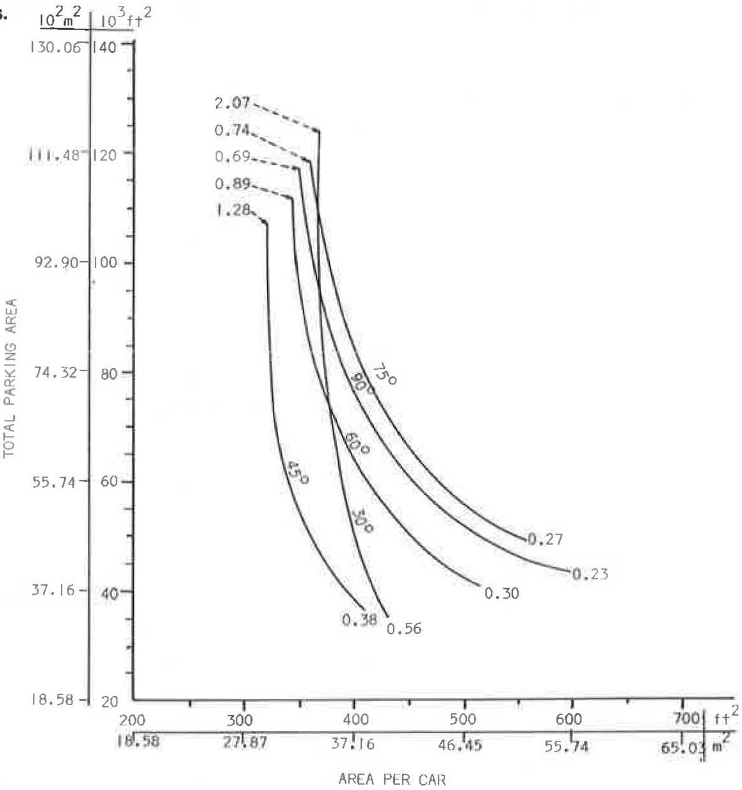


Figure 8. Parking configuration 2 for S = 2 blocks.

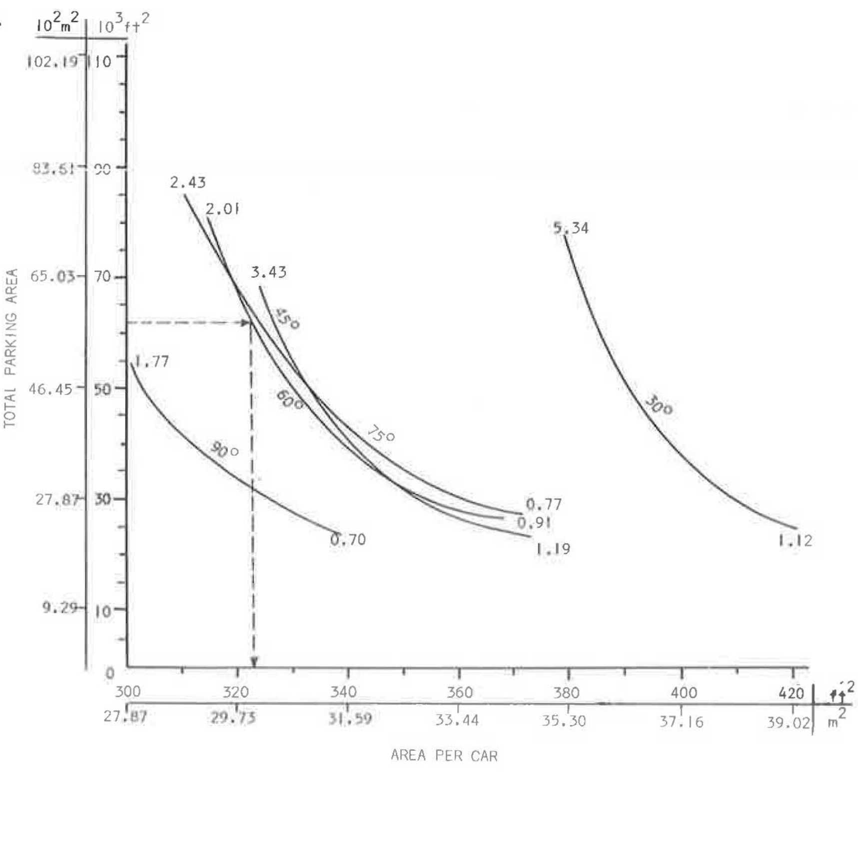


Figure 9. Parking configuration 2 for S = 4 blocks.

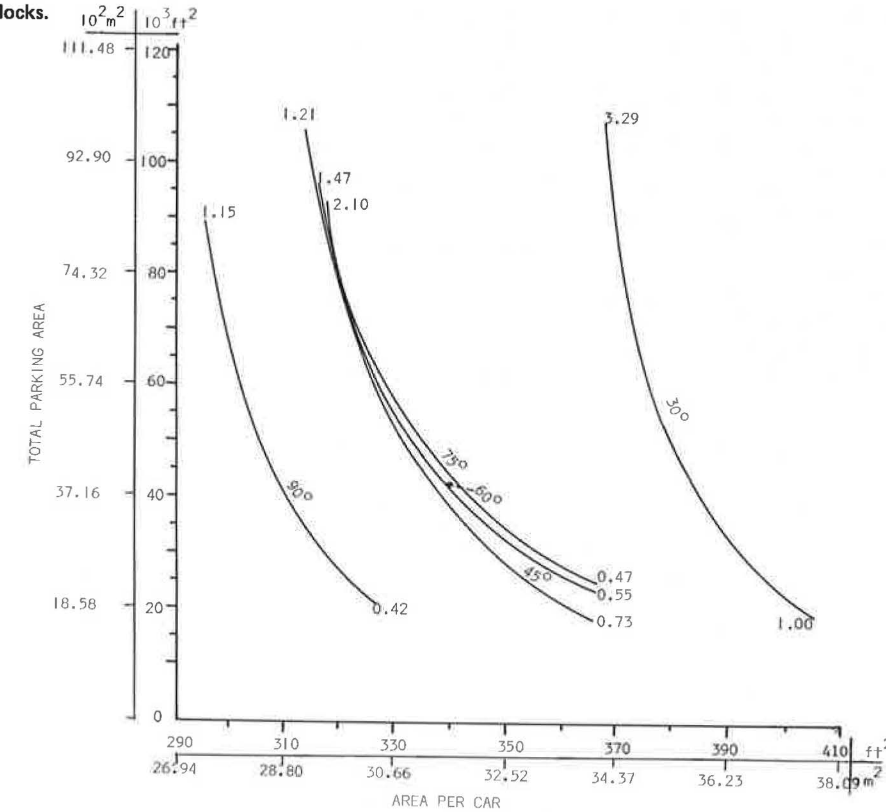
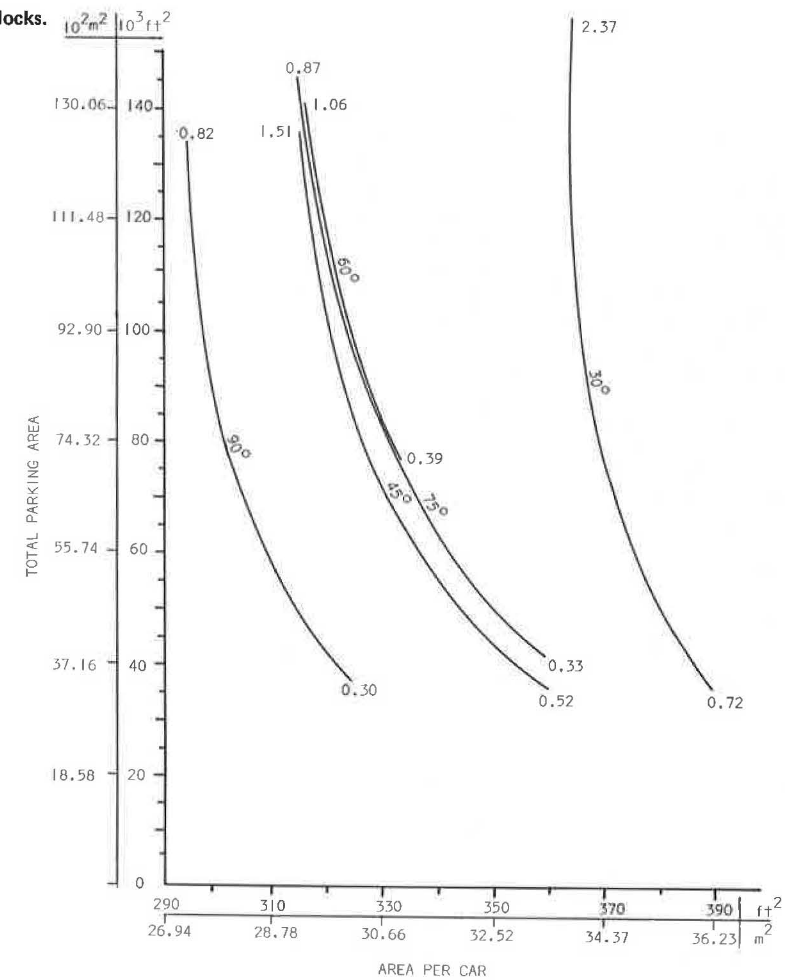


Figure 10. Parking configuration 2 for S = 6 blocks.



angle that requires the minimum area per car (or maximum parking capacity) for a facility with given total area and dimensions (facility width and length). Compute the value of width/length (W/L). Locate on the graphs (Figures 5 through 10) the respective facility area, and proceed to the right and find the lines intersected at a point where the value of W/L is approximately that computed. Several curves will probably contain the specified W/L value. It is possible to find more than one of these points. If more than one is close, choose the one that gives the least area per car value. When the minimum area per car is found, the intersected line will give the angle of parking for the maximum parking capacity for a given size and shape of land. This procedure for a specific design problem can be completed in a very short time period.

Determine the maximum parking capacity of a parcel of land having the dimensions of 53.3 by 106.7 m (175 by 350 ft).

First, determine the size of the area and parking lot width-length ratio; that is, $A = 53.3 \times 106.7 = 5687.11\ m^2$ ($61\ 250\ ft^2$) and W/L ratio = 106.7/53.3 = 2.00. The following data are obtained by using graphs:

Figure	Area per Car (m^2)	Parking Angle (deg)
6	37.16	30
8	29.91	60
9	30.35	45

The minimum area per car is $29.91\ m^2$ ($322\ ft^2$) for a parking angle of 60 deg, as given by Figure 8. The maximum number of cars N that can be parked is N = 5687.11/29.91 = 190 cars.

CONCLUSIONS

The ever-increasing number of automobiles has been creating a parking space shortage, particularly in high-density urban areas. Therefore, maximizing parking space use is a very important consideration in parking facility design. Although the geometric design problem is not new, the development of a better method for practical design purposes is still an urgent need for present and future parking facilities. The basic approach used to maximize storage capacity for any parking facility simply decides which type of configuration will prove most satisfactory for a specific parcel of land. However, because the design problem differs in each type of parking facility, the solution becomes very difficult to generalize. The approach undertaken in this study has developed a simple and practical tool for maximizing storage capacity of parking facilities, specifically for larger parking lots.

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Planning Transit Facility Parking for the Boston Metropolitan Area

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This paper analyzes the basic policy issues, planning parameters, and demand estimates associated with developing a regional park-and-ride plan for the Boston metropolitan area. It shows how transit facility parking can complement downtown parking supply, and it sets forth planning procedures for estimating the number and location of park-and-ride facilities. These methods have applicability in other large metropolitan areas.

Park-and-ride facilities are essential parts of regional transportation strategies that emphasize public transport, limit radial express highway construction, and stabilize downtown parking supply. An increasing number of communities look to transit facility parking as a means of achieving environmental, air quality, and energy conservation measures.

Establishing a transit-oriented parking policy calls for a realistic assessment of demands, costs, and consequences. It is necessary to know when transit facility parking is needed, where it should be located, and how it should be developed. This paper analyzes the basic policy issues and planning parameters associated with developing a park-and-ride system for the Boston metropolitan area.

TRANSIT FACILITY PARKING

As people move farther away from the city center, their tendency to drive increases. Transit facility parking can intercept those motorists and provide express transit for their line-haul trips. It is essential to maintain the existing suburban transit market, attract new transit riders, and reduce commuter car trips to the city center. Transit facility parking offers the following advantages.

1. It can reduce core-oriented automobile travel, and the attendant air pollution that this travel generates, by intercepting motorists in outlying areas and encouraging line-haul commuter trips by transit.

2. When transit facility parking is used, the automobile can be used for local collection and distribution to and from express transit stations in suburban areas where population densities are too low to generate walk-in patronage or to sustain local bus services. Parking is essential at all express transit stations outside of high-density areas.

3. Transit facility parking with secondary distribution by automobile (a) helps increase the public transport market, (b) reduces the extent of express and local transit routes, and (c) permits wider station spacings on express transit routes, thereby improving line-haul speeds and operating efficiency.

Transit facility parking is successful where the multimodal trip to the city center is cheaper and faster than driving. It has greatest applicability in urban areas where car travel to the city center is inhibited and where daily parking costs average $2.00 or more. Its feasibility depends on (a) demonstrated needs for additional downtown parking, (b) availability of express transit services, and (c) community willingness to limit downtown parking supply. These conditions prevail in the Boston metropolitan area.

Contemporary Practice

The role of transit facility parking in serving the downtown travel market is increasingly recognized. Today, outlying park-and-ride and kiss-and-ride facilities are provided in virtually every American metropolitan area with rail transit. Facilities range in size from about 200 spaces in Chicago to more than 1500 spaces in Boston, Cleveland, Philadelphia, New York, and Toronto. Parking is also provided along express bus routes in many urban areas including Milwaukee, New York, St. Louis, Seattle, Washington, D.C., and Hartford. Lots generally have at least 150 spaces, and 300-space facilities are common.

Three rapid transit systems—Cleveland, Lindenwold, and San Francisco (BART)—incorporate parking as an integral part of their overall operation. Approximately 7000 spaces are provided at nine stations in Cleveland, where daily inbound patronage approximates 30 000.

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Nearly 9000 spaces are provided at six stations along New Jersey's Lindenwold Line, where daily inbound patronage is approximately 18 000. BART initially provided about 17 000 spaces at 23 stations and is planning another 4000.

Typical ranges in the number of boarding passengers per parking space are given in Table 1. Parking patterns vary both among and within cities. There are usually two to three boarding passengers per parking space in suburban areas, but the ratio is much higher in densely developed areas, which rely on walk-in and bus riders. Studies of existing outlying change of mode parking facilities show about 1.2 daily person arrivals per parking space (1, 2, 3).

Kiss-and-ride patrons may represent 20 to 40 percent of total peak-hour arrivals. Median access distances of 4.8 to 6.4 km (3 to 4 miles) for park-and-ride passengers and 1.6 to 3.2 km (1 to 2 miles) for kiss-and-ride patrons are common.

Boston Park-and-Ride

There are 18 000 parking spaces along commuter rail and rapid transit lines in the Boston metropolitan area as compared with 40 000 spaces within the city center. These park-and-ride spaces are nearly 70 percent occupied by 9:30 a.m., and many have occupancies of 80 to 100 percent.

Boardings along Boston's Red Line extension to Quincy average about 3 passengers/parking space at North Quincy, 5 at Wollaston, and 15 at Quincy Center. Overall, the Blue Line averages about 10 boarding passengers per space; the Red, Blue, and Orange Lines combined average about 20 (downtown boardings excluded).

The wide range reflects variations in parking availability, bus frequency, and population density. Stations that are served by numerous bus lines and that have substantial walk-in traffic usually experience the highest patronage and least dependence on automobile arrivals. These stations are generally located along well-established transit lines, often near the city center. Conversely, stations in outlying areas with little or no bus traffic mainly rely on automobile arrivals. Thus, nearly 60 percent of all passengers arrive at Harvard Square by bus as compared with 16 percent at stations in Quincy, and fewer than 10 percent access the Penn Central and Riverside stations along Route 128 by bus.

POLICY OPTIONS AND TRADE-OFFS

The total parking supply for downtown-oriented commuters includes two complementary and interdependent components: (a) parking within the city center and its environs and (b) parking along express transit lines. There is generally a trade-off between the amount of parking provided downtown and that provided along express transit lines for any specified level of downtown activity. An increase in one suggests a decrease in the other.

The optimum balance between the two types of parking is a basic policy choice facing many large urban centers. The challenge is usually where to provide additional parking, rather than whether additional parking should be provided. Viewed in this context, transit facility parking represents a transference of parking supply from the city center outward along express transit lines.

The basic policy options are

1. Alleviate existing deficiencies, meet CBD growth, and hold modal split constant;
2. Accommodate existing needs, CBD growth, and automobile use trends; or
3. Maintain downtown parking freeze.

Based on the complementary relationship between parking downtown and along express bus lines, the basic policy options would mean the following number of additional spaces for the Boston metropolitan area:

Option	CBD Spaces	Outlying Spaces	Total
1	10 520	4 000	14 520
2	14 520	0	14 520
3	0	14 520	14 520

The 1980 needs for both downtown and outlying parking spaces in the Boston metropolitan area are shown in Figure 1 and given in Table 2. Stabilizing downtown parking supply would result in about 14 500 additional effective spaces along outlying transit lines by 1980. Maintaining the existing modal distribution of CBD travelers would result in about 4000 additional transit facility parking spaces.

DEMAND ESTIMATES

Future demands for transit facility parking depend on the magnitude of downtown growth, existing and future deficiencies in downtown parking supply, origins and travel modes of downtown employees, amount and location of regional population growth, and basic changes in roads, transit services, and downtown parking supply.

The amount of potential transit facility parking demand depends on the increase in CBD employment (direct variation), increase in CBD parking supply (inverse variation), and extension of express transit services (generally direct variation). The location of demand depends on the geographic distribution of future CBD employment, especially the anticipated increment of employment growth, and the relation of this additional CBD employment growth to transit service extensions and street patterns.

Transit facility parking demand forecasts for the Boston metropolitan area show how the preceding approaches can be applied. Anticipated 1980 transit facility parking demands were based on the population and employment projections furnished by the Boston Transportation Planning Review (BTPR) and the Boston Redevelopment Authority (BRA). They were developed by analyzing the station arrival patterns of the additional downtown employment growth assuming that (a) a downtown parking freeze would continue, (b) planned rapid transit extensions would be open, and (c) commuter rail service frequency would be improved or at least remain at present levels. Demands were cross-checked by estimating parking requirements directly from boarding passenger-parking ratios.

Number of Spaces

The number of anticipated 1980 parking spaces for downtown Boston-oriented transit riders was based on the following assumptions:

1. An increase of approximately 15 percent in CBD person accumulations and employment between 1972 and 1980,
2. A stabilization of downtown off-street parking supply at 1972 levels, and
3. Transit service extensions of the Green Line to Somerville, the Orange Line to Malden and Needham, and the Red Line to Alewife (or beyond) and South Braintree.

To stabilize automobile use within the city center at 1972-1980 levels requires that the entire additional increment of employment growth commute by transit. This assumption provides a reasonable approximation of the potential increase in transit riders. It was further assumed that one transit parking space would be provided for every two additional boarding transit passengers, overall.

Parking demands of transit riders destined for downtown Boston resulting from these assumptions are given in Table 3. Approximately 16 000 additional effective spaces would be needed. This translates into an additional 18 000 total spaces when allowance is made for the efficiency of space use.

Distribution of Spaces

Anticipated 1980 transit facility parking demands were allocated to the Boston metropolitan area rings and sectors. It was assumed that each community would attract additional CBD workers relative to its population change and in general proportion to its present CBD work-trip attraction rates. These trip attraction rates were based on interviews conducted at major generators in downtown Boston during 1972 and show a decreasing downtown orientation with distance from the city center (Figure 2).

Anticipated 1980 transit facility parking demand computations are given in Table 4. This table is described below.

1. Population in the 150-town Eastern Massachusetts Regional Planning Project area, based on BTPR estimates, is expected to increase from 3.8 million in 1972 to 4.2 million in 1980. The anticipated population changes were identified by ring and sector (columns 1, 2, and 3).
2. The 1972 per capita work-trip attraction rates were applied to the anticipated population increases in rings 1, 2, and 3. However, for ring 4, CBD trip attraction rates of four per 100 persons in sectors 1, 2, and 3 and six per 100 persons in sectors 4, 5, and 6 were used (columns 4 and 5).
3. The resulting CBD employment increases were normalized to reflect 39 000 for the strong core projection (column 6).
4. Increases in employment were then translated into parking space needs. The 1972 modal distributions of workers to the city center by ring and sector were used to approximate the relative proportion of 1980 car drivers to transit stations (columns 7 and 8). This provided a more realistic approach than direct translation by means of a constant percentage.
5. The resulting parking demands were then normalized to the basic control total of 18 000 total parking spaces for the strong core projection (column 9).

This procedure can be expressed analytically as follows:

$$e_i = \frac{39\,000\, r_i p_i}{\sum (p_i r_i)} \tag{1}$$

$$t_i = \frac{18\,000\, Z_i e_i}{\sum (Z_i e_i)} \tag{2}$$

where

e_i = increase in employment in area i,
r_i = attraction rate of CBD work trips from area i,
p_i = population growth in area i,
Z_i = assumed percentage of work trips by automobile driver in area i, and
t_i = potential transit facility parking spaces in area i.

Transit facility parking demands were also derived from Massachusetts Bay Transportation Authority (MBTA) forecasts of inbound passengers. Estimates of boarding passengers per parking space based on current experience at suburban and urban transit stations were applied to anticipated station patronage along proposed extensions. These estimates (Table 5) were generally consistent with the previously developed demands. They produced considerably greater demands in the northwest corridor, which may include parkers destined to central Cambridge. They did not identify increases in demands in sectors where no additional rail transit is anticipated, inasmuch as the forecasting methodology tended to distribute these demands to adjacent corridors.

Composite 1980 Demands

Anticipated 1980 transit parking demands by ring and corridor are given in Table 6 and shown in Figure 3. Although based on the previous computational steps, these demand forecasts were modified to more closely reflect the results of the alternate forecasts.

Potentials of any given site or station will depend on locations of roads, transit lines, and parking facilities. Therefore, accumulative demands by ring are also shown since potential parkers can be expected to travel inward toward the city center to reach parking locations and logical intercept points.

The demand estimates reflect anticipated rates of downtown employment, growth, and metropolitan population change over the next decade. If growth occurs at a slower rate, then the estimates should be reduced accordingly or the horizon year should be extended. Thus, the estimates provide a framework and methodology for adjusting park-and-ride forecasts to varying population and employment changes in the city center and throughout the metropolitan region.

PLAN DEVELOPMENT

Transit facility parking should intercept motorists at threshold points between suburbs and downtown where it is efficient to transfer from car to transit. Ideally, such parking facilities should be located where land is relatively inexpensive, environmental impacts are minimal, and the rest of the journey by car is congested. The multimodal trip from the city center should be faster and less costly than the corresponding trip by car. Time savings should exceed 5 min to overcome passenger reluctance to change modes. Meeting these broad criteria suggests that transit facility parking generally should be located 8 to 13 km (5 to 8 miles) from the city center. These optimum distances will vary among communities, depending on specific local street, transit, and land use patterns.

Concept Plan

The emergent transit facility parking plan shown in Figure 4 provides a conceptual framework for detailed site development. It depicts the logical transit-highway intercept points for consideration as possible park-and-ride sites. The plan includes

1. Existing outlying parking facilities along principal MBTA routes;
2. Additional large parking facilities of 500 to 2000 spaces located along rail rapid transit extensions, mainly in suburban communities, and clustered along the Route 128 axis;

Table 1. Transit facility parking patronage comparisons in major metropolitan areas, 1970 to 1972.

Location	Estimated Boarding Passengers	Off-Street Parking Spaces	Boarding Passengers per Parking Space
Boston			
Wollaston	2 700	500	5.4
North Quincy	2 400	800	3.0
Quincy Center	7 500	930	8.1
B&M	11 000	3 360	3.3
Penn Central	3 800	2 640	1.4
Chicago			
Dempster	4 000	500	8.0
Desplaines	4 000	500	8.0
Cleveland			
West Side (Brookpart, Puritas, Triskett, W. 117th, W. 98th)	20 000	6,400	3.1
East Side (E. 55th, Superior, Windermere)	10 000	900	11.1
Philadelphia (commuter rail)			
Bucks County	4 000	1 800	2.2
Chester County	3 900	1 100	3.5
Delaware County	15 500	2 200	7.0
Montgomery County	19 500	4 300	4.5
Penn Central, in city	45 000	2 100	21.4
Reading, in city	31 600	2 700	14.6
Lindenwold (New Jersey)	20 000	9 000	2.2
Toronto			
Islington	23 500	1 300	18.0
Warden	24 600	1 500	16.4

3. Additional rail transit-oriented facilities of 300 to 500 spaces located along main commuter rail lines; and

4. New bus-oriented parking facilities of 300 to 500 located along Route 128 at (a) interchanges with major radial express highways and (b) within regional shopping centers (they would be served by express buses, in conjunction with priority use of radial expressways).

Table 2. Summary of 1980 parking space needs for Boston metropolitan area.

Item	1972	1980	Net Increase
Legal supply	38 500[a]	41 500[b]	2 900
Effective supply	33 020	35 480	2 460
Peak demand	39 650	50 000	10 350[c]
Need	6 630	14 520	7 890

[a] About 5600 curb and 32 900 off-street spaces.
[b] Spaces under construction or built since 1972.
[c] Based on following breakdown: 6350, CBD growth (policy option 1), and 4000, modal split change (options 1 and 2).

Figure 1. Anticipated 1980 downtown-oriented parking demands in relation to parking policy options.

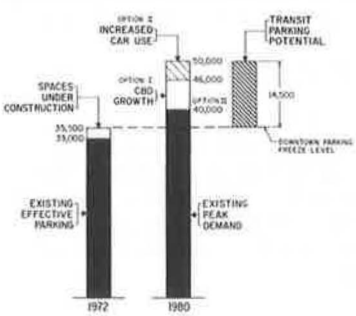
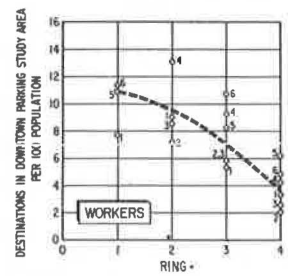


Table 3. Estimated 1980 transit-related potentials for downtown Boston (increase over present levels).

Item	Effective Additional Spaces for CBD Travelers	Remarks
Transferred 1980 CBD parking demand	14 520	Includes illegal parking and walk-in demands
15 percent increase in peak CBD person accumulation (30 000 persons)	15 000	60 percent automobile driver arrivals at stations; 1.2 occupancy-turnover adjustment
CBD employment growth of 39 000 persons	19 500	60 percent automobile driver arrivals; 1.2 occupancy-turnover adjustment
Average	16 340	

Figure 2. Distribution of worker and nonworker trip attractions in downtown Boston.



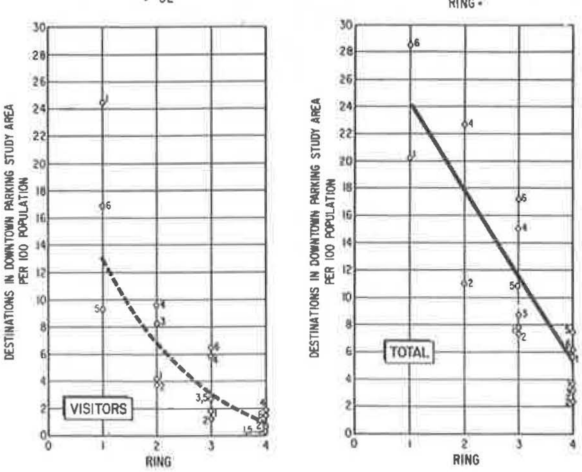


Figure 3. Anticipated 1980 additional transit parking demands in downtown Boston (18 000 actual spaces, CBD demand only).

Table 4. Anticipated 1980 transit facility parking demands.

Corridor of Origin (1)	Ring (2)	Change in Population (3)	Assumed Downtown Boston Employment (%) (4)	Daily CBD Employment Increase (5)	Daily Resultant Employment (6)	Percent as Car Driver to Transit Station (1972 Percent by Car to CBD) (7)	Daily Automobiles per Transit Facility Parking Space (8)	Parking Spaces (9)
1	1	2 800	7.6	213	291	7.6	22	24
	2	3 000	9.1	273	374	27.1	101	110
	3	28 200	5.6	1 579	2 161	40.6	877	959
	4	26 900	4.0	1 076	1 472	49.1	723	791
Subtotal		60 900		3 141	1 298		1 723	1 884
2	2	4 700	7.5	353	483	35.4	171	187
	3	14 900	5.9	879	1 203	31.4	378	413
	4	114 900	4.0	4 596	6 290	51.9	3 265	3 572
Subtotal		134 500		5 828	7 976		3 814	4 172
3	2	2 600	8.7	226	309	6.9	21	23
	3	14 000	5.8	812	1 111	31.8	353	386
	4	27 800	4.0	1 112	1 522	56.2	855	935
Subtotal		44 400		2 150	2 942		1 229	1 344
4	2	1 700	13.5	230	315	36.8	116	127
	3	8 900	8.9	792	1 084	35.6	386	422
	4	52 300	6.0	3 138	4 294	53.9	2 314	2 531
Subtotal		62 900		4 160	5 693		2 816	3 080
5	1	35 200	10.9	3 837	5 251	16.8	882	965
	3	1 100	7.7	85	118	28.8	34	37
	4	42 100	6.0	2 484	3 399	50.9	1 730	1 892
Subtotal		78 400	9.1	6 406	8 768		2 646	2 894
6	1	8 000	12.0	960	1 303	25.4	331	362
	3	19 000	10.5	1 995	2 756	40.2	1 108	1 212
	4	64 100	6.0	3 846	5 264	53.0	2 790	3 052
Subtotal		91 100		6 801	9 323		4 229	4 626
Total		472 200		28 486	39 000		16 457	18 000

Table 5. Additional outlying parking potentials based on 1975 estimated passenger volumes.

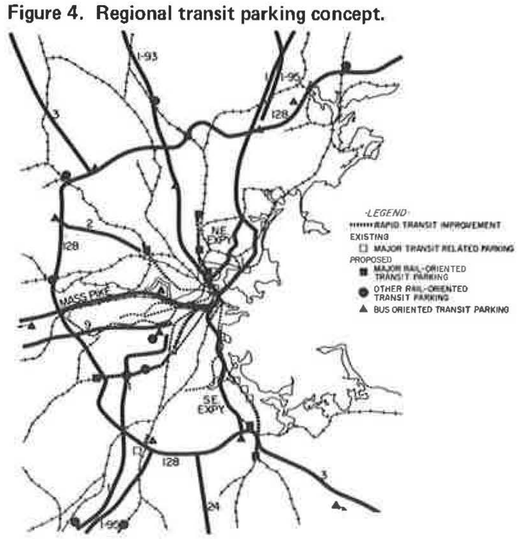
Corridor	24-Hour Boarding Passengers	Boarding Passengers Per Parking Space	Estimated Parking Spaces[a]
Red Line Southeast			
South Braintree	5 500	2	2 000
North Braintree	3 830	2	1 900
Orange Lines			
VFW (and Route 128)	7 530	2	2 000
West Roxbury	5 570	8	700
Roslindale Center	6 700	8	800
Green Line Northwest			
Washington	7 140	10	700
Red Line Northwest			
Alewife	17 100	4	2 000
Orange Line North			
Oak Grove	5 000	4	1 200
Malden Center	6 500	8	800
Wellington	5 700	8	700
Sullivan Square	7 000	10	700
Community College	11 370	10	1 100
Total			14 600

[a]Maximum capacity of 2000 spaces at any location.

Table 6. Anticipated 1980 transit facility parking demands in downtown Boston (increase over present levels).

	Ring				
Corridor	4	3	2	1	Total
1	790	950	140	0	1 880
2	3 070	410	190	–	3 670
3	1 430	420	0	–	1 850
4	2 530	420	130	–	3 080
5	1 720	240	–	960	2 920
6	3 540	700	–	360	4 600
Total	13 080	3140	460	1320	18 000

Figure 4. Regional transit parking concept.



Plan Implementation

It is necessary to continually adapt this regional concept plan to physical, economic, and environmental reality. This calls for detailed analysis of costs, benefits, and impacts, environmental impact assessments of alternative site developments, and cooperative implementation by state and regional transportation agencies and impacted communities.

The locations where park-and-ride facilities physically and environmentally can be provided will influence the locations and capacities of the transit facility parking system. They will have important bearing not only on the parking program but on the feasibility of express transit extensions as well.

CONCLUSIONS

This paper has set forth an approach to formulating and quantifying a regional transit facility parking plan. The methodology has potential application in other large metropolitan areas as well. It represents a first step before detailed demand estimates, plans, and designs for specific park-and-ride sites are developed.

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Framework for Design and Operation of Passing Zones on Two-Lane Highways

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Current design and marking standards for passing and no-passing zones are based on the results of field studies conducted more than 35 years ago. Many of the assumptions used to derive these standards are not valid for current highway operations. In addition, design and marking standards are almost exclusively concerned with passing sight distance. Design standards contain no provision to establish the minimum length over which the design passing sight distance must be made available to the passing driver to constitute a safe passing zone. Marking standards indirectly set the minimum length of passing zones at 122 m (400 ft). This zone length is inadequate for the majority of higher speed passes. A review of recent research, especially studies dealing directly with design or marking practice, indicates that sufficient data are available to develop design and marking standards based on contemporary field measurements. The results of a study on short passing zones demonstrate the safety and operational deficiencies of passing zones less than 268 m (880 ft) long. Therefore, it is recommended that design and marking standards consider both passing sight distance and passing zone length. Because the design of highway geometrics should be based on analysis of subsequent highway operations, the authors recommend that design and marking standards be identical. Specific criteria for the design and marking of passing and no-passing zones are suggested.

In the age of interactive graphics, automatic photogrammetric plotting, freeway surveillance and control, real-time motorist communications, and other sophisticated design and operational tools, the highway community still designs and marks passing zones on two-lane rural highways according to false and archaic principles.

The passing maneuver is a very complex phenomenon because it involves not only the dynamic time-space relationship of three moving elements (passing vehicle, passed vehicle, and oncoming vehicle) but also the dynamic perceptual reference (sight distance) of the passing driver. The total phenomenon of the interrelationships of moving vehicles and changing sight distance, therefore, defines visualization in the nominal time-space reference frame.

Although the passing phenomenon is very complex, this is little justification for the false concepts used in the design and marking of passing zones. Actually in the current practice (1, 20), passing zones are neither designed nor marked directly. Current marking practices, for example, are directly concerned with no-passing zones, and passing zones merely happen where no-passing zones are not warranted. In highway design, the design of passing sight distance (PSD) only considers the percentage of highway distance that has PSD, regardless of whether that PSD constitutes passing zones of adequate length. In other words, PSD design ignores the number, location, and length of passing zones. Also, PSD design practice is based on different criteria and, therefore, bears little resemblance to the intended operational (marking) practice.

Besides the inconsistencies already discussed, there are flaws in the hypothesis underlying current design and marking practices. Although this hypothesis considers two elements of the critical passing situation, it throws out the baby with the bath water. The hypothesis correctly looks at an opposing vehicle as an integral component of the critical maneuver and correctly considers a minimum safe separation distance between the passing and opposing vehicles at the completion of the pass. What it fails to recognize is the critical time-space relationship among all three vehicles involved and how that relationship bears on the determination of required sight distances and lengths of passing zones.

CURRENT DESIGN PRACTICES

Current design standards set forth in the AASHTO policy (1) are based on the results of field studies (17, 18) conducted between 1938 and 1941 and validated by another study (16) conducted in 1957. Based on these studies, the AASHTO policy defines the minimum passing sight distance as the sum of the following four distances:

d_1 = distance traveled during perception and reaction time and during initial acceleration to the point of encroachment on the left lane,

d_2 = distance traveled while the passing vehicle occupies the left lane,

d_3 = distance between passing vehicle and opposing vehicle at the end of the passing maneuver, and

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d_4 = distance traveled by an opposing vehicle for two-thirds of the time the passing vehicle occupies the left lane or two-thirds of d_2.

Design values for each of these four distances were developed by using the previously mentioned field data and the following assumptions.

1. The overtaken vehicle travels at uniform speed.
2. The passing vehicle reduces speed and trails the overtaken vehicle as it enters a passing section.
3. When the passing section is reached, the passing driver requires a short period of time to perceive the clear passing section and to react to start his maneuver.
4. Passing is accomplished under what may be termed a delayed start and a hurried return in the face of opposing traffic. The passing vehicle accelerates during the maneuver, and its average speed during the occupancy of the left lane is 16.1 km/h (10 mph) higher than that of the overtaken vehicle.
5. When the passing vehicle returns to its lane, there is a suitable clearance length between it and an oncoming vehicle in the other lane.

The design standards obtained when the four elements of the passing maneuver are combined under these assumptions are shown below. Sight distance on a vertical or horizontal curve is defined as the distance at which an opposing vehicle 1.37 m (4.50 ft) above the pavement surface can just be seen by a passing driver 1.14 m (3.75 ft) above the pavement.

Design Speed (km/h)	Minimum Passing Sight Distance (m)	Design Speed (km/h)	Minimum Passing Sight Distance (m)
48	335	105	701
64	457	113	762
80	549	121	792
97	640	129	823

An examination of the state of the art indicated that many of the assumptions used to develop the AASHTO standards are invalid for current highway operations (22). First, no provision exists for establishing the length over which the design PSD must be made available to the passing driver. Second, assumed speeds considerably lower than design speeds are used because the standards are based on an average rather than a critical passing maneuver. And, third, the assumed speed differential between passing and passed vehicles measured for low-speed passes was extrapolated as a constant for higher speed passes. These assumptions lead to inadequate sight distance standards except that the elements of the passing maneuver are combined in an extremely conservative manner. The summation of the four distance elements d_1, d_2, d_3, and d_4 bases passing sight distance requirements on the completion of an entire passing maneuver rather than on the critical portion of the maneuver after the driver is committed to pass. More realistic standards can be developed based on current measurements of passing performance and a revised model of the sight distance requirements of passing drivers.

CURRENT MARKING PRACTICE FOR NO-PASSING ZONES

The 1971 Manual on Uniform Traffic Control Devices (MUTCD) specifies that a vertical or horizontal curve shall warrant a no-passing zone and shall be so marked where the sight distance is equal to or less than that listed below for the prevailing (off-peak) 85th percentile speed. Sight distance on a vertical or horizontal curve is defined as the distance at which an opposing vehicle 1.14 m (3.75 ft) above the pavement surface can just be seen by a passing driver 1.14 m (3.75 ft) above the pavement.

85th Percentile Speed (km/h)	1971 MUTCD Sight Distance (m)	85th Percentile Speed (km/h)	1971 MUTCD Sight Distance (m)
48	152	97	305
64	183	113	366
80	244		

The reasons for selecting these minimum sight distances are not stated in the MUTCD, nor is the source given. However, MUTCD sight distances are identical to those presented in a 1940 AASHTO publication (2). These recommended sight distances represent a subjective compromise between distances computed for flying passes and distances computed for delayed passes. As such, they do not represent any particular passing situation. Table 1 gives the basic assumptions and derived sight distances.

Because of the lack of documentation, the source of flying pass sight distances cannot be validated. The delayed pass sight distances are calculated as the sum of the left-lane distance and the opposing vehicle distance based on the stated assumptions and additional assumptions regarding the passing phenomenon contained in another AASHTO policy (3). For these calculations, the perception-reaction distance, d_1, and clearance distance, d_3, are ignored, and the opposing vehicle is assumed to appear at the initiation of the passing maneuver.

With all the subjective assumptions and manipulations needed to derive the 1940 AASHTO no-passing zone sight distances, it is very difficult to see how they relate in any way to the safety and operational efficiency of passing zones.

In regard to MUTCD recommendations, the other part of the no-passing zone marking practice relates to the distance between no-passing zones, which indirectly sets the minimum length of passing zones. The basis for this length, which is set at 122 m (400 ft), is not documented. A passing zone of this length is wholly inadequate for the majority of higher speed passes on main rural highways (13).

A final consideration in current marking practice is that there is no universally accepted meaning of centerline markings. Two concepts are in use today. The first, known as the short-zone concept, requires that all passes be completed within the marked passing zone. This concept has been adopted by the Uniform Vehicle Code (15) and incorporated by reference in the MUTCD. A less common interpretation, known as the long-zone concept, allows a driver to complete a pass within a marked no-passing zone if the pass was started within a passing zone. Criteria for marking passing and no-passing zones are based on the meaning of centerline markings. Thus, passing zones based on the long-zone concept can be shorter, since passes need not (legally) be completed within the zone. However, longer passing sight distance should be required at the end of a passing zone, inasmuch as a pass could legally be initiated at that point. Either concept can be used, together with appropriate marking warrants, to operate highways safely. However, uniform nationwide use of a single interpretation of passing and no-passing zone markings is desirable.

RECENT RESEARCH FINDINGS

Although a considerable number of studies have been conducted on various aspects of the passing maneuver, the fact remains that current AASHTO and MUTCD standards are primarily based on studies conducted 35 to 50 years ago. For this reason alone, the validity of current standards is questionable.

Many recent studies have provided valuable insights that would allow improvements to these standards, but several of these studies were purely empirical and gave little attention to application of the results to current practice. These results should be synthesized to make them applicable.

In the interest of space, a thorough discussion of all pertinent aspects of all previous studies is impractical, but several of the more important ones are discussed briefly and a few significant studies are discussed in more detail.

Several studies were concerned with the driver's ability to estimate variables such as available sight distance, closure speed between the passing vehicle and the passed or opposing vehicle, required passing distance or time under various impedance conditions (either by an approaching vehicle or by available sight distance), and other judgment aspects of the passing maneuver. One study (4) was conducted to determine how drivers understand and act at no-passing zones. Another study (5) involved mathematical simulation of a two-lane rural highway.

Research conducted by Gordon and Mast (10) was concerned with the ability of drivers to judge the distance required to overtake and pass. Their results (government car and own car) are shown in Figure 1 compared with previous results by Matson and Forbes (14), Prisk (18), and Crawford (6). Although none of these researchers was concerned with passing zone length, the best fit curves clearly indicate the inadequacy of the MUTCD 122-m (400-ft) passing zone.

Jones and Heimstra (12) performed studies to determine how accurately drivers estimate clearance time. They found that many subjects were not capable of accurately judging the last safe moment for passing without causing the approaching vehicle to take evasive action.

Farber and Silver (7, 8, 9, 19) defined requirements for the overtaking and passing maneuver. The major findings of their studies were that drivers judged distance accurately in passing situations, but that their ability to judge speed variables was marginal. Subjects could not discriminate even grossly different opposing vehicle speeds. Ability to judge time available to pass was substantially improved when the need to judge opposing vehicle speed was eliminated.

Research was conducted by Hostetter and Seguin (11) to determine the singular and combined effects of impedance distance, impedance speed, passing sight distance, and traffic volume on driver acceptance of passing opportunities. In general, sight distance was found to be the major determinant of the probability that a driver would accept a passing opportunity. The probability of a pass increased as the sight distance increased.

Bacon, Breuning, and Sim (4) conducted a questionnaire study of passing practices and no-passing policies to determine how drivers understand and act at no-passing zones. The research revealed that only 30 percent of the sample (424 respondents) claimed to observe no-passing zones according to enforcement intentions.

Cassel and Janoff (5) used a mathematical simulation model to study passing maneuvers. It simulated the movement of vehicular traffic for various road geometry and traffic volume conditions. Results of simulation runs indicate that (a) when drivers were given knowledge of opposing vehicle speed on tangents, there appeared to be an increase in safety but the average speed was reduced, so that a significant loss in time occurred; and (b) as the percentage of no-passing zones increased, there was a decrease in throughput as indicated by average speed, time delay, and number of passes.

Two 1971 studies (21, 23) added significant insight on how to consider the passing phenomenon in designing and marking adequate passing zones. These studies independently recognized the critical position in the passing maneuver where the passing driver requires the maximum sight distance for safe execution of the maneuver. Weaver and Glennon (23) called this the critical position, and Van Valkenburg and Michael (21) called it the point of no return. Although it is difficult to determine the exact relationship between the passing and passed vehicles, this position occurs essentially when the two vehicles are abreast. It is the point at which the sight distances needed either to safely (with adequate clearance to an opposing vehicle) complete the pass or to abort the pass are equal.

Figure 2 shows a critical passing maneuver. The passing and passed vehicles are at points A and B as they enter the passing zone. The passing vehicle travels distance d_1 while determining that the left lane is clear to pass. At point C, the passing vehicle is abreast of the passed vehicle. The results of early studies, reconfirmed by Weaver and Glennon, show that the average distance traveled by a passing vehicle in accelerating from the trailing position to the critical position adjacent to the passed vehicle is approximately $\frac{1}{3} d_2$. Until the passing vehicle reaches point C, the sight distance required to abort the maneuver is less than the sight distance required to complete the maneuver. Beyond point C, greater sight distance is required to abort than to continue the maneuver, so the passing driver is committed to complete the pass. At point E, the passing vehicle returns to the right lane with proper clearance to the passed vehicle, now located at D. An opposing vehicle will travel a distance d_4 (equal to $\frac{2}{3} d_2$) from G to F while the passing vehicle moves from C to E. Proper sight distance must be provided to ensure that the opposing vehicle is no closer to the passing vehicle than point F when the passing vehicle returns to the right lane. The sight distance required by the passing driver in the critical position is, therefore, the sum of the remaining distance before the passing vehicle returns to the right lane ($\frac{2}{3} d_2$), the distance traveled by an opposing vehicle before the passing vehicle returns to the right lane (d_4), and a clearance distance (d_3).

The studies described above used both the critical position hypothesis and results of field studies on passing maneuvers to derive recommended sight distance standards. Although the two studies used different assumptions in choosing the distances associated with the elements of the passing maneuver, their recommended sight distances are very similar, as shown in Figure 3. Also shown in this figure are the sight distance standards from the current edition of MUTCD and from an earlier draft of that edition. Although the basis for the sight distance standards proposed in the draft was never documented (nor was the reason for rejecting these recommendations), the new standards proposed in that draft are very similar to those of the two research studies described above.

In addition to the sight distance recommendations, Weaver and Glennon also stressed the need for minimum passing zone lengths. Their recommendations are based on the sum of 85th percentile distances found in their field studies for the perception-reaction distance, d_1, and the left-lane distance, d_2. These recommended zone lengths are

Design Speed (km/h)	Minimum Length of Passing Zone (m)	Design Speed (km/h)	Minimum Length of Passing Zone (m)
80	270	105	407
97	361	113	453

These minimum zone lengths represent the distance required to make a pass with the 85th percentile condition. With a shorter zone, the possibility of a passing vehicle being in the critical position beyond the end of the zone (where available sight distance is less than the recommended) is greatly increased.

Another study by Jones (13), done in conjunction with the Weaver and Glennon study, was undertaken to prove that the MUTCD allowance of a 122-m (400-ft) passing zone length was inadequate. Although this study was not rigorous, it shed light on the relationship of marking practice and actual highway operations.

The Jones study evaluated the use and safety of short passing zones on two-lane highways. Three short passing zones of 122, 195, and 268 m (400, 640, and 880 ft) were chosen. The three sites had similar ADT volumes and geometrics and reasonably similar lengths of no-passing stripe on the approach to the zone (490 to 670 m or 1600 to 2200 ft). In addition, two longer zones having lengths of 500 and 792 m (1640 and 2600 ft) were studied for comparative purposes. The posted speed limit for all five sites at the time of study was 113 km/h (70 mph).

The study included a subjective evaluation of the proportion of passing opportunities that resulted in completed passes. A passing opportunity was defined as a situation in which a vehicle entered one of the study areas trailing another vehicle within four car-lengths (approximately 25 m or 80 ft) and was, in the judgment of the observer, awaiting a chance to pass the lead vehicle. An average of 125 such passing opportunities occurred at each of the three short zones during the study period.

Figure 4 shows the results of the evaluation of passing zone use (13). Fewer than 9 percent of the passing opportunities were accepted at each of the three short passing zones. By contrast, the 500-m (1640-ft) zone had 22.8 percent use, and the 792-m (2600-ft) zone had 41.0 percent use. These results, though based on limited observation, cast doubt on any claim that short passing zones add substantially to the level of service on two-lane highways.

Additional data were collected at the three short zones about each passing opportunity that resulted in a passing maneuver. The safety of the return of the passing vehicle to the right lane at the completion of the maneuver was subjectively rated on a severity scale of 0 to 2 based on the following definitions:

Rating	Definition
0	Smooth return from passing lane to normal operating lane
1	Forced return in which the passing driver apparently realized that the remaining sight distance was less than adequate
2	Violent return in which the passed vehicle or an opposing vehicle was forced to brake or move to the shoulder

Also, the location of the return to the right lane was recorded for each completed pass.

Figure 5 shows the distribution of severity ratings for the return maneuvers of completed passes for each of the three short zones. The proportion of observed hazardous maneuvers decreased as the zone length increased. Forced or violent returns occurred in 63 percent of the passes on the 122-m (400-ft) zone, 45 percent of the passes for the 195-m (640-ft) zone, and 10 percent of the passes for the 268-m (880-ft) zone. Only the results for the 268-m zone appear tolerable under any reasonable safety standard.

The point of return of passing vehicles to the right

Table 1. Basic assumptions and derived sight distances.

Assumption	Speed of Passing Vehicle (km/h) 48	64	80	97	113
Speed differential between passing and passed vehicles, km/h	16.1	20.1	24.1	32.2	40.2
Speed of opposing vehicle, km/h	40.2	52.3	64.4	74.4	88.5
Sight distance for flying passes, m	134	168	201	201	201
Sight distance for delayed passes, m	155	232	332	421	543
Suggested minimum sight distances, m	152	183	244	305	366

Note: 1 km/h = 0.62 mph; 1 m = 3.28 ft.

Figure 1. Passing distance in relation to speed.

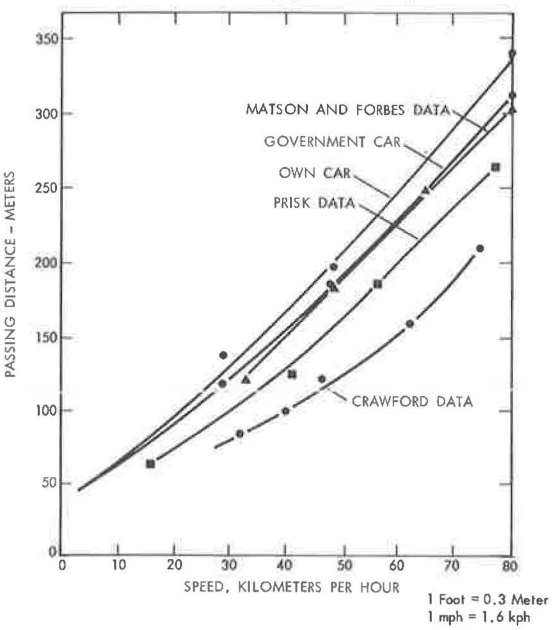

Figure 2. Completion of a critical passing maneuver.

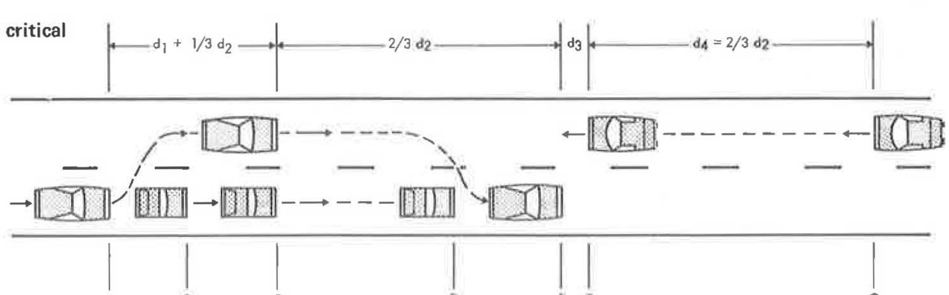

Figure 3. Minimum sight distances required to pass.

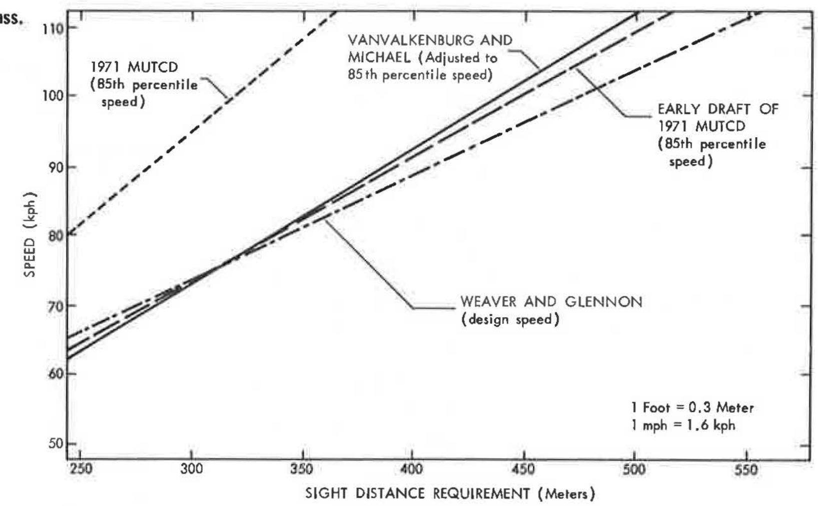

Figure 4. Driver acceptance of passing opportunities.

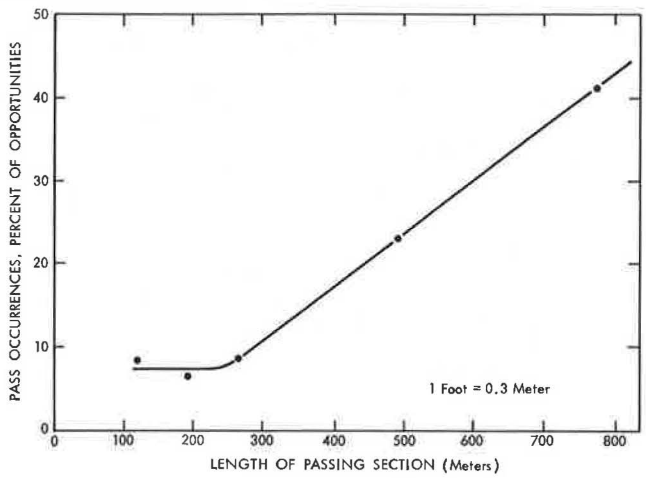

Figure 5. Return maneuver severity rates.

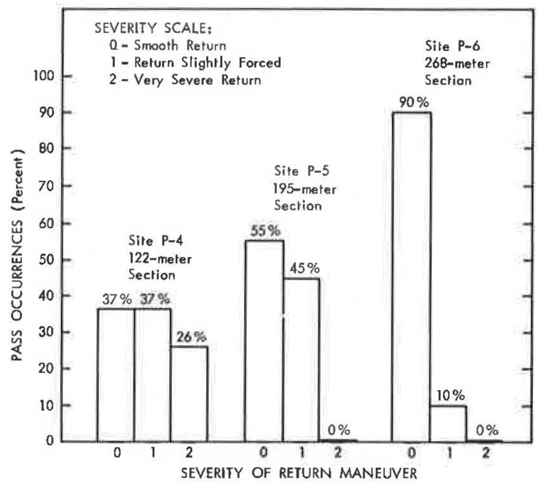

Table 2. Design criteria and marking warrants.

Design Speed (km/h)	Minimum Passing Sight Distance (m)			Minimum Length of Passing Zone (m)		
	Recom-mended	AASHTO	MUTCD	Recom-mended	AASHTO	MUTCD
48	–	335	152	–	–	122
64	–	457	183	–	–	122
80	346	549	244	270	–	122
97	451	640	305	361	–	122
113	556	762	366	453	–	122

Note: 1 km/h = 0.62 mph; 1 m = 3.28 ft.

lane was also recorded as an indication of safety and legality. The standards established by the MUTCD and the laws governing highway operation in many states require a driver to complete a pass before entering a no-passing zone (15). On this basis, all 11 of the observed passes on the 122-m (400-ft) section were illegal. On five of these 11 passes, the passing vehicle did not return to the right lane until more than 122 m after the beginning of the no-passing stripe. Only one pass out of nine on the 195-m (640-ft) section and two of the 10 passes observed on the 268-m (880-ft) section were legal. For all three study sites, the drivers who penetrated the no-passing zone entered an area of extremely restricted sight distance.

The results of the Jones study indicate that most drivers are reluctant to use passing zones shorter than 268 m long. The overwhelming majority of drivers who did use such zones did so illegally or unsafely. A more complete study is needed to establish in detail the relationship of frequency and safety of passing zone use to zone length and available sight distance. However, available data suggest that the MUTCD standards for striping highways should be modified.

NEED FOR INTEGRATED DESIGN AND OPERATIONS

Because as many as 5000 fatalities a year may be associated with passing maneuvers on two-lane highways, there is a definite need to critically reevaluate the current design and marking standards associated with the passing maneuver. Also, in a time of increasing litigation against highway agencies concerning their safety responsibility in highway accidents, these agencies are concerned about the adequacy of these standards and their compatibility with existing state laws regarding the use of passing and no-passing zones. Such a reevaluation is particularly timely since nationwide speed limits have been reduced. Many passing zones that were marginally safe at 113 km/h (70 mph) may be completely adequate at 88 km/h (55 mph), and improved standards that reflect this apparent benefit should be promulgated before highway agencies begin remarking these zones according to lower speed standards.

Most highway agencies use either the AASHTO and MUTCD standards directly or some subjective modification of them. Therefore, these standards (and related state laws) need to be revised to relate more objectively and practically to the safety of passing maneuvers, as discussed earlier. Although the most predominant need is for objective and sound marking standards, design standards based on the same principles could also derive significant safety and operational benefits on the many kilometers of two-lane highways (particularly primary highways) that are reconstructed each year.

Indeed, it is time to reconsider whether the continuation of separate design criteria and marking warrants for passing and no-passing zones is desirable. The development of combined design and marking standards can promote both operational efficiency and safety. Such standards should include both minimum passing sight distances and minimum lengths of passing zones.

Consideration of passing zone operations in the design process is highly desirable because subsequent operations are largely fixed by design decisions. Minor adjustments of horizontal and vertical geometrics can increase both the safety of passing operations and the use of marginal passing zones in rolling terrain. Adjustment of geometrics to optimize passing operations must be integrated into the design process, because such adjustments are not economically feasible after the highway is built. Combined standards for design and operation of passing zones would encourage the designer to assess directly the effects of design decisions on the length and sight distance available in marked passing zones on the completed highway.

SUMMARY

An evaluation of passing sight distance standards shows that $\frac{2}{3} d_2 + d_3 + d_4$ is a more logical model of the sight distance requirements of passing drivers than is $d_1 + d_2 + d_3 + d_4$, which is used in current AASHTO design standards. Current MUTCD passing sight distance standards do not relate objectively to the safety and operational efficiency of passing zones. AASHTO design standards place no restrictions on the length of passing zones. The MUTCD allows the use of passing zones as short as 122 m (400 ft) long. Jones' study demonstrates that passing zones shorter than 268 m (880 ft) long do not increase service substantially but do increase accident potential.

Combined design criteria and marking warrants for passing and no-passing zones will promote both operational efficiency and safety. The criteria given in Table 2 are suggested for both design and marking of safe passing zones. Passing should not be allowed unless the minimum sight distance is available to the passing driver for at least the minimum passing zone length. The basis for these criteria has been discussed and is thoroughly documented by Weaver and Glennon (23). For comparative purposes, the present AASHTO and MUTCD criteria are also given.

The question that remains is what is the trade-off between traffic service and safety for the variety of traffic and highway conditions present at passing zones. This suggests a need for more rigorous examination of the distributions of passing maneuver variables for various passing zone environments.

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Roadside Encroachment Parameters for Nonfreeway Facilities

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A recently published NCHRP report (1) has shed light on how to evaluate and compare the degree of hazard associated with roadside obstacles. To evaluate the effectiveness of roadside safety improvements, a probabilistic hazard index model was developed. This model accounts for (a) vehicular roadside encroachment rates, (b) percentile distribution for the lateral displacement of encroaching vehicles, (c) encroachment angle, (d) lateral placement of the roadside obstacle, (e) size of the obstacle, and (f) accident severity associated with the obstacle. The predicted difference between the hazard index before and after improvement indicates the effectiveness of the roadside safety improvement.

The objective of the research (2) described here was to enlarge the applicability of the hazard model developed in NCHRP Report 148 so that it can be used for predicting the effectiveness of roadside safety improvements on all classes of highway. This involved collecting additional data for estimating roadside encroachment rates, encroachment angle distributions, lateral displacement distributions, and obstacle severity indexes for all classes of highway other than freeways: urban arterial streets, rural two-lane highways, and rural multilane surface highways.

ROADSIDE OBSTACLE SEVERITY INDEXES

The severity index is a measure of the average consequence of a vehicle impact and is an integral part of the roadside hazard index model. Generally, any safety program is aimed at reducing total fatal, injury, and property damage accidents. Therefore, any improvement scheme that assigns higher weights to the more severe accidents will tend to satisfy these aims. The severity index considered here is the proportion of total accidents that are either fatal or nonfatal injury accidents.

So that the hazard model would be usable for all classes of highways, severity indexes were identified for the various roadside obstacles classified by type of highway. The major premise was that, as average operating speeds increase, the severity index of a particular roadside obstacle increases. Therefore, for a particular obstacle, the severity index is expected to increase from urban streets to rural at-grade highways to freeways.

Severity data on single-vehicle roadside obstacle accidents were requested from 34 city and 13 state agencies. Of these agencies, 8 cities and 10 states were able to provide data suitable to the needs of this study. City agencies were asked for accident severity data on urban roadside obstacles on streets with speed limits of 48 to 72 km/h (30 to 45 mph). State agencies were asked for accident severity data on roadside obstacles along rural nonfreeway roadways with speed limits of 80 to 112 km/h (50 to 70 mph).

The subject research report lists developed severity indexes for freeways, rural surface highways, and urban streets for the following roadside obstacles: utility poles, trees, sign posts, light poles, traffic signal poles, railroad signal poles, curbs, guardrails, roadside slopes, ditches, culverts, drainage inlets, bridge abutments and piers, bridge rails, retaining walls, fences, and fireplugs.

ROADSIDE ENCROACHMENTS

A roadside encroachment occurs when a vehicle leaves the traveled way either because of loss of driver control or because of an emergency maneuver to avoid collision with another vehicle. The parameters that describe the nature of these encroachments are the encroachment rate, distribution of encroachment angles, and distribution of lateral displacements of encroaching vehicles.

Because of time and funding constraints, estimates of the pertinent encroachment parameters were made by using roadside accidents as the basic data source. Officials of Kansas City, Missouri, and the Missouri State Highway Commission were very cooperative in providing the necessary accident data and roadway and traffic inventory data.

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Table 1. Equations of roadside accident rate versus ADT for various classes of highway.

Highway Class	Regression Equation	Correlation Coefficient	Standard Error
Rural freeway	y = 0.476 + 0.000 172 (ADT)	0.731	1.330
Rural multilane divided	y = 0.663 + 0.000 113 (ADT)	0.641	0.861
Wide rural two-lane (roadbed ≥ 10.9 m)	y = 0.182 + 0.000 142 (ADT)	0.445	0.355
Narrow rural two-lane (roadbed < 10.9 m)	y = 0.159 + 0.000 142 (ADT)	0.590	0.400
Urban arterial streets	y = 0.474 + 0.000 254 (ADT)	0.608	2.570

Note: 1 m = 3.28 ft.

Encroachment Rates

Roadside encroachment rates are normally higher than reported roadside accident rates because all encroachments do not result in accidents. The only pure encroachment data available are those of Hutchinson and Kennedy (3) for freeway medians. Therefore, for this study, the encroachment rates were estimated from accident rates by multiplying all accident rates by the ratio of freeway encroachment rates (twice the median encroachment rate of Hutchinson and Kennedy) to freeway accident rates (measured in this study).

The accident data were analyzed by using simple linear regression analysis in which data points were weighted by roadway section length. To achieve maximum discrimination, several classification variables were also investigated including type of street, speed limit, frequency of fixed objects, and presence of curbs in urban areas and type of highway, roadbed width, and average operating speed in rural areas. Of these classification variables, type of highway in rural areas and roadbed width for two-lane rural highways were the only ones that provided some discrimination of accident rates.

The resulting roadside accident rate versus ADT relationships are shown in Table 1. These relationships are defined by the regression equations and the standard descriptors of goodness of fit, the correlation coefficient, r, and the standard error, S.E. The encroachment frequencies for highways other than freeways are estimated by multiplying the slope of each accident line by the ratio (5.23) of freeway encroachments to freeway accidents. These are simply order of magnitude estimates to be used in the absence of true encroachment data.

Encroachment Angles and Lateral Displacements

Collision diagrams from accident reports were used to record dimensions of accident encroachments for determining encroachment angles and the lateral displacements of encroaching vehicles. Lateral displacement was measured to the right-front corner of the vehicle at its final resting place and requires identification of either that dimension directly or the other two sides of the encroachment triangle. To compute the angle of encroachment requires that any two sides of the encroachment triangle be identified.

The subject research report shows resulting exceedance distributions for encroachment angles and lateral displacement distances for urban arterial streets and rural two-lane highways. With minor variations, these distributions are similar to those found for freeway medians by Hutchinson and Kennedy.

CONCLUSIONS AND RECOMMENDATIONS

Applying the developed roadside encroachment parameter estimates in the hazard model suggests that relatively little effectiveness can be gained by implementing roadside safety improvements on highways other than freeways. This negative conclusion, however, must be interpreted in light of the limitations of the data presented in this report. The roadside encroachment rates developed here are only average rates and do not account for higher rates at specific locations. For example, the encroachment rates for highway curves, for weaving sections, or for sections with extremely low skid resistance are expected to be much higher than the average. Unfortunately, the data needed to detect these variances from the average are difficult to compile and therefore have not been investigated by anyone.

Because the hazard index is directly proportional to encroachment rate, it is easy to investigate the sensitivity of the hazard index to a change in encroachment rate. For example, if the encroachment rate for a particular highway curve geometry is three times the average, the hazard index would be three times the average. If this kind of condition could be detected, more kinds of roadside hazard improvements at more highway locations could be justified.

To be able to identify and justify roadside safety improvements for highways other than freeways, therefore, requires further research to improve the precision of the hazard model so that it accounts for hazard-sensitive site-specific parameters.

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Effect of the Energy Crisis on Existing Design Standards

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An immediate reaction to the energy crisis by highway engineers was a proposal to summarily reduce design standards, especially design speed. This paper discusses why such a reduction should not be effected. Factors that should be considered before standards are reduced are (a) AASHTO definition of design speed, (b) possibility of 88-km/h (55-mph) speed limit being temporary, (c) the effect reducing design speed could have on multimodal corridors, (d) higher order of safety provided by higher design speeds, (e) increased use of smaller cars, (f) liability of highway engineers, and (g) current research on situational design criteria.

The energy crisis is one of many socioeconomic and political factors that have adversely affected the funds available for highway operation, both construction and maintenance. As an immediate reaction to the reduced availability of highway funds, some design engineers have proposed that geometric design standards be reduced. These proposals have, for the most part, centered on a reduction in the design speed to 88 km/h (55 mph), since that is the current maximum speed limit specified in the Federal-Aid Highway Act of 1974.

While it does not appear that the adoption of an 88-km/h (55-mph) design speed is an appropriate solution to the current problem of reduced highway funding capabilities, a brief discussion of the considerations involved in such a change is appropriate. A review of the various geometric design policies of the American Association of State Highway and Transportation Officials (1) indicates that there is nothing to prevent such a change since there are nine specific design speeds ranging from 32 to 129 km/h (20 to 80 mph). Although 88 km/h (55 mph) is not one of these, the geometric design requirements at this speed can be determined by interpolating between the values specified for the 80 and 97-km/h (50 and 60-mph) design speeds. However, there are several factors that should be considered before a policy of using an 88-km/h (55-mph) design speed is adopted.

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CONSIDERATIONS IN REDUCTION OF DESIGN STANDARDS

AASHTO Definition of Design Speed

AASHTO (1) defines design speed as "a speed determined for design and correlation of the physical features of a highway that influence vehicle operation. It is the maximum safe speed that can be maintained over a specified section of highway when conditions are so favorable that the design features of the highway govern." AASHTO also states (2) that design speed should be selected consistent with the terrain, type of highway, expected traffic volumes, and economic considerations. AASHTO policy indicates that "every effort should be made to use as high a design speed as practicable to attain a desired degree of safety, mobility and efficiency." The requirements for selecting design speed in urban areas (3) are consistent with those outlined in the earlier publication.

Thus, AASHTO policy, which is adopted by FHWA, is that a design speed should be higher than the anticipated operating speed. Table III-1 of the "blue book" (2) indicates that the assumed operating speed for wet pavements is between 94 and 80 percent of the design speed between 48 and 129 km/h (30 and 80 mph) respectively. To be consistent with this policy, a design speed of 105 km/h (65 mph) should appropriately be selected if an 88-km/h (55-mph) operating speed is to be maintained over a given section; and 105 km/h (65 mph) is specifically provided for in the "blue book." Further, a recent study of traffic speeds to determine the effect of the 88-km/h (55-mph) speed limit on operating speeds shows that, while there has been a definite decrease, only 53 percent of all vehicles on main rural roads conform to the 88-km/h (55 mph) speed limit.

88-km/h (55 mph) Speed Limit May Be Temporary

At hearings before the Senate Public Works Committee on increased truck size and weight, Senator Bentsen suggested that the 88-km/h (55-mph) restriction might be lifted in the future. Just as the maximum size and weight limitation on trucks has now been increased, so, too, could the current speed limit be increased by future legislation. In fact, a recent article (7) that listed congressional objectives with respect to automobile energy conservation did not include the maintenance of the 88-km/h (55-mph) speed limit in either short- or long-term objectives.

Throughout the years, highways have always been constructed by using the best available technology; as a result roads built in 1925 are in use today. This fact constitutes one of our major concerns: "How do we upgrade these older facilities to meet the safety standards in use today within the availability of highway funds?" By the same token, highways built today may well be in use in the year 2025; thus, it is imperative that engineers use the best standards available.

Although we do not know what the future automobile will look like, the technology available in the automobile industry will most probably produce vehicles with more efficient power plants and vehicles that are not so dependent on petroleum. Thus, the future safety and mobility demands placed on the highway may very likely be similar to those we know today.

Accordingly, highways should be designed with safety, mobility, and costs in mind. Where standards below those that have proved to provide a high degree of safety are proposed, a case to use these less-than-minimum designs should be made to the FHWA on a project-by-project basis.

Use of Highway Facilities as Multimodal Corridors

In some instances, the use of lower design speeds would seriously limit, if not prevent, the use of highway facilities as multimodal transportation corridors. Primarily, rail is the only other transportation mode that could jointly use a land corridor. The geometric requirements for rail are more restrictive than those for a highway facility in both horizontal and vertical alignment. However, by combining modes in a corridor, some of the costs can be shared, such as right-of-way acquisition and construction. For example, Metro, the 158-km (98-mile) rail transit system being built in the Washington, D.C., area, was proposed to use the I-66 median for rail lines. Because it now appears that I-66 will not be built, Metro wants to maintain control of sufficient right-of-way within the current highway corridor to build lines. California is now considering multimodal transportation corridors, particularly when new facilities are planned.

There are many instances across the country in which right-of-way lines of a highway and a railroad abut each other; thus, both modes are using the same corridor.

Table 1. Accident rates on Interstate and federal-aid primary and secondary highway systems.

System	Fatalities	Injuries	Rate[a] Fatality	Rate[a] Injury
Interstate	4 946	169 225	2.31	78.92
Federal-aid primary				
Interstate	1 680	65 916	4.39	172.44
Other	18 681	750 852	4.79	192.42
Total	20 361	816 768	4.75	190.63
Federal-aid secondary				
State	9 262	283 493	6.31	193.13
Local	4 542	227 730	5.21	261.22
Total	13 804	511 223	5.90	218.50

[a]Per 62 million vehicle-km (100 million vehicle-miles) traveled.

Safety Features of Higher Design Speeds

Higher design speeds not only meet today's mobility demands but also generally provide a higher order of safety. Several research studies (4, 5) have shown that geometric elements such as longer sight distances, flatter horizontal curves, and flatter grades decrease accident experience. A comparison of the 1973 fatal and injury accident rates for the Interstate, federal-aid primary, and federal-aid secondary highway systems (Table 1) indicates generally that as design speed decreases the accident rate increases (10).

Recent research has shown that the flatter grades and flatter horizontal curves associated with the higher design speeds reduce fuel consumption (11). Thus, the use of a lower design speed to reduce construction costs will cause an increase in vehicle operating costs.

An issue related to the general safety implications of lower design speeds is the mixing of design speeds, e.g., a section having a 113-km/h (70-mph) design speed followed by a section with an 88-km/h (55-mph) design speed. An example of this is in Washington, D.C., where the Capitol Beltway (I-495) goes through Rock Creek Park. In the Rock Creek Park section, a lower design speed was imposed; however, in spite of the 80-km/h (50-mph) speed limit, the accident rate is higher in this section than in the two adjoining sections having higher design speeds. The accident rate through Rock Creek Park was 106/100 million vehicle-km (171/100 million vehicle-miles) as compared to 84/100 million vehicle-km (135/100 million vehicle-miles). Such situations violate driver expectancy (8); consequently, highway engineers have been accused of ignoring the human element in the design process.

Increase in the Use of Small Cars

The increasing use of small cars (compacts and subcompacts) is causing concern about whether the current sight distance requirements—1.14-m (3.75-ft) eye height and 0.15-m (0.5-ft) object height—are inadequate (9). New Jersey has indicated that inadequate sight distance may be a cause for the increase in small car accidents occurring in that state. Canada is also concerned that the eye height is inappropriate because of the increasing number of smaller cars on their highways; therefore, as part of their metrication effort, a 1.05-m (3.45-ft) eye height has been recommended as a change to Canadian geometric design standards. Should a lower eye height be accepted, a longer sight distance on crest vertical curves would be required.

Liability of Highway Engineers

The question of liability of highway engineers in negligence suits resulting from highway accidents should be considered. Paul W. Clark, former chief of litigation for the State Highway Commission of Kansas, pointed out that, in some instances, the courts have specified that the design be implemented when they believe the highway agency has not appropriately improved a highway feature in accordance with advanced state of the art in highway safety (6). Many states have given up their sovereign immunity and have thus opened the way for personal negligence suits against the highway engineer at all levels.

SUMMARY

An 88-km/h (55-mph) design speed or any other criterion should not be adopted as an immediate panacea to current fiscal problems without a good understanding of the implications of such decisions. Currently under way and planned is research that will provide a better understanding of the many unknowns of this situation. The objectives of one ongoing study (12) are to

1. Quantify the effect on accident frequency and severity of varying the magnitude, size, or dimension of each roadway and roadside design element and combinations of the elements,
2. Develop a methodology for measuring the cost effectiveness of these elements or combinations, and
3. Provide a readily usable design guide for the highway design engineer.

FHWA has plans to expand the work of Laughland and Schoon (12) so that most, if not all, of the geometric design elements or combinations recommended for further research will be investigated. The FHWA study will use the earlier work as a basis for beginning the FHWA research. Both a literature review and evaluation methodology will be used, and this research will only be done on those geometric elements or combinations that cannot be comprehensively evaluated in the earlier work.

Formal documentation of these studies will not be available until late in 1977; however, interim results of various investigations will be readily available from the progress reports required during the conduct of these research efforts.

Thus, the energy crisis should not be permitted to affect existing geometric design standards at least until ongoing or planned research is completed.

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Roadside Hazards on Nonfreeway Facilities

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The increasing emphasis on highway safety, due in part to the Highway Safety Act of 1966 and the more recent congressional hearings on highway safety, design, and operations, is directed primarily toward improving the Interstate Highway System. Numerous safety improvement projects around the country are applying the clear roadside concept to existing Interstate facilities, an effort certainly warranted by the fact that 16 percent of all vehicle-kilometers of highway travel occurs on this system. However, additional attention to highway safety is warranted on non-Interstate systems, which carry the remaining 84 percent of all travel and account for more than 91 percent of highway fatalities. In both a technical and financial sense, however, the design and operational features for reducing single-vehicle accidents on freeways are not directly applicable to nonfreeway facilities. This is especially true for the largest class of single-vehicle accidents, those involving fixed objects along the roadside.

CONCEPT OF A ROADSIDE HAZARD

Most research on the roadside environment has been directed toward specific types of items. Previous reports (1, 5) have discussed the elements of the roadside environment and have referred to them jointly as roadside hazards. The extensive use of photographs in these reports to depict the poor design and installation of roadside elements creates the impression that any reasonably capable engineer should be able to identify and correct these hazards on existing roadways and eliminate them from future designs. Notwithstanding some conceptual development (3), the technical literature does not contain a comprehensive, definitive statement for determining whether a particular object is in fact a roadside hazard. In the absence of a formal definition, engineers could respond that they know a roadside hazard when they see one. In a somewhat circular vein, a roadside hazard could be described as any element that conflicts with the much-publicized 9.1-m (30-ft) clear roadside recovery area. Inasmuch as these two indirect criteria do not apply to nonfreeway facilities, the following set of definitions are proposed

Roadside furniture: Includes all fixed or semipermanent objects, both publicly and privately owned, that are located off the traveled portion of the roadway but that are within 9 m (29.5 ft) of the nearest edge of the traffic lanes.

The definition is intended to include the general category of fixed objects and certain objects of a less permanent nature (e.g., temporary signs, debris left by traffic accidents). The limitation of roadside furniture to objects within 9 m (29.5 ft) of the traveled roadway is admittedly arbitrary but not particularly critical in the subsequent concept development.

Roadside obstacle: Any element of roadside furniture that, because of its size, rigidity, design, or manner of placement, causes an impacting vehicle and its occupants traveling at prevailing highway speeds to be severely decelerated (or redirected) or causes the interior of such a vehicle to be seriously violated.

This definition recognizes the extent to which roadside furniture impedes the operation of a vehicle after it has departed from the traveled way. The severe deceleration noted in the definition has not been quantified, although the numerical criteria suggested by research involving attenuators (6) may be appropriate. According to this definition, unprotected bridge piers, most trees, and some drainage facilities are roadside obstacles. Although a completely safe roadside environment would be one that was free of all obstacles, it is apparent that the elements fitting this definition do not have a uniform accident experience.

Roadside hazard: Any roadside obstacle that, because of its placement and the design and operational characteristics of the adjacent roadway, has an above-average probability of being struck and causing severe occupant injury.

The concept of a roadside hazard combines the severity

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level characteristic of an obstacle with the likelihood of impact. While severity of impact at a given speed is most closely related to design characteristics of the obstacle, the probability of impact is more closely related to several roadway and obstacle parameters, including roadway geometrics and obstacle position. For example, previous research (2, 7) has found that accident experience is higher on horizontal curves than on tangent sections.

CHARACTERISTICS OF SINGLE-VEHICLE, FIXED-OBJECT ACCIDENTS

Although the suggested definitions may be lacking in certain respects, they do clearly indicate that the probability of impact and the accident severity for particular fixed objects are the important considerations. These characteristics were initially examined by using the 236 000 accidents in the Maryland accident record system for 1970 to 1972. To analyze single-vehicle, fixed-object (SVFO) accidents required that a modified accident data base be established based on the following criteria:

1. Only one vehicle was involved,
2. The accident occurred on a Maryland route or a U.S. route in Maryland, and
3. The fixed object was coded for manner of collision.

For the 3-year period, 19 743 accidents (8.4 percent of the statewide total) met these criteria.

Several general characteristics describe these SVFO accidents. Passenger cars were involved in 88 percent of the accidents. Approximately 73 percent of the SVFO accidents were reported as "non-intersection related," as opposed to 45 percent for all other accidents on Maryland and U.S. routes. In comparison with other accidents, a higher percentage (60 versus 47) occur on two-lane roads without access control. Analysis of the records indicates that SVFO accidents occur most frequently on weekends, during the hours of darkness, under adverse pavement conditions, during inclement weather, and on horizontal curvature. Vehicle speed was cited as a probable cause in 44 percent of the accidents. Driving under the influence of alcohol was listed as the probable cause in 8 percent of the SVFO accidents, although in 27 percent the drivers were characterized as "had been drinking." In 10 percent of the SVFO accidents an unknown vehicle was considered at fault.

The severity index (SI) for all SVFO accidents is 0.44, considerably higher than the SI of 0.34 for other accidents on Maryland and U.S. routes. From the modified data base, the SI was evaluated for each of the 14 types of fixed object used in the accident record system. Accidents involving trees had the highest severity index (0.61), followed closely by those involving utility poles (0.59). The most frequently struck objects were utility poles, which accounted for more than 16 percent of all SVFO accidents. The severity and frequency data were combined by using a ranking procedure (4). Based on this procedure, which represents the highest level of sophistication that can be obtained solely from the record system, utility poles are the most serious roadside hazard.

ACCIDENT SITE INVESTIGATIONS

Because the record system does not provide information sufficient for evaluating the engineering aspects of SVFO accidents, the modified accident data base was used to choose locations for field study. The selection procedure (4) identified 105 study sections, ranging in length from 0.8 to 6.4 km (0.5 to 4.0 miles) and having a total length of 270 km (168 miles). Although the sections accounted for only 3.4 percent of the total Maryland and U.S. highway distances, 13.5 percent (2664) of the reported SVFO accidents occurred on these routes. The combined study sites have 3.3 SVFO accidents/km/year (5.3/mile/year), approximately four times the statewide average. At some sites, utility poles are involved in half of all SVFO accidents. Other sections, lacking bridges, curbs, or guardrail, obviously have no accidents involving these objects. Overall, the study sites experienced approximately the same relative accident frequencies and characteristics as the modified data base.

Field investigations were conducted at 75 percent of the study sections, and photographic logs were used to examine the remaining sites. The investigations identified those fixed objects involved in reported accidents as well as those that had been struck but were not included in the accident record system. On some routes, it was difficult to locate the specific objects cited in the accident record probably because of inaccurate coding of mileposts and the use of nondescriptive collision codes. This problem has been corrected in the current accident record system.

The field investigations identified several characteristics that were common to many of the SVFO accident sites.

1. Narrow highway right-of-way. Many of the rights-of-way were 9.1 to 12.2 m (30 to 40 ft) wide. This restricts the lateral placement of features maintained by the highway administration as well as utility poles, which frequently share the highway right-of-way.
2. Curves. Thirty-five percent of the SVFO accidents occurred on curves, the majority involving objects on the outside of the curve.
3. Lateral placement. Comparatively few of the SVFO accidents on two-lane roads involved objects farther than 4.5 m (14.8 ft) from the edge of the roadway. The most serious problems occurred with respect to trees, which were occasionally at the edge of the pavement.
4. Outdated designs. Many of the objects struck, most notably drainage facilities and guardrails, were not in accord with currently accepted design practices. In some cases, this increases the likelihood of them being struck, but, more commonly, it increases the severity of a collision.
5. Treatment. In many cases, the treatment of obstacles was inadequate. This problem was especially noticeable at terminals of bridges and drainage headwalls, and side slopes were too steep.
6. Combination effects. In some instances, isolated obstacles were placed adjacent to continuous objects (e.g., ditches, guardrail) in a manner that increased the likelihood that they would be struck. The redirecting effect of continuous objects should obviously be considered in the location of roadside elements.

PLANS FOR CONTINUING RESEARCH

The preliminary work on this research, consisting of the accident record evaluation and the field site investigations, has verified that SVFO accidents on nonfreeway facilities warrant increased consideration. The continuing phases of this research will attempt to make the definitions of roadside obstacles and hazards more operational by developing criteria for distinguishing them. To facilitate a continuing program of roadside improvement, the roadside hazard identification procedures will be designed for use with photographic logs.

At this stage of the research, it is not possible to state with certainty which roadside obstacles are the most hazardous. Because the technical literature fails to adequately discuss roadside hazards on nonfreeway facilities, the preliminary findings in Maryland cannot be compared to the situation in other states. Although the problems found in this study are serious, there is no reason to believe that the situation in Maryland differs significantly from that nationwide.

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Effect of Commercial Vehicles on Delay at Intersections

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This paper reports the results of a study of the effects of commercial vehicles on intersection delay. The objective was to determine the delay to through traffic at signalized intersections caused by commercial vehicles and to determine the effect of intersection corner radii on right-turn speeds of commercial vehicles. Commercial vehicles are defined as any vehicle having at least six tires and two or more axles. Data were collected at intersections in five cities in Indiana. Twenty-three intersection approaches were studied for commercial vehicle delay, and 19 intersection corner radii were studied for right-turn speeds of commercial vehicles. The results of this research are both quantitative and qualitative. It was found that the average travel time of a passenger car through a signalized intersection was increased from 39.9 to 49.4 s when one or more commercial vehicles were traveling ahead of it in the same platoon of vehicles. Significant factors or variables were found to either increase or decrease delay due to commercial vehicles. Right-turn speeds of passenger cars, truck combinations, and commercial vehicles were found for varying intersection corner radii. From a delay viewpoint, a 9-m (30-ft) radius was found to be optimum for a single-unit truck and an 18-m (60-ft) radius was found optimum for a truck combination.

Traffic control at an intersection is of critical importance to the traffic engineer because it is at intersections that travel time delays and accidents are at a maximum. One factor contributing to delay at intersections is the presence of commercial vehicles or trucks. This factor is becoming increasingly important because registered trucks are increasing as a percentage of all vehicles, and the number of larger trucks being sold in the United States is increasing twice as fast as the total number of trucks and buses being sold. From 1960 to 1970, sale of trucks with six wheels and three axles increased 310 percent, and sale of motor trucks and buses increased 160 percent (2).

The objective of this research report was to quantify the delay to through traffic caused by commercial vehicles and to determine the effect of intersection corner radii on right-turn speeds of commercial vehicles.

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COMMERCIAL VEHICLE DELAY

Intersection delay is the difference between the actual travel time through an intersection and the travel time through the same intersection at normal roadway speed without deceleration, stopping, and acceleration. Recently, Geiger, Sofokidis, and Tilles conducted a literature review on the subject of intersection capacity and performance and concluded that the majority of authors preferred delay as the most desirable and tangible measure of intersection performance (5).

We conducted a literature review and found no studies that specifically evaluate the delays caused by commercial vehicles at intersections. It is believed that the difficulty in measuring delay has caused the lack of research in this area.

Measuring Delay Due to Commercial Vehicles

The first step in determining delay due to commercial vehicles was to define the length of roadway affected by a signalized intersection. This roadway distance originates at a point before an intersection where the average running speed on the roadway is reduced because of the presence of the intersection. The distance terminates after the intersection at a point where the average running speed on the roadway is continued. For this research, an average running speed of 40 km/h (25 mph) and an average maximum queue length of 75 m (250 ft) at an intersection approach during a peak period were assumed. The Traffic Engineering Handbook indicates that a deceleration length of 150 m (500 ft) is required for a vehicle to stop from a speed of 40 km/h (25 mph) (3, p. 50), and it has been determined that an average semitrailer requires 150 m (500 ft) to accelerate to a speed of 40 km/h (4, p. 215). Thus, vehicular movements were studied from 225 m (150 + 75) before an intersection to 150 m after the intersection.

For this study, the floating car method was used to measure travel times through an intersection. In this method, a test car repeatedly and at random enters a platoon of vehicles approaching an intersection and remains within the platoon until a point beyond the intersection. The travel time of the test car was measured from a point 225 m (750 ft) before the intersection to a point 150 m (500 ft) after the intersection. Each run was timed, and the following factors affecting commercial vehicle delay were determined by the observer in the test car:

1. Number of commercial vehicles ahead of the test car in the platoon,
2. Position of test car in the platoon,
3. Approach lane occupied by the test car,
4. Stop time caused by the red signal (if applicable),
5. Delay caused by pedestrians (if applicable), and
6. Delay caused by a slow right-turning vehicle (if applicable).

A stationary observer counted the following additional factors:

1. Approach volume per lane,
2. Number of loaded phases,
3. Right-turn volume,
4. Left-turn volume, and
5. Commercial vehicle volume.

Because they might affect commercial vehicle delay, the following items were inventoried at each intersection approach studied:

1. Approach width,
2. Parking conditions,
3. Number of approach lanes,
4. One-way or two-way street,
5. Metropolitan area population,
6. Curb parking on approach,
7. Type of traffic signal,
8. Curb radius,
9. Speed limit on approach,
10. Degree of right turn at cross street,
11. Length of green phase,
12. Curbing on approach, and
13. Exclusive turning lanes.

Delay due to commercial vehicles or trucks was determined by subtracting the average travel time through the intersection when commercial vehicles were present from the average travel time through the intersection when commercial vehicles were not present. The average travel time was composed of the average running time and the average stop time. The stop time in this research is the time from when a vehicle stops because of a red traffic signal to the time the traffic signal turns green. The running time is the running time of a vehicle through an intersection and the time that vehicle waits to start up after the traffic signal has turned green. The running time and the stop time were measured separately for each test car run.

Pilot Study for Delay Due to Commercial Vehicles

The number of vehicle runs or sample size needed to determine an average travel time was determined statistically from a pilot study. The statistical equation used required that the standard deviation, population or vehicles per peak period, error tolerance, and a probability level of not exceeding the error tolerance be estimated (8). The standard deviation and population were obtained by averaging data from the three intersections used in the pilot study. An error tolerance of 4 s with an accompanying 90 percent probability level of not exceeding the error tolerance was allowed to exist within each average travel time. The resulting sample size calculated from the statistical equation was 11.4. Thus, a minimum of 11 travel time runs was required. This required 22 travel time runs at each intersection approach or 11 runs for mean travel time with trucks and 11 runs for mean travel time without trucks.

Results of Data Collection

Twenty-three signalized intersection approaches in five cities in Indiana were analyzed for delay due to commercial vehicles. These intersections were located in the fringe areas and outlying business districts of the five cities. Data were collected in the morning and evening peak traffic periods on clear days. Table 1 gives the results. The average delay due to trucks for all the intersection approaches was 9.5 s.

Testing the Results

The Wilcoxon signed rank test was performed on the 23 average truck delay times to determine whether trucks caused significant delay (9). A 95 percent confidence that trucks did cause delay was determined.

The next step was to determine whether the average stop time when trucks were present was significantly different from that when trucks were not present at a signalized intersection approach. The Wilcoxon signed rank test was again used. It was found that the hypothesis of no difference between the two average stop times could not be rejected. It is therefore concluded that the presence of trucks at an intersection does not significantly increase or decrease the average stop time of a vehicle. As a result, stop time was not used in determining delay due to trucks. Only the running time was used.

Regression analysis was used to develop a model for predicting delay due to commercial vehicles at an intersection and for determining which traffic, intersection, and metropolitan area characteristics affect truck delay at intersections. Nineteen factors that were thought to cause most of the commercial vehicle delay were measured at each of the 23 intersection approaches studied. These factors or variables and their measurement range are given in Table 2.

Stepwise linear regression was the analysis procedure selected. This regression procedure enters predictor variables one at a time into the regression equation, in order of highest partial correlation with the dependent variable. Predictor variables continue to be added until no significant variables remain or until the list is exhausted. The significance of a particular variable is determined by evaluating its F-ratio and tolerance level. The SPSS 15: Regression program was used to perform the stepwise linear regression (11). All the variables given in Table 2 and various interactions between these variables were entered into the regression analysis.

A method used to increase the significance of the regression equation or model was to eliminate those predictor variables from the model that displayed small multiple correlation coefficients or r^2 values. A partial F-test was conducted after each predictor variable was added to determine whether adding that predictor variable to the model resulted in a significant increase in the r^2 value.

The model in its final form is presented below. This model produced an r^2 value of 0.971 and an F-value of 4.67, significant at the 0.025 alpha level. An r^2 value of 0.971 indicates that the predictor variables explain 97.1 percent of the variation about the mean of the given values for the dependent variable, truck delay. The individual r^2 values for the significant predictor variables are given in Table 3. The estimate of the standard error of the model was 2.23 s. (Because the model accepted values only in customary units, the results are given only in those units.)

$$Y = -2.436 + 0.009\,69X_{13} + 0.000\,042\,7X_{26} + 0.236X_{19} - 3.867X_8 + 0.000\,022\,2X_{22} + 5.238X_{12} + 0.000\,000\,886X_{24} - 0.236X_3 - 0.222X_5 + 5.920X_9 + 0.336X_{17} - 0.000\,020\,3X_{18} - 9.572X_{10} \quad (1)$$

where

Y = truck delay in seconds,
X_{13} = peak-hour volume,
X_{26} = degree of right turn times percentage of trucks,
X_{19} = percentage of trucks,
X_8 = exclusive left-turn lane (yes = 1 and no = 0),
X_{22} = percentage of left turns times peak-hour volume times left-turn lane times left-turn green phase,
X_{12} = left-turn green phase (yes = 1 and no = 0),
X_{24} = metropolitan area population times speed limit in miles per hour,
X_3 = percentage of right turns,
X_5 = curb radius in feet,
X_9 = right-turn lane (yes = 1 and no = 0),
X_{17} = approach width in feet,
X_{18} = metropolitan area population, and
X_{10} = curbing on approach (yes = 1 and no = 0).

The residuals from the final model were investigated to check the practicability of the model in predicting truck delay. The residuals are the differences between the actual truck delay and the truck delay predicted by the model. The frequency chart of the residuals is shown in Figure 1. A W-test (10) was performed on the residuals to determine whether they followed a normal distribution. [Regression analysis was used under the assumption that these residuals are normally distributed (9).] The results of the W-test did not reject the hypothesis of normality.

An examination of the arithmetic sign preceding the predictor variables in the model also revealed the practicality of the model. The arithmetic sign that preceded many of the predictor variables was obviously to be expected. However, some of the arithmetic signs were not obvious until the study intersections were reviewed. The imperceptible signs were interpreted to read as follows.

1. The negative sign preceding left-turn lane indicates that a left-turn lane reduces truck delay. The intersection approaches that had left-turn lanes also had at least two other lanes. Most of the through passenger cars traveled in the center lane or lanes, and most of the through trucks traveled in the right lane. Because most of the through passenger cars are not in the same lane as the trucks, delay due to trucks is minimal.
2. The positive sign preceding approach width indicates that an increase in pavement width causes an increase in delay due to trucks. Most of the three- and four-lane approaches studied displayed speed limits equal to or greater than those on the two-lane approaches. The higher speed limits increase delay due to trucks because trucks take longer to accelerate to a higher speed than passenger cars.
3. The positive sign preceding left-turn green phase indicates that a left-turn green phase increases delay due to trucks. Many drivers assume that a separate left-turn green phase results in a reduced through green phase time. This may cause drivers to accelerate faster than normal from a stop at an intersection. Because trucks are unable to equal the faster acceleration of passenger cars, a greater delay results.
4. The positive sign preceding right-turn lane indicates that a right-turn lane increases truck delay. The presence of a right-turn lane usually indicates that there is at least one other lane for through movements only. Consequently, the through-only lane or lanes are forced to carry all the through trucks. This condition increases delay due to trucks because (a) right-turning vehicles cause no delay to through vehicles and, thus, do not offset truck delay and (b) through vehicles cannot change lanes to avoid a more slowly moving truck.
5. The negative sign preceding metropolitan area population indicates that a larger metropolitan area reduces truck delay. In this research, it was found that fringe areas and outlying business districts in larger metropolitan areas had speed limits that were usually lower than those in similar locations in smaller metropolitan areas. As previously stated, lower speed limits reduce delay due to trucks. Another reason is that drivers in larger cities tend to be more aggressive in their driving habits and take more chances to avoid a slow-moving truck.

The arithmetic signs preceding the interaction variables are dependent on the magnitude and effect of each variable in the interaction. It is difficult to determine how the magnitude and effect of each variable in the interaction influence the preceding arithmetic sign; thus, the arithmetic signs preceding interaction variables were not examined.

A final check of the practicability of the model was accomplished by testing it against an independent intersection. Data at this test intersection were obtained from observers stationed in a 12-m-high (40-ft) fire tower overlooking the intersection. A sample of 38 percent of the total through vehicles was collected. This sample size was statistically proved to yield acceptable average travel times. The sample produced a delay due to trucks of 2.78 s/vehicle. Values for the variables that occur in the model were determined at the test intersection and inserted into the model to produce a predicted delay due to trucks of 2.82 s. This small error substantiates the model's practicality.

Although several checks of the model's practicality proved to be positive, it is not concluded that the model is acceptable at every intersection. Certainly it may not be applicable for intersections with variables that fall outside the ranges given in Table 2.

RIGHT-TURN STUDY

One of the largest single contributors to vehicle delay at intersections is long trucks negotiating right turns. Many urban area intersections are not able to accommodate turning movements of truck combinations without encroachment on adjacent lanes. Often, one large truck combination will delay a lane of traffic for an entire signal cycle because of its inability to negotiate a right turn without encroaching on the opposing or adjacent lane on the cross street.

Data Collection for Right-Turn Study

Nineteen curb radii were studied to determine their effect on right-turn speeds of commercial vehicles. Each curb radius was measured as a simple curve radius, and curb radii from only right-angle intersections were studied. Speeds of right-turning vehicles were obtained by timing a vehicle along a predetermined distance with beginning and end reference points. The beginning reference point was located on the front tangent of the curve 18 m (60 ft)

Table 1. Data for study of delay due to commercial vehicles.

Intersection	City	Approach	Trucks Present Travel Time (s)	Trucks Present Stop Time (s)	Trucks Not Present Travel Time (s)	Trucks Not Present Stop Time (s)	Delay Due to Trucks (s)
Kentucky at Morris	Indianapolis	Northeast	49.5	10.0	43.7	10.3	5.8
Kentucky at West	Indianapolis	Southwest	57.3	23.5	41.7	15.6	15.6
Kentucky at Harding	Indianapolis	Northeast	49.4	19.4	38.4	9.7	11.0
Morris at Tibbs	Indianapolis	East	44.4	11.7	32.6	10.7	11.8
Morris at Harding	Indianapolis	East	72.9	5.1	64.7	5.2	8.2
Morris at Tibbs	Indianapolis	West	40.7	12.3	33.2	6.6	7.5
Morris at Harding	Indianapolis	East	58.7	14.5	46.4	13.8	12.3
Virginia at Stevens	Indianapolis	Northwest	35.8	1.4	28.9	0.5	6.9
Kentucky at White R.	Indianapolis	Northeast	39.2	12.4	26.5	8.0	12.7
Morris at Harding	Indianapolis	West	54.3	4.2	38.6	6.0	15.7
Meridian at South	Indianapolis	North	43.1	7.8	32.8	10.9	10.3
South at West	Indianapolis	East	52.2	40.1	44.2	40.8	8.0
Morris at Belmont	Indianapolis	East	40.9	0	36.3	0	4.6
Morris at Holt	Indianapolis	East	38.5	2.9	28.7	2.1	9.8
Northwestern at 16th	Indianapolis	South	38.2	6.6	39.4	0.2	-1.2
US-52 at South	Lafayette	South	50.7	20.6	39.2	27.9	11.5
US-52 at South	Lafayette	North	44.8	13.9	31.1	15.4	13.7
US-52 at Main	Lafayette	South	41.7	9.8	34.0	16.8	7.7
US-52 at Main	Lafayette	North	41.2	14.0	30.8	17.8	10.4
Union at 18th	Lafayette	West	60.4	11.3	53.7	26.1	6.7
US-31 at Markland	Kokomo	South	51.4	27.9	44.6	40.3	6.8
Kennedy at 169th	Hammond	North	76.7	33.7	69.2	41.3	7.5
Indianapolis at 141st	East Chicago	North	54.0	6.2	38.8	5.3	15.2
Mean			49.4	13.4	39.9	14.4	9.5

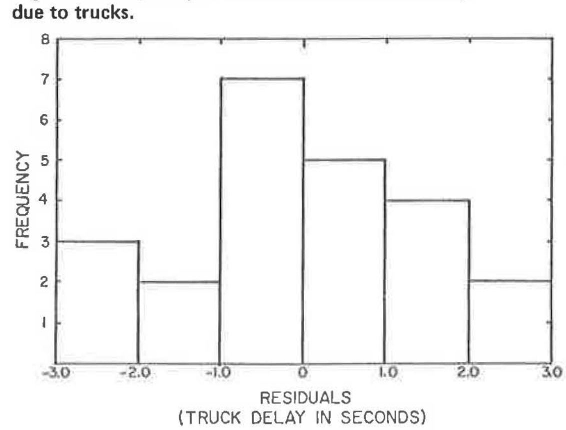
Table 2. Range of variables affecting delay due to commercial vehicles.

Variable Number	Variable	Unit of Measurement	Range
X1	Degree of right turn	Degrees	42 to 135
X2	Green phase length	Seconds	21 to 54
X3	Percentage of right turns		0 to 37.9
X4	Peak-hour load factor	Number of loaded phases/ total phases	0 to 0.95
X5	Right-turn curb radius	Meters	3 to 30
X6	Parking on approach		Yes and no
X7	Peak-hour factor	Vehicles per peak hour/ 4(vehicles per peak 15 min)	0.66 to 0.95
X8	Exclusive left-turn lane		Yes and no
X9	Right-turn lane		Yes and no
X10	Curbing on approach		Yes and no
X11	Percentage of left turns		0.6 to 47.8
X12	Left-turn green phase		Yes and no
X13	Peak-hour approach volume	Vehicles per hour	544 to 1505
X14	Percentage of single units		1.1 to 11.9
X15	Percentage of truck combinations		0 to 12.3
X16	Speed limit on approach	Kilometers per hour	40.2 to 88.5
X17	Approach width	Meters	6 to 15
X18	Metropolitan area population		83 000 to 1 110 000
X19	Percentage of trucks		1.5 to 20.4

Table 3. Significant predictor variables for delay due to trucks.

Variable	Description	r^2 Change
X13	Peak-hour volume in vehicles	0.132
X26	Degree of right-turn × percentage of trucks	0.107
X19	Percentage of trucks	0.099
X8	Left-turn lane (yes = 1 and no = 0)	0.092
X22	Percentage of left-turns × peak-hour volume × left-turn lane × left-turn green phase	0.091
X12	Left-turn green phase in seconds	0.075
X24	Metropolitan area population × speed limit in miles per hour	0.056
X3	Percentage of right turns	0.052
X5	Curb radius in feet	0.049
X9	Right-turn lane (yes = 1 and no = 0)	0.046
X17	Approach width in feet	0.038
X18	Metropolitan area population	0.021
X10	Curbing on approach (yes = 1 and no = 0)	0.014

Figure 1. Frequency of residuals for model of delay due to trucks.



ahead of the point of intersection of the curve tangents. The end reference point was located on the back tangent 18 m (60 ft) back from the point of intersection of the curve tangents. The vehicle was required to be traveling in free flow the entire timing distance. Vehicles were subdivided into passenger cars, single-unit trucks and buses, and truck combinations. Times were taken for each vehicle subclass for each curb radius studied until a good statistical average was obtained. Also measured at each curb radius studied were approach turning width, curbing on approach, and cross-street turning width.

Analysis of Data

The first step was to define the relationship between curb radii and vehicle speeds. Transformations were performed on the predictor variable curb radius, and the stepwise linear regression program (11) was used to determine the best correlation between curb radius and vehicle speeds. The regression line plots resulting from each of the regression equations are shown in Figure 2. The shaded area in Figure 2 represents the 9 to

Figure 2. Right-turn speed as a function of curb radius.

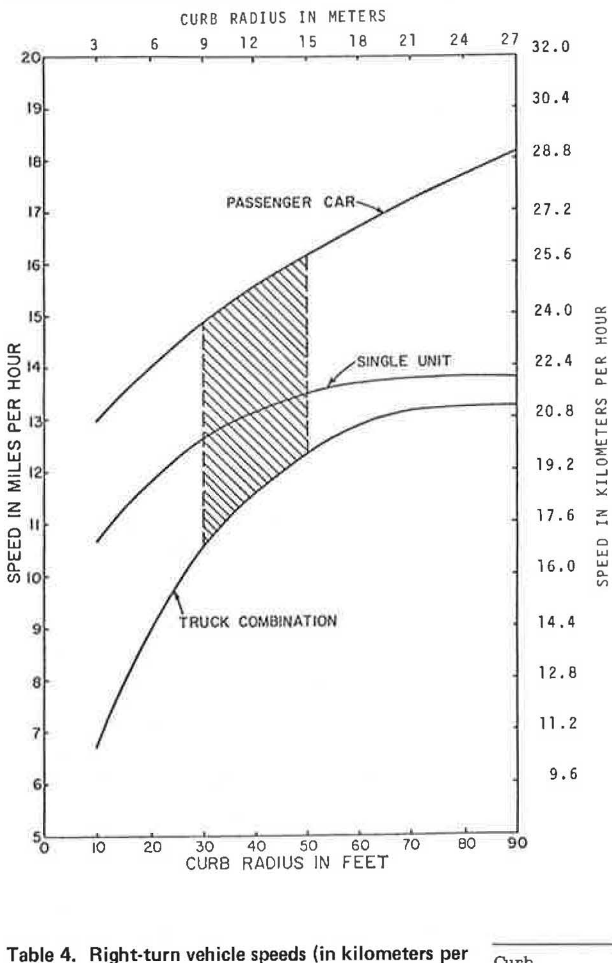


Figure 3. Right-turn speed for passenger cars.

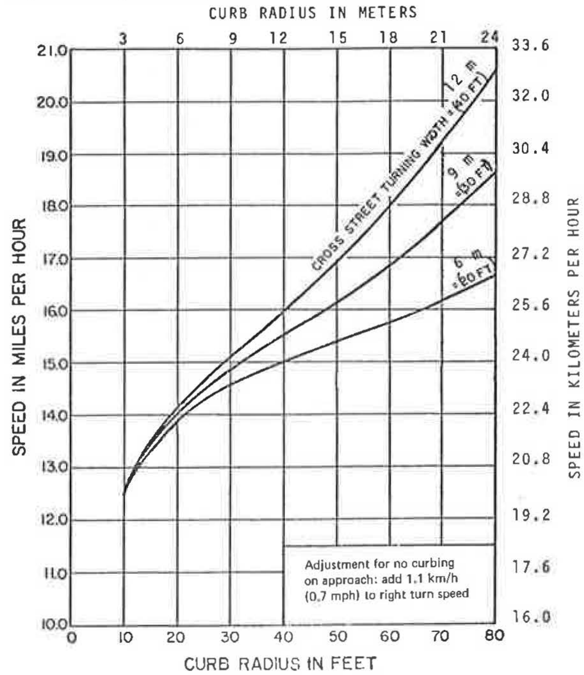


Figure 4. Right-turn speed for truck combinations.

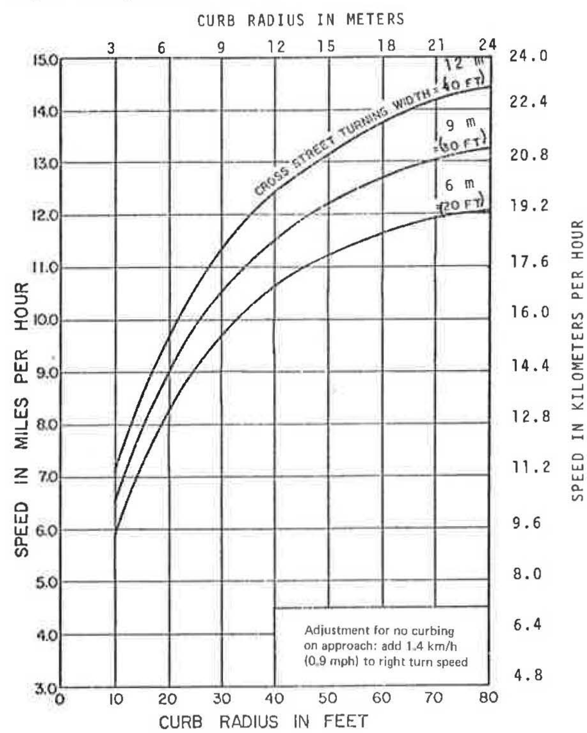
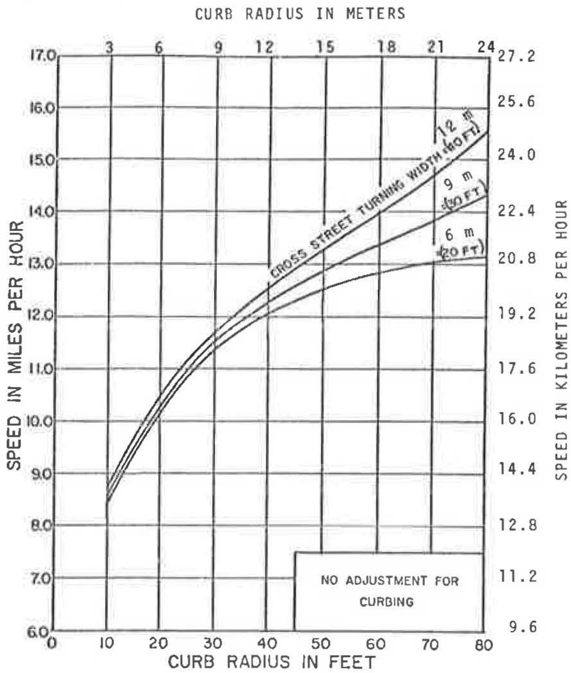


Table 4. Right-turn vehicle speeds (in kilometers per hour) for various curb radii.

Curb Radius (m)	Passenger Car	Single-Unit Truck	Truck Combination	Passenger Car Minus Single-Unit	Passenger Car Minus Truck Combination
9	23.76	20.16	16.86	3.60	6.90
10.5	24.34	20.62	17.78	3.72	6.56
12	24.88	20.99	18.53	3.89	6.35
13.5	25.39	21.31	19.18	4.08	6.21
15	25.87	21.55	19.71	4.32	6.16

Note: 1 m = 3.28 ft; 1 km/h = 0.62 mph.

Figure 5. Right-turn speed for commercial vehicles.



15-m (30 to 50-ft) curb radius that is recommended by the American Association of State Highway and Transportation Officials (1) and by a recent Institute of Traffic Engineers subcommittee report (7) for trucks at intersections of major streets carrying heavy traffic volumes. This 9 to 15-m (30 to 50-ft) range was subdivided into 1.5-m (5-ft) intervals, and the regression equations were used to calculate the resulting vehicle speeds. The results are given in Table 4. The difference between average right-turn speed of a passenger car and that of a single-unit truck was smallest at a 9-m (30-ft) curb radius and largest at a 15-m (50-ft) curb radius. The difference between average right-turn speed of a passenger car and that of a truck combination was smallest at a 15-m (50-ft) curb radius and largest at a 9-m (30-ft) curb radius. From a passenger car delay viewpoint and within AASHTO recommended limits, minimum delay caused by a right-turning single-unit truck is incurred at a curb radius of 9 m (30 ft), and minimum delay caused by a right-turning truck combination is incurred at a curb radius of 15 m (50 ft). Further inspection of Figure 2 reveals that the speeds of single-unit trucks and truck combinations increased very little beyond an 18-m (60-ft) radius. From an 18-m (60-ft) to 27-m (90-ft) curb radius, the increase in speed for a single-unit truck was less than 0.16 km/h (0.1 mph) and for a truck combination was 0.64 km/h (0.4 mph). These small increases in speed do not justify a 9-m (30-ft) increase in curb radius, and result in a maximum desirable curb radius of 18 m (60 ft). Therefore, it is recommended that a 9-m (30-ft) curb radius be used at intersections on major streets when a single-unit truck is used as the design vehicle. At intersections on major streets that use a truck combination as the design vehicle, an 18-m (60-ft) curb radius is recommended. These recommendations apply to intersections located in fringe areas and outlying business districts of metropolitan areas.

To increase the predicting power of the regression equations, we considered three additional variables: approach turning width, curbing on approach, and cross-street turning width. Also, a new regression equation for commercial vehicles was determined. The data for right-turn speeds of single-unit trucks and truck combinations were averaged to obtain speed data for commercial vehicles. The regression equations for passenger car, truck combination, and commercial vehicle right-turn speeds yielded high r^2 and F-values. Figures 3, 4, and 5 show these regression equations.

CONCLUSIONS AND RECOMMENDATIONS

The following general conclusions concerning the effects of commercial vehicles on intersection delay were determined from this research report.

1. The presence of commercial vehicles in a platoon of vehicles approaching a signalized intersection does not significantly increase or decrease the average vehicle stop time at the signalized intersection.
2. The factors or variables that have a significant effect on increasing commercial vehicle delay are peak-hour volume, percentage of commercial vehicles, the presence of a left-turn green phase, the presence of a right-turn-only lane, and approach width. The factors that have a significant effect on reducing commercial vehicle delay are the presence of a left-turn-only lane, percentage of right turns, right-turn curb radius, metropolitan area population, and the presence of curbing on the approach.
3. An analysis of the right-turn study revealed that the maximum right-turn speed for a truck combination at a signalized intersection is approximately 22.4 km/h (14 mph) and approximately 23 km/h (15 mph) for a single-unit truck.
4. The presence of a curb at a signalized intersection approach was found to decrease the right-turn speed of passenger cars by 1.12 km/h (0.7 mph) and to decrease the right-turn speed of a truck combination by 1.44 km/h (0.9 mph).

Based on this research, the following recommendations are made.

1. The right-turn speeds of passenger cars, truck combinations, and commercial vehicles as shown in Figures 3, 4, and 5 are recommended for application in corner radius design, delay, and capacity analysis.
2. A 9-m (30-ft) radius caused the least delay for a passenger car following a single-unit truck, and a 15-m (50-ft) radius caused the least delay for a passenger car following a truck combination. Also, corner radii greater than 18 m (60 ft) did not appreciably increase right-turn speeds of single-unit trucks and truck combinations. Therefore, it is recommended that a 9-m (30-ft) corner radius be used at intersections on major streets that use a single-unit truck as the design vehicle. On major streets at intersections that use a truck combination as the design vehicle, an 18-m (60-ft) corner radius is recommended where economically feasible.

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Evaluation of Factors Influencing Driveway Accidents

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Full control of access will obviously not lower the number of accidents that occur on urban arterial highways. Both land access and traffic movement must be allowed on this type of facility, so the causes of resulting traffic accidents must be identified, and deficiencies must be corrected. The literature contains much on intersection accidents, but relatively little has been written to identify the major causes of driveway-related traffic accidents, which account for almost 14 percent of total arterial highway traffic accidents. This paper identifies some of the characteristics of driveway accidents and relates driveway accident occurrence to various physical and environmental features of the roadway and traffic characteristics. Through statistical analysis, it is shown that the driveway accident rate tends to decrease as the spacing between two driveways and the spacing between a driveway and an adjacent intersection increase. Multiple regression analysis was used to develop a series of mathematical models relating the driveway accident rate to the physical and environmental features of the roadway and traffic characteristics. This procedure reveals that the driveway accident rate decreases as the number of commercial driveways per kilometer decreases, as the number of through-traffic lanes decreases, as the number of total intersections per kilometer increases, as the number of total driveways per kilometer decreases, or as the traffic volume on the arterial highway decreases. The results of this study provide the engineer or public official with tools to better identify the circumstances related to driveway accidents, to predict driveway accident rates, and to estimate the effectiveness of measures to reduce such accidents.

Reducing traffic accidents is and always will be one of the primary objectives of the highway engineer. The introduction of full control of access, which has been hailed as the most significant factor in accident reduction developed thus far (2), was designed to meet that objective. However, full control of access cannot be used as a sole solution to the accident problem because a complete highway system must provide both land access and traffic movement (4). The accident problem is further complicated on those facilities where these two functions must be simultaneously coordinated without delegating an advantage to either. A case in point is the urban arterial highway, which must encourage efficient through-traffic movement and provide access for abutting landowners.

Landowners access the arterial highway by means of driveways, each of which introduces an additional conflict to through traffic. As the number of such conflict points along an arterial highway increases, the opportunity for driveway-related traffic accidents increases. A driveway accident is a traffic accident in which at least one of the participants was moving to or from a driveway at the time of the accident or an accident resulting from such a movement.

The number of a particular type of accident on an arterial highway can be reduced only after the major factors contributing to its occurrence have been identified. Many studies have been conducted to mathematically identify causative factors of accidents at intersections, but relatively little has been written to identify the major contributors to driveway accidents. Driveway accidents represent a significant percentage of total arterial highway traffic accidents, as revealed in a recent study made in Skokie, Illinois, which found that driveway accidents composed 12 percent of that city's major street accidents (1). Because there is little reason to doubt that this figure is representative of urban arterial highway accident experience throughout the country, research directed toward a better understanding of the factors causing driveway accidents on urban arterial highways is of obvious benefit.

This research was developed to provide a means for improving overall highway safety and to expand on the limited existing data on driveway accidents. A literature search disclosed numerous voids and some conflicts of opinion on the subjects of driveways and driveway accidents (3). Given these observations, the following research objectives were developed:

1. Identify and evaluate characteristics of driveway accidents;
2. Relate driveway accident rates to the average spacing on a section of roadway between adjacent driveways;
3. Relate driveway accident rates to the average spacing on a section of roadway between a driveway and an adjacent intersection leg; and

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4. Relate driveway accident rates to characteristics of the roadway and its abutting environment and to traffic characteristics.

DATA COLLECTION TECHNIQUES

Relevant data obtained from 100 sections of urban arterial highway were analyzed. Ten roadway sections were taken from each of 10 central Indiana cities whose population exceeded 30 000. Three specific types of roadway data were collected: physical roadway, traffic volume, and traffic accident data.

Data on roadway characteristics were obtained by traveling to each site and inventorying all existing physical features of each roadway. A measuring wheel was used on both sides of every section to obtain an accurate measurement of all access and intersection spacing details. However, because many factors could conceivably influence the driveway accident rate, homogeneity with respect to certain variables throughout the length of each study section was mandatory. Therefore, sections selected for study had to meet the following criteria:

1. Curb parking characteristics must remain constant;
2. Curb-to-curb street width must remain constant;
3. No type of median divider can be present;
4. No major changes in traffic volume may occur between the termini of each section;
5. No major construction must have occurred on the section or on land abutting the section later than 1 year before the study (1968); and
6. Each section selected for study must be located outside the limits of the central business district but within the city limits.

Driveway types were classified into four categories: residential, commercial, industrial, and other. The first three classes refer to the principal land use served by the driveway; the fourth includes driveways to land uses other than the first three categories, such as fire houses, schools, and churches.

Most of the traffic volume data were obtained from state and local highway and planning officials. However, 19 percent of the data was collected by placing traffic counters at representative locations within each section. In all cases, however, pertinent traffic volume expansion factors and a 4 percent annual increase in traffic volume were used to obtain the average daily traffic (ADT) volume for each roadway section for each required study year.

Traffic accident data were collected for each of the 100 roadway sections for the period January 1, 1968, to December 31, 1971, from the standard accident report form as filed by the investigating police officer. Pertinent data on each driveway accident were transferred from the report to a preprepared form.

The methods of analysis determined how the data would be refined for study. To identify driveway accident characteristics, the first objective of this study, required the entire 4-year accident history of each roadway section. No special treatment of the data was necessary. The next three objectives, however, required the development of accident rates to be mathematically related in multiple regression analysis to particular characteristics of the study sections. Two accident rates, accidents/1.6 km/year and accidents/160 million vehicle-km, were developed (the latter rate was discarded early in the analysis because it did not relate so well to the roadway characteristics as did the accidents/1.6 km/year). A 3-year annual average of the 1968-1970 accident data was used to develop the accident rates used in multiple regression analysis. A chi-square goodness-of-fit test was applied to data from each of the 100 sections to test the hypothesis that the 3 years of accident data originated from the same population. Inasmuch as all but eight of the sections passed the test and it was determined in initial analyses that more statistically reliable results could be obtained without those eight sections, only 92 roadway sections were selected for analysis and used for the latter three study objectives. The accident rates used in testing the resulting regression equations were developed from 1971 accident data.

ANALYSIS OF DATA

Driveway Accident Characteristics

All 100 Roadway Sections

The 4-year accident history of 100 central Indiana urban arterial highway sections totaling 96.853 km (60.436 miles) in length revealed a total of 1212 driveway accidents. This represented 13.95 percent of all reported traffic accidents on these same roadway sections.

The following results were obtained when similar characteristics of each of the 1212 driveway accidents were grouped together:

1. The fewest number of driveway accidents occurred on Sunday when traffic volumes are lowest and when most business establishments are closed. A higher number was experienced on Friday and Saturday when traffic volumes are heavier and when more trips to commercial establishments, on the average, occur. The following figures show the number of driveway accidents and the percentage of the total number of driveway accidents that occurred each day.

Day	Number	Percentage of Total	Day	Number	Percentage of Total
Sunday	92	7.59	Thursday	188	15.51
Monday	166	13.70	Friday	255	21.04
Tuesday	146	12.05	Saturday	233	19.22
Wednesday	132	10.89			

2. Of all driveway accidents, 71.62 percent involved a maneuver into or from a commercial establishment.
3. Most of the driveway accidents (85.56 percent) resulted in property damage only; the remainder (14.44 percent) involved personal injury. None of the reported accidents resulted in a fatality.
4. Vehicles turning left into or from driveways were involved in 64.60 percent of all driveway accidents, and 76.00 percent of all driveway accidents resulting in personal injury involved a left turn maneuver.
5. A vehicle entering a driveway was involved in 53.47 percent of the driveway accidents; the remainder involved an exit maneuver.
6. Right-angle collisions made up 60.07 percent of the driveway accidents; rear-end collisions made up 33.09 percent. A majority of the rear-end collisions occurred when the driveway vehicle was struck while waiting to turn into a driveway.
7. Driveway vehicles were struck by through-traffic vehicles in 57.01 percent of the cases, whereas they struck the through-traffic vehicles in 33.34 percent of the cases. The driveway vehicle was not directly involved in the collision in the remainder of the driveway accidents studied.
8. Of all driveway accidents, 72.28 percent occurred during daylight hours when traffic volume is heaviest.
9. Seventy-five percent of the driveway accidents

occurred during periods of nonprecipitation, and 70.05 percent occurred under dry pavement conditions. These results probably reflect the low number of days annually on which weather is inclement.

Significant Data Splits

In this analysis, the data were split into two or more logical categories, and the differences between the groups were compared. Three significant data splits were analyzed in this phase of the study.

The sample consisted of 29 one-way streets and 71 two-way streets. During a 4-year period, two-way streets experienced, on the average, almost 2.75 times the number of driveway accidents per kilometer as did one-way streets (Table 1). However, associated with this statistic is the fact that the one-way streets in this sample had a lower ADT and fewer commercial driveways per kilometer, both of which may explain this difference.

The entire sample of 100 arterial street sections had from one lane to four lanes. Data given in Table 2 show that, as the number of through-traffic lanes increases on the average, the number of driveway accidents per kilometer, the number of commercial driveways per kilometer, and the ADT increase, indicating that different combinations of these variables could have a significant effect on the driveway accident rate.

Although a consistent relationship does not occur, there is a definite trend toward more driveway accidents per kilometer and more commercial driveways per kilometer in higher ADT ranges (Table 3). The ADT ranges into which the sections were categorized were determined by plotting the number of driveway accidents per kilometer against ADT and selecting definite clusters of points as intervals.

Driveway Spacing Analysis

The literature recommends longer distances between adjacent driveways and between driveways and adjacent intersection legs, but these conclusions are based on criteria other than the driveway accident rate. To determine the relationships between driveway spacing and the driveway accident rate, scaled maps were reproduced for each of the 92 roadway test sections from the measurements obtained in the earlier field inventories. Two average driveway spacing variables were developed for each roadway section. The average spacing between two adjacent driveways was defined as the sum of the centerline-to-centerline distance between two adjacent driveways divided by the number of times in a section two driveways appeared next to each other. Likewise, the average spacing between a driveway and an adjacent intersection leg was defined as the sum of the centerline-to-centerline distance between a driveway and an adjacent intersection leg divided by the number of times in each section that a driveway appeared adjacent to an intersection leg. Both average spacing variables were developed by considering driveways on both sides of the street.

The technique of analysis was to plot each average spacing variable against the corresponding driveway accident rate of each roadway section. Inasmuch as both plots displayed such a scatter of points that useful, significant information could not be obtained, a least squares technique was used to fit the best possible straight line through the points. The following two regression equations were developed:

$$Y = 7.728 - 0.055\,X_1 \tag{1}$$

$$Y = 11.584 - 0.068\,X_2 \tag{2}$$

where

Y = number of driveway accidents per mile per year,
X_1 = average spacing in feet over a section of roadway between adjacent driveways, and
X_2 = average spacing in feet over a section of roadway between a driveway and an adjacent intersection leg.

(Both equations were developed from an array of data defined in U.S. customary units; therefore, SI units are not given for the variables in these models.)

The correlation between Y and X_1 was -0.166, and the correlation between Y and X_2 was -0.318, indicating that neither of the two driveway spacing variables is related linearly to the driveway accident rate. However, the negative sign preceding the coefficients of the independent variables and the negative sign preceding each of the correlation coefficients suggest a trend toward lower driveway accident rates as driveways are located fur-

Table 1. Driveway accident characteristics on one-way and two-way streets.

Item	One-Way Streets	Two-Way Streets
Number of roadway sections	29	71
Driveway accidents per kilometer	14.816	5.444
Driveway accidents as percentage of total	6.37	16.35
Average ADT, vehicles per day	7582	9905
Commercial driveways per kilometer	10.067	13.248

Note: 1 km = 0.62 mile.

Table 2. Driveway accidents per kilometer, commercial driveways per kilometer, and ADT as a function of the number of through-traffic lanes in both directions.

Number of Lanes	Number of Section Samples	Driveway Accidents per Kilometer	Commercial Driveways per Kilometer	ADT
1	3	0.086	4.711	1 713
2	74	7.464	9.804	7 843
3	5	9.490	17.471	11 648
4	18	31.812	21.373	15 522

Note: 1 km = 0.62 mile.

Table 3. Driveway accidents per kilometer and commercial driveways per kilometer as a function of ADT.

ADT Range	Number of Sections: Total	One Lane	Two Lanes	Three Lanes	Four Lanes	Driveway Accidents per Kilometer	Commercial Driveways per Kilometer
0 to 5000	22	3	19	0	0	2.451	3.088
5001 to 6800	16	0	15	1	0	4.381	9.461
6801 to 8800	17	0	15	1	1	8.347	10.505
8801 to 10 200	7	0	7	0	0	7.236	8.370
10 201 to 11 900	8	0	5	1	2	10.978	11.094
11 901 to 14 600	13	0	6	1	6	20.126	15.357
More than 14 600	17	0	7	1	9	30.347	26.467

Note: 1 km = 0.62 mile.

ther from other driveways or intersection legs. An increase in either spacing variable implies a decrease in the number of driveways on a section of roadway. This would surely contribute to a decrease in the driveway accident rate.

Relationship Among Driveway Accidents, Roadway Characteristics, and Traffic Volume Characteristics

Analysis Technique

One method for establishing which factors have the greatest effect on the driveway accident rate and their relative order of importance is stepwise linear regression analysis. This technique involves defining a dependent variable, the number of driveway accidents per mile per year, and the number of independent variables that are suspected to have an influence on the dependent variable. This analysis considered a total of 26 independent variables representing the 24 most important and most logical roadway, environmental, and traffic volume factors. Because stepwise multiple regression analysis works by scanning an array of independent variables and choosing in succession those most closely related to the dependent variable, as many independent variables as possible were provided. The independent and dependent variables, as they were coded for computer analysis, are given in Table 4. Curb parking restrictions are further denoted as follows:

X_{23}	X_{24}	Definition
0	0	No parking on both sides of street
1	0	Parking on one side of street only
0	1	Parking on both sides of street

Curb conditions are further denoted as follows:

X_{25}	X_{26}	Definition
0	0	No curbs on both sides of street
1	0	Curbs on one side of street only
0	1	Curbs on both sides of street

Stepwise multiple regression analysis was used to develop equations for combinations of the study sections. One regression equation was developed for each of the following categories: all 92 study sections, all one-way street sections, all two-way street sections, and all two-lane street sections. Four equations were developed by using the same categories of data with the sections from Indianapolis removed. This was done to test the effect of urban area population on the driveway accident rate; nine of the urban areas had populations between 30 000 and 80 000 whereas the population of Indianapolis was 750 000.

The eight regression equations were developed in a three-step process. In the first step, the data were subjected to stepwise regression analysis. Only linear independent variables that effected a significant increase in the multiple correlation coefficient (r^2) were used in the equation. In the second step, a stepwise multiple linear regression was again used, but the independent variables were all of the significant linear terms from the first step and all possible two-way products of these linear terms. These two-way products represent interactions between two independent variables, and these products proved in every case to be more significant than the sum of their component variables. Once again, only those terms that contributed to the increase in the multiple correlation coefficient were used in the equation. Because a model that contains interaction terms must also contain the main effect terms that make up the interaction and because in some cases one or more of the main effect terms were not significant enough to enter the equation in the second step, the third step was introduced to force these main effect terms into the final equations.

Regression Equations

Eight regression equations relating the driveway accident rate to significant roadway and environmental characteristics were developed. They are given below. Based on data from all 92 study sections,

$$\begin{aligned}Y = &-7.067 + 0.300(X_1) + 15.550(X_2) + 2.250(X_6) + 0.636(X_{13})\\ &+ 0.075(X_{16}) + 0.024(X_{19}) + 0.024(X_6)(X_{16}) - 0.372(X_6)(X_{13})\\ &+ 0.280(X_2)(X_{19}) - 0.009(X_1)(X_{19}) - 0.010(X_{13})(X_{16})\\ &+ 0.067(X_1)(X_{13}) - 8.067(X_1)(X_2) + 0.461(X_6)^2\\ &+ 0.928(X_2)(X_{16})\end{aligned} \tag{3}$$

The definitions of the variables are given in Table 4. Based on data from all 92 study sections except Indianapolis,

$$\begin{aligned}Y = &+0.130 - 4.583(X_2) - 0.494(X_6) + 0.764(X_{13}) + 0.178(X_{16})\\ &- 0.130(X_{20}) + 0.079(X_2)(X_{16}) - 0.009(X_{13})(X_{16})\\ &+ 0.082(X_6)(X_{20}) + 320.927(X_2)^2 - 0.448(X_6)(X_{13})\end{aligned} \tag{4}$$

Based on data from all two-lane study sections,

$$\begin{aligned}Y = &+0.170 + 0.010(X_1) + 20.034(X_2) + 0.014(X_{13}) + 0.111(X_{16})\\ &+ 1.413(X_2)(X_{16}) - 0.011(X_{13})(X_{16}) - 0.030(X_1)(X_{16})\end{aligned} \tag{5}$$

Based on data from all two-lane study sections except Indianapolis,

$$\begin{aligned}Y = &-2.211 + 69.795(X_2) + 0.191(X_{13}) + 0.021(X_{14}) + 0.026(X_{16})\\ &+ 1.609(X_2)(X_{16}) - 0.009(X_{13})(X_{16}) + 0.003(X_{14})(X_{16})\\ &- 3.978(X_2)(X_{13})\end{aligned} \tag{6}$$

Based on data from all one-way street study sections,

$$\begin{aligned}Y = &-1.592 + 8.996(X_2) + 0.179(X_{18}) - 0.006(X_{19}) + 0.970(X_{24})\\ &+ 1.096(X_2)(X_{19}) - 32.035(X_2)(X_{24})\end{aligned} \tag{7}$$

Based on data from all one-way street study sections except Indianapolis,

$$\begin{aligned}Y = &-2.333 + 25.728(X_2) - 0.428(X_7) + 0.378(X_{13}) + 0.031(X_{19})\\ &+ 1.020(X_2)(X_{19}) - 0.032(X_{13})^2 - 0.028(X_7)(X_{19})\end{aligned} \tag{8}$$

Based on data from all two-way street study sections,

$$\begin{aligned}Y = &+21.425 + 0.041(X_1) - 11.070(X_6) + 0.216(X_9) - 0.378(X_{13})\\ &+ 0.043(X_{16}) - 0.041(X_{17}) - 0.053(X_{21}) + 0.060(X_6)(X_{16})\\ &- 0.001(X_{13})(X_{21}) - 0.015(X_{16})(X_{17}) - 1.379(X_6)(X_9)\\ &- 0.022(X_1)(X_{16}) + 0.019(X_9)(X_{21}) + 2.475(X_6)^2\\ &+ 0.119(X_9)(X_{13}) + 0.029(X_9)(X_{16})\end{aligned} \tag{9}$$

Based on data from all two-way street study sections except Indianapolis,

$$\begin{aligned}Y = &+0.098 + 23.967(X_2) + 1.513(X_6) + 0.225(X_{13}) + 0.167(X_{16})\\ &- 0.004(X_{21}) + 0.995(X_2)(X_{16}) - 0.016(X_{13})(X_{16})\\ &+ 0.014(X_6)(X_{16}) - 0.010(X_{13})(X_{21})\end{aligned} \tag{10}$$

Again, these equations were developed from a large array of data, most of which was defined and coded for computer analysis in U.S. customary units. The statistical complexity of the equations makes it difficult to merely apply the conversion factors directly to the coefficients of the appropriate terms to devise equations for the number of driveway accidents per kilometer. However, an increase or decrease in a certain property per mile implies an increase or decrease in that same property per kilometer. This accounts for the use of metric units throughout this paper in spite of the fact that the results were originally obtained in U.S. customary units.

Evaluating the Regression Equations

The most obvious feature of the eight regression equations is their relatively high multiple correlation coefficients, which are given below.

Equation	r^2	Equation	r^2
3	0.85	7	0.86
4	0.86	8	0.87
5	0.71	9	0.84
6	0.78	10	0.82

Table 4. List of variables.

Index	Description
Y	Driveway accidents per mile per year
X_1	1970 urban area population in hundred thousands
X_2	1969 average daily traffic volume in hundred thousands
X_3	Street type; $X_3 = 0$ for one-way streets; $X_3 = 1$ for two-way streets
X_4	Roadway section speed limit (mph)
X_5	Curb-to-curb street width (ft)
X_6	Number of through-traffic lanes
X_7	Lane markings; $X_7 = 0$ for no lane markings; $X_7 = 1$ for lane markings visible
X_8	Number of stop signs and red flashing traffic signals per mile
X_9	Number of traffic signals per mile
X_{10}	Number of yield signs and yellow flashing traffic signals per mile
X_{11}	Number of 3-way intersections per mile
X_{12}	Number of 4-way intersections per mile
X_{13}	Number of total intersections per mile
X_{14}	Number of alleys per mile
X_{15}	Number of residential driveways per mile
X_{16}	Number of commercial driveways per mile
X_{17}	Number of industrial driveways per mile
X_{18}	Number of other driveways per mile
X_{19}	Number of total driveways per mile
X_{20}	Number of friction points per mile
X_{21}	Average spacing between adjacent driveways (ft)
X_{22}	Average spacing between driveways and adjacent intersection legs (ft)
X_{23}, X_{24}	Curb parking restrictions
X_{25}, X_{26}	Curb condition

Introducing cross products into the models added considerably to the numerical value of r^2.

Without a doubt, the variable having the most significant effect effect on the driveway accident rate was the number of commercial driveways per kilometer. Only in the one-way street analysis did this variable prove to be insignificant. This is probably due to the low number of one-way streets and the lack of substantial commercial development fronting those one-way street sections used in this analysis. Further computations using the models indicated that each commercial driveway to an arterial street adds between 0.1 and 0.5 driveway accident/mile/year (0.6 and 3.1 accidents/km), depending primarily on the ADT and the number of traffic lanes on the arterial. Other independent variables that seem to have an important effect on the driveway accident rate are the number of through-traffic lanes, the arterial highway ADT, and the number of total intersections per kilometer. Computations revealing the mathematical sign preceding each of these significant variables indicated that the driveway accident rate increases as the number of commercial driveways per kilometer increases, as the urban arterial ADT increases, as the number of traffic lanes increases, and as the number of total intersections per kilometer decreases. Only minor deviations attributable to variable interactions were evident in the analysis. These results are not only inherent from the models, but exactly as one would expect in a real situation. It is significant to note that the number of residential driveways per kilometer is related in no way to the driveway accident rate.

The effect of urban area population was significant. This variable entered the regression equation as a moderately significant predictor of the dependent variable when the Indianapolis sections were included in the analysis, but it had no significance whatsoever when these sections were omitted. In addition, in most cases, the arterial highway ADT was more significant as an independent variable when the Indianapolis sections were not included in the analysis.

One of the major findings of this study was that the product of two independent variables was superior to the sum of the same two variables as a predictor of the dependent variable. These products represent interactions between variables, and their possible use in this analysis was first brought to light during the discussion on driveway accident characteristics. The two most significant interactions in this study were those between the number of through-traffic lanes and the number of commercial driveways per kilometer and between the ADT and the

Table 5. Range of significant variables.

Sample	Variable	Index	Maximum	Minimum	Range
92 sections	Driveway accidents per kilometer per year	Y	17.0	0	17.0
	Urban area population	100 000(X_1)	744 624	31 403	713 221
	ADT	100 000(X_2)	31 034	1153	29 881
	Number of traffic lanes	X_6	4	1	3
	Lane markings	X_7	1	0	1
	Traffic signals per kilometer	X_9	7.3	0	7.3
	Total intersections per kilometer	X_{13}	14.7	1.7	13.0
	Alleys per kilometer	X_{14}	21.9	0	21.9
	Commercial driveways per kilometer	X_{16}	45.5	0	45.5
	Industrial driveways per kilometer	X_{17}	16.7	0	16.7
	Other driveways per kilometer	X_{18}	13.6	0	13.6
	Total driveways per kilometer	X_{19}	74.1	16.1	58.0
	Friction points per kilometer	X_{20}	81.0	22.0	59
	Driveway-driveway spacing (meters)	X_{21}	40.6	7.7	32.9
	Parking	X_{24}	1	0	1
64 two-way sections	Driveway accidents per kilometer per year	Y	17.0	0	17.0
	Urban area population	100 000(X_1)	744 624	31 403	713 221
	Number of traffic lanes	X_6	4	2	2
	Traffic signals per kilometer	X_9	5	0	5
	Total intersections per kilometer	X_{13}	12.9	1.7	11.2
	Commercial driveways per kilometer	X_{16}	45.5	0	45.5
	Industrial driveways per kilometer	X_{17}	16.7	0	16.7
	Driveway-driveway spacing (meters)	X_{21}	40.1	8.7	31.4

Note: 1 km = 0.62 mile; 1 m = 3.28 ft.

number of commercial driveways per kilometer. In the first case, more commercial driveways per kilometer are generally found on highways with more traffic lanes. As both of these variables increase in numerical value, so does the driveway accident rate, on the average. The same analogy can be found in the second case. That is, the two variables seem to increase or decrease in value at approximately the same rate as the dependent variable; thus, the interaction is represented by the product of the two terms.

At first glance, any one of the eight equations may seem difficult to use. However, an engineer can quickly determine the driveway accident rate of a particular stretch of roadway by knowing a maximum of only seven roadway factors and applying them in proper sequence in the appropriate regression equation. Each equation should produce reasonably reliable results, as long as the data entered into the equation are within the range used to develop each. The range of each variable, however, varies in each of the eight equations because of the different source of data from which each was developed. For purposes of comparison and general information, the ranges of significant variables associated with equation 3, which was developed from the data of all 92 roadway sections, and with equation 9, which was developed from the data of all 64 two-way street sections, are given in Table 5. Table 5 also indicates the range of all other variables that were significant in at least one of the other equations. In most cases, as is evident by comparing the data, the range of a certain variable in a given equation will be less than that given in Table 5. These equations can be used not only to provide a reasonable estimate of the driveway accident rate on a particular segment of undivided arterial highway, but also to indicate how much of a change is required in one or more roadway characteristics to effect a desired change in the driveway accident rate.

Testing the Models

The eight models were tested by using each to predict the driveway accident rate that would occur on a particular study section in 1971 and by comparing those results with the actual 1971 driveway accident rate. These two figures were compiled for each study section, and the individual differences between the actual and the predicted driveway accident rates were used to develop a multiple correlation coefficient.

When these eight multiple correlation coefficients were compared to those associated with each respective model, differences were obvious and excessive. In addition, the differences between the actual and predicted driveway accident rates indicated extreme variance. For example, the regression equation developed on the basis of data from all 92 study sections explained 85 percent of the variation of the averaged 1968 to 1970 driveway accident rate, but this same equation explained only 53 percent of the variation in the 1971 driveway accident rate. Likewise, the accompanying residuals ranged in value from 19.8 to -13.4 for a range of 33.2.

The results are not an indication of unreliability in the predictive capacities of the models. Rather, they emphasize the importance of the major controls incorporated throughout this study. In all cases, models developed on the basis of the annual average of a 3-year accident history (1968 to 1970) were used to predict a 1-year accident history. It is possible, in some cases, that the 1971 driveway accident rate for a given section does not agree statistically with its corresponding 1968 to 1970 averaged driveway accident rate. In fact, preliminary computations indicated a higher value of r^2 and a smaller range of residuals when sections having obvious discrepancies between the two driveway accident rates were omitted from this phase of the analysis. It is obvious from the study that the models will predict a 3-year annual average driveway accident rate and not a driveway accident rate based on a 1-year accident history.

CONCLUSIONS

The following conclusions concerning driveways and driveway accidents on urban arterial highways in central Indiana are presented.

1. Driveway accidents represent a significant percentage of the total traffic accident experience on urban arterial highways, and steps taken to effect their decrease would improve overall highway safety.
2. Public officials should consider measures such as barrier medians, traffic signals, left turn lanes, and left turn prohibitions at certain driveways as a means to effect a reduction in driveway accidents and in personal injuries resulting from such accidents.
3. The driveway accident rate tends to decrease when the average spacing over a section of arterial highway between adjacent driveways and between a driveway and an adjacent intersection leg increases.
4. Certain roadway and environmental factors and traffic volume characteristics can be used to predict the annual number of driveway accidents per mile significantly better than they can predict the number of driveway accidents per 100 million vehicle-miles.
5. The interaction, or product, of two variables proved to be more significant, in every case, than the sum of the same two variables in predicting the number of driveway accidents per mile per year.
6. Driveway accident rates based on the annual average of a 3-year accident history produce better results in regression analysis when it can be shown that the 3 separate years of accidents used to derive the rates originated from the same population.
7. The number of driveway accidents per kilometer per year will decrease when (a) the number of commercial driveways per kilometer is reduced, (b) the number of through-traffic lanes is reduced, (c) the number of total intersections per kilometer is increased, (d) the number of total driveways per kilometer is reduced, or (e) the arterial highway ADT is reduced. Other factors were shown to have a less pronounced effect on the driveway accident rate.
8. Urban area population can be used as a significant predictor of the driveway accident rate when the study samples are derived from urban centers whose population differences are significantly large. However, the effectiveness of this variable as a predictor decreases rapidly as this difference becomes less pronounced. This may be due to the fact that motorists from larger cities are more accustomed to traveling on urban arterial highways.
9. Mathematical models describing the driveway accident rate on one-way streets, two-way streets, and two-lane streets are not statistically or analytically different from the model describing all study sections.
10. These mathematical models can be used not only to predict a future driveway accident rate but also to present facts defending decisions on controlling the number of access points for the public well-being. Used within the constraints from which they were developed, the models can be valuable tools to all public officials concerned with the number of driveway accidents in their cities.

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Effect of Bridge Shoulder Width on Traffic Operational Characteristics

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In 1970, West Virginia University, in cooperation with the West Virginia Department of Highways and the Federal Highway Administration, began a study to develop analytical techniques for determining the best shoulder and curb width on long-span bridge structures from the standpoints of safety and cost. The major objective of this research was to determine whether providing full-width shoulders across long-span bridge structures would improve traffic and safety. Completed in 1975, the study included a structural cost analysis, accident record analysis, a controlled laboratory study, and a before and after field study to relate the results of the laboratory study to actual field conditions. The structural cost analysis study revealed that the additional cost for widening would be about 3 percent of total bridge cost per 0.3 m (1 ft) of bridge width. The accident study revealed no strong relationship between bridge shoulder width and accidents although the laboratory study showed that erratic behavior of drivers is at a minimum for a bridge shoulder width of 1.8 m (6 ft).

The before and after field study, which is reported here, was carried out in two major stages. The first stage consisted of studying the effect of various bridge shoulder curb widths on the operational characteristics of vehicles on the bridge. The second stage consisted of making these same studies on the effects of various bridge shoulder curb widths with a guardrail type of barrier flush with the face of the curb and offset 0.6 m (2 ft) from the face of the curb.

The before condition consisted of determining the speed and lateral placement, as measured from the edge of the roadway shoulder, of vehicles in the vicinity of the bridge. The lateral placement of the vehicle is the distance of the right front wheel from the edge of the roadway at the point where the shoulder begins. After the observations for the before period were made, the physical characteristics of the bridge were altered to simulate the effect of 0.6, 1.2, 1.8, and 2.4-m (2, 4, 6, and 8-ft) curbs alone and with guardrail both flush with the face of the curb and offset 0.6 m (2 ft) from the face of the curb. This resulted in a minimum of 11 conditions in the after period.

Although the basic study was concerned with curbs as wide as 2.4 m (8 ft), this study was confined to investigating the effect of 0.6, 1.2, and 1.8-m (2, 4, and 6-ft) curbs on a bridge with 3-m (10-ft) shoulders.

The speed and lateral placement of automobiles only were collected at six locations on and in the vicinity of the study bridge under conditions of no curb and 0.6, 1.2, and 1.8-m (2, 4, and 6-ft) curbs. These various curb widths resulted in effective bridge shoulder widths of 3, 2.4, 1.8, and 1.2 m (10, 8, 6, and 4 ft). Data were collected on the speed and lateral placement of at least 200 free-flowing automobiles for each of the test conditions at six locations for a minimum of 2 days under each condition.

The site selected for study is located on Interstate 79, approximately 32 km (20 miles) south of Morgantown and just east of Fairmont, West Virginia. The nearest on-ramp is approximately 1.6 km (1 mile) upstream and the nearest off-ramp is approximately 0.8 km (½ mile) downstream. The roadway throughout the test section has four 3.6-m (12-ft) lanes, divided by a grass median. The study site is located on the southbound lane with a 5 percent downgrade.

The study bridge, which is 67 m (220 ft) long, is slightly skewed to the right and has W-beam guardrails on both shoulders on the upstream and on the right shoulder on the downstream side. This particular site was selected because it was built to AASHTO-recommended design standards, and it also possessed other qualities desirable for obtaining data.

The proper measurement of the speed and placement variables was considered to be critical to the successful completion of this study. Measurements of vehicle speed and placement were made by using a tape switch installed 305 and 152 m (1000 and 500 ft) upstream of the bridge, 152 m (500 ft) downstream, at the upstream and downstream ends of the bridge, and in the middle of the bridge. Data were collected for a 4-day period with no modifications to the bridge so as to establish the before condition. After the base data were collected, the curb

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was simulated by fabricating a wood curb to resemble the actual concrete curb as nearly as possible. The curbing was fabricated in 0.6-m (2-ft) sections so that it could be used to simulate 0.6, 1.2, and 1.8-m (2, 4, and 6-ft) curb conditions. Data were collected for each of the conditions after the traffic had adjusted to the new situation.

To evaluate the effect of position and curbing conditions on vehicle speeds and placements, a fixed effects analysis of variance model was formulated since only discrete levels of each factor were to be analyzed. The model was tested to determine whether there were significant differences between levels of curb conditions or levels of positions and for an interaction effect between the levels of curb conditions and positions.

The conclusions drawn from this study were, like any research effort, necessarily limited by many factors. As many of the confounding factors as possible were controlled or eliminated where possible so that more reliable conclusions could be reached regarding those variables of interest.

The following conclusions were reached after the results of the study were evaluated.

1. Vehicle speed and placement data may be combined for different days of the week without any major loss of information.
2. Relative location had a significant effect on speeds as the vehicles moved through the test section. Average speeds increased from 96.88 km/h (60.20 mph) 305 m (1000 ft) upstream to 101.0 km/h (62.78 mph) 152 m (500 ft) downstream, a difference of 4.12 km/h (2.58 mph). It was concluded that the increase in speed was probably due to the 5 percent downgrade throughout the test section.
3. All curb conditions had a significant effect on vehicular speeds in that the speeds with curb in place were significantly lower than those with the base condition of no curb. The lowest average speed of 97.73 km/h (60.73 mph) occurred with 1.2-m (4-ft) curbs; the speed with no curbing was 100.36 km/h (62.36 mph), a difference of 2.62 km/h (1.63 mph).
4. There is a significant interaction between positions and conditions for vehicle placements, which leads to the conclusion that some positions and conditions affect vehicle placement while others do not.
5. Vehicles travel farther from the roadway edge at the center of the bridge under all curbing conditions.
6. Vehicles travel farther from the shoulder at the center and the upstream and downstream ends of the bridge under the 1.8-m (6-ft) curb conditions. There is a small but definitely significant displacement of vehicles on the bridge for the 1.8-m (6-ft) curb condition.
7. With the 1.8-m (6-ft) curb, vehicles tend to move slightly away from the shoulder edge as they cross the bridge, then tend to overcorrect, and move nearer the shoulder downstream of the bridge.

Future research of the type conducted in this study should include, at least in the initial phase, an additional evaluation to verify the conclusion from this study that data for separate days of the week may be combined. Ideally, this evaluation should be based on speed and placement data collected for a full 5-day week as a minimum and check for significant differences between days.

The results of this study, in the context of implications for design or vehicle operations, tend to support the conclusion that the effects of bridge safety curb on vehicle speeds and placements, although statistically significant, are not practically significant, at least during daylight hours. The difference in speeds between positions can probably be attributed to the 5 percent downgrade and is therefore highly suspect. Further research in this area should be conducted on a level or near level roadway grade if possible.

Although the effect of curbing on speeds is statistically significant, the rank order of these effects creates some doubt about the practical importance of this difference. The highest speed occurred with no curb and the lowest with 1.2 and 1.8-m (4 and 6-ft) curb conditions. The higher speed with no curb is probably true, and, although the lower speeds for the three curb conditions are also true, the seemingly significant difference in speeds between the 1.2-m (4-ft) curb and the other curb conditions is probably due to chance alone. This can be verified with additional research. In this case, however, the difference in speed of 2.62 km/h (1.63 mph) between the base condition of no curb and the 1.8-m (6-ft) curbing condition is less than 4.0 km/h (2.5 mph), which can be defined as the range of accuracy for design standards. Design standards are established in increments of 8 km/h (5 mph); therefore, a change of less than half this increment would have no effect on these standards.

The conclusion about interaction between positions and conditions for vehicle placement is not surprising. Drivers definitely tend to move away from any obstacle placed near the edge of the roadway. The simulated curb on the bridge caused the drivers to displace as they approached and crossed the bridge and then to return to a position closer to the shoulder edge after crossing the bridge. In fact, drivers tend to overcorrect after crossing the bridge and to move nearer the shoulder edge than they were upstream of the bridge. This is particularly true with the 1.2 and 1.8-m (4 and 6-ft) curb conditions.

The maximum difference in vehicle placement occurred at the upstream end of the bridge and was only 0.16 m (0.54 ft). The maximum differences in placement from the other conditions at the center of the downstream end of the bridge were 0.11 and 0.08 m (0.35 and 0.26 ft) respectively. It can be concluded from these differences that the displacement of vehicles as they crossed the bridge would not be large enough to affect the lane width on the bridge.

It should be pointed out that the conclusions drawn from this study can only be applied to bridges on relatively high-speed, one-way roadways with two lanes. Two-way bridges, even though they may have two lanes in each direction, may and probably would give entirely different results.

The findings and conclusions developed from this study are significant but must be considered in the light of the many restrictions that were of necessity placed on the project. The major restrictions were as follows:

1. Only free-flowing vehicles were considered;
2. A single data site was used;
3. Only daylight conditions were investigated;
4. The test bridge was relatively short; and
5. Data were collected during fair weather conditions only.

Even with these restrictions, there is strong evidence to support the conclusion that 1.8-m (6-ft) outside shoulders on rural freeway bridges would not seriously affect the operational characteristics of vehicles as they crossed the bridge.

It is recommended that further research be carried out to investigate design factors not included in this project. If the findings of this further study support the findings of this project, then the recommendation can be made that bridge designers give serious consideration to reducing the outside shoulder width on rural freeway bridges to a minimum of 1.8 m (6 ft).

ACKNOWLEDGMENTS

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The contents of this paper reflect the views of the author who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the state or the Federal Highway Administration.

Abridgment

Effect of Guardrails on Interstate Bridges on Vehicle Speed and Lateral Placement

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West Virginia University conducted a research study to determine the best shoulder and curb widths on highway bridges from the standpoint of safety, operational, and cost. A major objective of the study was to determine the behavior of traffic (speeds and lateral placements) on long-span bridge structures with different shoulder and curb widths. A secondary objective was to determine whether the addition of a guardrail barrier flush with the face of a curb on a bridge affects the lateral placement and speed of moving vehicles on the structure.

Data were collected in the vicinity of and on a bridge on I-79 approximately 1.6 km (1 mile) from the downtown area of Fairmont, West Virginia, for speeds and lateral placements with various bridge shoulder curb widths with a guardrail type of barrier both flush with the face of the curb and offset 0.6 m (2 ft) from the face of the curb. Ten conditions were studied and are reported in this paper. Two were a base condition (no guardrail or curb present) with and without a sign saying TEST BRIDGE AHEAD. Four were a guardrail mounted flush with the curb 0.6, 1.2, 1.8, and 2.4 m (2, 4, 6, and 8 ft) from the parapet. Three were offset guardrails 1.2, 1.5, and 2.4 m (4, 5, and 8 ft) from the parapet. One was only a curb 2.4 m (8 ft) from the parapet.

The site, data collection procedure, and data reduction procedure are identical to those used by Roberts in a paper in this Record. Further information on the data collection procedure is contained in Byrne and others in a paper in this Record. Much greater detail on results may be found in Byrne (1).

RESULTS

The data were analyzed by using the analysis of variance technique and Tukey tests to find significant differences between speeds and placements for different conditions and positions. Position refers to tape switch trap position. Positions 1 and 2 were 300 and 150 m (1000 and 500 ft) upstream of the bridge. Positions 3, 4, and 5 were at the upstream end, middle, and downstream end of the bridge respectively. Position 6 was 150 m (500 ft) downstream of the bridge.

The following analyses were performed for both speed and placement:

1. One-way analysis of variance for each condition at all positions to determine whether there is any significant difference between positions for any condition,
2. One-way analysis of variance for all conditions at each position to see whether there is any significant difference between conditions for each position,
3. Two-way analysis of variance for all conditions at all positions to determine whether there is any interaction between positions and conditions, and
4. Tukey's test to find the significant difference between positions and conditions.

The one-way analysis of variance of speeds showed that there is a significant difference between speeds at positions for the following conditions: (a) 0.6-m (4-ft) guardrail, (b) 1.8-m (6-ft) guardrail, and (c) 2.4-m (8-ft) curb. For the other seven conditions, there was no significant difference between speeds within positions.

The one-way analysis of variance of placement showed that there is a significant difference in placements between positions for each condition except the base condition with sign. The analysis of variance of speeds and placements for conditions within positions showed that there is a significant difference between both speeds and placements for all conditions and for each position.

The two-way analysis of variance of speeds revealed a significant difference between speeds for positions and conditions for all combinations except for base conditions. For the base conditions, there was no significant difference between conditions, but there was a significant difference between positions. However, there was no interaction of positions and conditions.

The two-way analysis of variance of placements showed a significant difference for positions and conditions as well as significant interactions between positions and conditions for vehicle placements. Breaking down

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the two-way analysis of variance to related conditions shows interaction within the guardrail conditions and the offset guardrail conditions but not within the two base conditions and the 2.4-m (8-ft) conditions.

The last statistical test run was Tukey's test, which shows where significant differences exist within conditions and positions. Tukey's test was run twice on the main effects of speeds because there was no interaction (as shown in the two-way analysis of variance). The first run was for six positions and overall means of 10 conditions. The second run was for the overall means for each condition at all positions.

Tukey's test was run 16 times on simple effects of placements inasmuch as there was interaction. The first 10 were for each condition at all positions. The rest were for all conditions at each position.

Figure 1. Mean placements for guardrail conditions at positions.

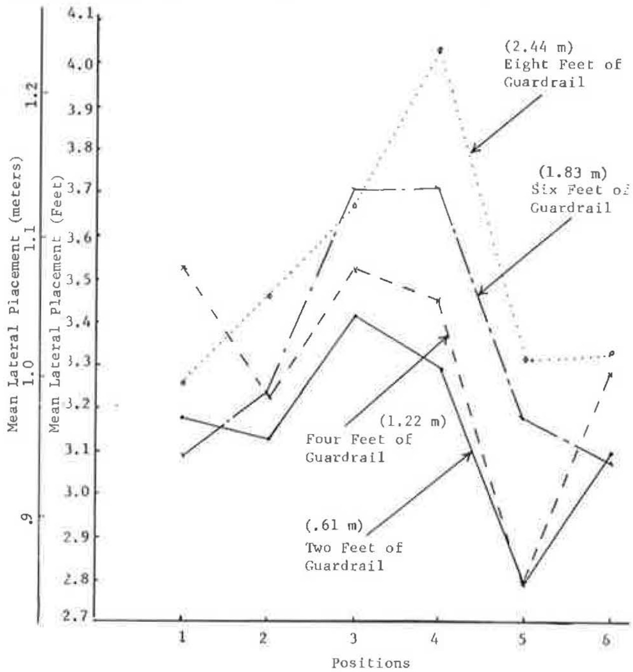
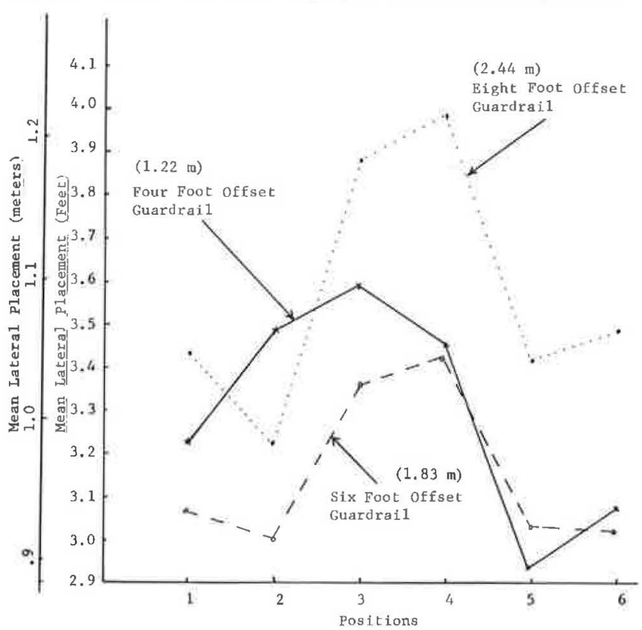


Figure 2. Mean placements for offset guardrail conditions at positions.



There was no significant difference between the two base conditions in speeds and placements at all positions. Also the least frequency of significant differences within positions was observed in the two base conditions; the greatest frequency was observed in the 1.8-m (6-ft) guardrail, 2.4-m (8-ft) guardrail, 1.8-m (6-ft) offset guardrail, and 2.4-m (8-ft) curb conditions.

Figure 1 shows the mean placements for the guardrail conditions. The maximum placements occurred at positions 1 and 3 for the 0.6-m (2-ft) guardrail condition and at position 3 for the 1.2-m (4-ft) guardrail condition. These two conditions are not significantly different at position 3, but they are at position 1 where the difference is 0.113 m (0.37 ft). The maximum placement occurred at position 4 for the 1.8 and 2.4-m (6 and 8-ft) guardrail conditions, but these were not significantly different from each other at all positions.

Figure 2 shows the mean placements for the offset guardrail conditions. The maximum placement occurred at position 4 for the 1.8 and 2.4-m (6 and 8-ft) offset guardrail conditions, while the maximum placement occurred at position 3 for the 1.2-m (4-ft) offset guardrail condition. Note the inconsistencies where the placements for the 1.2-m (4-ft) offset guardrail condition are greater than those for the 1.8 m (6-ft) offset guardrail condition at positions 1, 2, 3, 4 and 6.

In addition there was no significant difference in placements for the 1.2-m (4-ft) guardrail and offset guardrail conditions at positions 2, 3, 5 and 6, and there was no significant difference between the 1.8-m (6-ft) guardrail and offset guardrail conditions at all positions.

Mean speeds were at a maximum mostly at position 6, and they were at a minimum mostly at position 1. In general, mean speeds increased at positions 2 and 3, then decreased at positions 4 and 5, and then increased again at position 6. The increase in speed is attributed to the 5 percent grade. The decrease in speed is attributed to the guardrail and the bridge.

Placements at positions 3 and 4, the beginning and the middle of the bridge, are in general the highest and significantly different from the other positions. The mean placement was at a maximum at position 4, 1.23 m (4.03 ft) for the 2.4-m (8-ft) guardrail condition, while the maximum mean placement at position 3 was 1.17 m (3.89 ft) for the 2.4-m (8-ft) offset guardrail condition.

CONCLUSIONS

The following conclusions are drawn regarding speed.

1. There is a significant difference in speeds between positions and conditions but no interaction between positions and conditions, as was shown in the two-way analysis of variance.
2. There is no significant difference in speed between the base conditions with and without the sign.

The following conclusions were reached regarding placements.

1. There is a significant difference and interaction in placements between positions and conditions.
2. There is a significant difference in placement between positions for the base condition with no sign, but there is no significant difference in placement between positions for the base condition with the sign.
3. Placements at positions 3 and 4, which are at the beginning and the middle of the bridge, are, in general, the highest and significantly different from the other positions.
4. Vehicles move away from the shoulder as they

approach the bridge and cross it but tend to move back toward the shoulder at the lower end of the bridge. This is particularly true for the 2.4-m (8-ft) conditions.

There is no significant difference in placement for the 2.4-m (8-ft) guardrail, 2.4-m (8-ft) offset guardrail, and 2.4-m (8-ft) conditions at all positions.

Wider guardrails have a definite effect on vehicle placement, particularly in the center of the bridge where the average maximum placements occurred.

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Characteristics of Intersection Accidents in Rural Municipalities

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Webb L. Tyler,* Bremner, Youngblood, and King, Inc.

Engineering guidelines for traffic and safety improvements have developed from studies conducted primarily in urban areas where traffic engineering expertise is available. This paper summarizes applicable data collected in several comprehensive studies of small city and town intersections. Conclusions are drawn concerning those areas in which urban and rural accident patterns and roadway conditions are both similar and different and concerning how the difference may affect traffic engineering decision making for rural areas.

STUDY AREA

More than 300 intersections in 42 towns and cities in Virginia were included in two studies funded by the Virginia Division of Highway Safety. Initial data collection funding was from U.S. Department of Transportation Highway Safety funds. Of the total number of intersections studied, 232 rural intersections are reviewed in this paper. The typical rural municipality has an average population of approximately 15 000. Accident data are based on state records obtained from the municipalities, which must report all accidents causing at least $100 damage. More than 2300 accidents are summarized by intersection for a 24-month period between 1969 and 1973.

If there are differences in accident characteristics between rural and urban areas, there logically should be differences in driver behavior or roadway conditions to cause these differences. In rural areas, drivers may be less aggressive because of less traffic congestion, a higher percentage of local drivers familiar with the intersections, and drivers not conditioned to extensive repetition and enforcement of standard roadway design and traffic operation.

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*Mr. Tyler was with Wilbur Smith and Associates during preparation of this paper.

ACCIDENT TYPES VERSUS INTERSECTION TRAFFIC CONTROL

A summary of all intersection accidents studied by accident type and intersection traffic control is given in Table 1. Rear-end collisions and angle collisions compose 36 and 43 percent of total accident types respectively. The percentage for angle collisions is low compared to that found in some other studies, which indicated that angle accidents constituted as much as 83 percent (1) of the total. The intersections with STOP and YIELD sign control have a higher percentage of angle and lower percentage of rear-end collisions than the signalized intersections, which is consistent with other studies (2).

ACCIDENT TYPES VERSUS INTERSECTION GEOMETRICS

A summary of accidents by intersection geometrics and type of traffic control is given in Table 2. Whereas Y-type and offset intersections have accident patterns characteristic of the summary totals in Table 1, there are some interesting facts in four-way and T-type intersection patterns. Previous studies indicate that four-way intersections have up to four times the number of accidents as T-types of intersections (3). Although the four-way intersection accident rate of 1.35 is higher than the 0.80 for T-types, this is only a 69 percent increase.

Signalized four-way and T-types of intersections have higher percentages of rear-end collisions than intersections controlled by STOP and YIELD signs (40 and 58 percent versus 22 and 28 percent respectively). On the other hand, four-way and T-types of intersections with STOP and YIELD sign control have a higher percentage of angle collisions than do signalized intersections (59 and 43 percent versus 40 and 29 percent respectively). This reconfirms general knowledge that signalization of an intersection tends to reduce angle collisions but increase rear-end collisions.

SIGNALIZED INTERSECTIONS

A summary of accidents at signalized intersections conforming with the minimum signal display criteria of the 1971 Manual on Uniform Traffic Control Devices— two indications per approach and one head within the 40-deg cone of vision—is given in Table 3. Also identified are those signalized intersections that meet the minimum traffic volume warrants of the manual. The most important fact is that the accident rate for all four categories is nearly identical. Whether or not a traffic signal control meets the volume warrants or standard display criteria appears to have no bearing on accident frequency. Even the breakdown by accident type is fairly consistent for all four categories despite the small sample size for two categories. These findings are inconsistent with other studies that indicate that signalized intersections with lower traffic volumes (4, chap. 4) and substandard signal display (5) tend to have higher accident rates.

It is interesting to note that both warranted and unwarranted signalized intersections with substandard displays have a higher percentage of angle collisions (48 and 46 percent) than the standard display intersections (40 and 35 percent). The occurrence of fewer angle collisions at standard display intersections is consistent with a previous study (5).

TRAFFIC VOLUMES

A comparison of accident rates under STOP or YIELD sign control versus traffic signal control was made for various intersection types and average daily traffic (ADT) volumes (Table 4). For a given intersection and ADT, signalized intersections have a higher accident rate than those intersections with STOP or YIELD sign

Table 1. Summary of accidents.

Intersection		Rear End		Angle		Sideswipe		Other[a]			Average
Type	Number	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Total	Accident Rate[b]
Signalized	76	508	43	445	37	142	12	100	8	1195	1.26
STOP or YIELD sign control[c]	156	316	29	542	49	110	10	138	12	1106	1.08
Total	232	824	36	987	43	252	11	238	10	2301	1.13

[a]Includes mostly head-on and fixed-object collisions. [b]Accidents per million entering vehicles. [c]Includes STOP sign control with flashing beacons.

Table 2. Accidents by intersection geometrics.

Intersection		Rear End		Angle		Sideswipe		Other			Average
Type[b]	Number	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Total	Accident Rate[a]
Four-way											
Signalized	52	379	40	384	40	105	11	79	9	947	1.47
STOP or YIELD sign control	66	125	22	340	59	55	10	50	9	570	1.27
Total	118	504	33	724	48	160	11	129	8	1517	1.35
T-type											
Signalized	12	68	58	30	25	13	11	7	6	118	0.82
STOP or YIELD sign control	48	72	28	109	43	30	12	44	17	255	0.79
Total	60	140	38	139	37	43	11	51	14	373	0.80
Offset											
Signalized	3	3	42	0	0	2	29	2	29	7	0.40
STOP or YIELD sign control	9	16	34	14	30	6	13	11	23	47	0.76
Total	12	19	35	14	26	8	15	13	24	54	0.58
Y-type											
Signalized	1	10	42	7	29	6	25	1	4	24	1.40
STOP or YIELD sign control	14	68	66	24	23	4	4	7	7	103	1.04
Total	15	78	61	31	25	10	8	8	6	127	1.22

[a]Accidents per million entering vehicles. [b]Accidents for miscellaneous intersection geometrics not summarized.

Table 3. Accidents by signalized intersections.

Intersection		Rear End		Angle		Sideswipe		Other			Average
Characteristics	Number	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Total	Accident Rate[a]
Meets warrants											
Standard display	50	390	45	306	35	107	12	69	8	872	1.26
Substandard display	5	32	36	41	46	8	9	9	9	90	1.28
Below warrants											
Standard display	14	67	38	70	40	25	14	13	8	175	1.26
Substandard display	7	19	33	28	48	2	3	9	16	58	1.23

[a]Accidents per million entering vehicles.

control. This is true for all four traffic volume categories despite significant variations in sample size for each category. The signalized intersections, in fact. have a 29 percent higher accident rate. This strongly suggests, as do other studies (6), that a typical signalized intersection will have a higher accident frequency than one with STOP or YIELD sign control.

SEVERE GRADES, POOR SIGHT DISTANCE, AND NIGHT VERSUS DAY

Accident data for intersections that provide poor driver sight distance on at least one traffic approach or that have an unusually steep grade are given in Table 5. (Poor sight distance is based on factors such as vehicle speed and degree of sight obstruction as well as sight distance. Severe grades are usually greater than 5 percent.) The accident rate of 0.97 for intersections with severe grades is unusually low in light of the high accident potential such roadway conditions possess and when compared to the accident rate of 1.13 for all intersections. During the study of the intersections, it was observed that intersections with extremely severe grades, such as many of those in the small municipalities in the Shenandoah and Blue Ridge mountains, experience unusually low accident rates. It appears that drivers are aware of the dangerous roadway conditions and exercise due caution.

Intersections with poor sight distance have a relatively high accident rate of 1.33. As would be expected, 56 percent of the accidents were angle collisions in which the driver was unable to properly view an approaching vehicle on the cross street.

Of the total accidents, 30 percent occurred at night with less than 3 percent variance from this amount for any traffic pattern or roadway geometry category. This strongly suggests that traffic and physical roadway conditions have no relationship to frequency of night accidents.

CONCLUSIONS AND RECOMMENDATIONS

Driver behavior and roadway conditions in rural municipalities differ from those in urbanized areas. However, the following conclusions concerning rural accident characteristics reconfirm results of previous studies.

1. A typical intersection with a given volume of traffic will have a higher accident frequency under traffic signal control than under STOP or YIELD sign control. Prior to the costly installation of traffic signal control, a thorough engineering analysis must be performed to clearly identify and quantify the benefits of signalization.
2. Intersections with poor driver sight distance on one or more traffic approaches tend to have a higher-than-normal accident rate, particularly with regard to angle collisions. Increasing driver sight distance will likely effect a reduction in this type of collision.
3. Standardizing signal display should result in reduced accidents at locations with a relatively high number of angle collisions.
4. The frequency of night accidents appears to be totally unrelated to traffic patterns, traffic control, or intersection geometrics. Although it was not determined from the data, adequacy of proper night lighting appears to be the controlling factor in this environment. At intersections where more than one-third of the accidents occur at night, the adequacy of street lighting should be determined.

Some unique conclusions, which seem to apply only to rural municipalities, can be drawn from the accident data.

1. Intersections with severe grades generally operate safely although they are obviously potential hazards. Accident histories should be closely studied before substantial funds are invested to alleviate a severe grade condition.
2. Signalized intersections with volumes exceeding the traffic volume warrants are no safer than signalized intersections with volumes below the warrants. This can possibly be explained by differences between rural and urban driver characteristics. The need to implement a policy of eliminating unwarranted signals in rural areas is perhaps not so urgent as in urban areas.
3. Signalized intersections with displays that meet approved standards are no safer than signalized intersections with substandard displays. Again, differences between rural and urban driver characteristics and physical surroundings could explain this discrepancy.

Although traffic engineers in rural jurisdictions should certainly continue to upgrade substandard signal displays, the need to implement this policy is perhaps not so urgent from a safety standpoint as it is with the urban counterpart.

Whether or not traffic control measures are to be implemented in an urban or rural area, sound traffic engineering analysis and judgment must be followed. To provide effective rural policy, the traffic engineer requires a knowledge of the particular conditions of the given study area and an awareness of the general findings of this paper.

Table 4. Average accident rate by intersection ADT.

ADT	Traffic Control	Number of Intersections	Average Accident Rate[a]
<10 000	Sign	93	1.12
	Signal	15	1.33
10 000 to 15 000	Sign	47	1.05
	Signal	35	1.26
15 000 to 20 000	Sign	11	0.97
	Signal	12	1.09
>20 000	Sign	5	0.52
	Signal	14	1.26

[a]Accidents per million entering vehicles.

Table 5. Accidents at intersections with severe grades and poor sight distance.

Intersection		Rear End		Angle		Sideswipe		Other			
Condition	Number	Number	Percent	Number	Percent	Number	Percent	Number	Percent	Total	Average Accident Rate[a]
Severe grades	35	106	39	104	38	24	9	37	14	271	0.97
Poor sight distance	41	73	20	207	56	32	9	54	15	366	1.33

[a]Accidents per million entering vehicles.

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Highway Accidents at Bridges

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Grade separation structures (bridges) at interchanges and crossroads and over streams and railroads, which are intended to provide greater convenience and safety, involve features that either obstruct the range of free travel or serve as containment barriers. The objective of this study was to identify the principal features of bridges and appurtenances that may be related to accident frequency and severity and to provide some further insights to highway safety.

PROCEDURE

State police files were searched for accident reports identifying a bridge (underpasses and overpasses) as being involved. Interstate routes and parkways (toll roads) were grouped together and analyzed as one system; accident records and total accident statistics were compiled for the 2-year period 1972 to 1973. Data on fatal accidents on primary and secondary systems covered the same 2-year period; however, nonfatal accident summary statistics were compiled for only 1 year (1972) from about one-third of the counties. The accidents were divided into several types, and the severity of each type was determined by means of a severity index (SI) (1). Roadway and environmental conditions at the time of the accidents were also noted.

RESULTS

Interstates and Parkways

Number of Bridges

At the end of the study period (1973), there were approximately 350 overpasses and 360 underpasses on the Interstate and parkway system (dual bridges were counted as one). About 35 percent of the overpasses had full-width shoulders. Approximately 10 percent of the overpass accidents occurred on those overpasses that had full-width shoulders. Ninety-eight percent of the underpasses had a pier in the median. The desirable clearance from the right edge of the roadway to the shoulder pier should be 9 m (30 ft). This was the case for only about 8 percent of the underpasses. The average lateral clearance rightward was slightly more than 4 m (14 ft).

Number of Accidents

Almost 8 percent of all accidents involved bridges. Of the 438 accidents involving bridges, only 31 involved underpasses. More than 14 percent of all fatal accidents involved bridges, and more than 17 percent of all fatalities involved bridges. Almost 9 percent of the injuries occurred in accidents involving bridges. These percentages show that bridge-related accidents compose a significant portion of the total accident experience and a significant portion of the more severe accidents. The severity index of bridge-related accidents was 3.24 compared to 2.75 for all accidents. Discussions of each type of accident follow.

1. Collision with bridge pier—This type of accident resulted in six fatalities during the 2-year study period. Five of the fatalities occurred where there was no safeguard about the piers. Of 14 accidents involving bridge piers (SI = 7.00), there were three fatal, nine injury, and two noninjury accidents. Severity was reduced significantly when the pier was shielded with guardrail (SI = 4.77) or an earth mound (SI = 1.00) (2). A very limited number of accidents involved earth mounds. Two reported accidents at earth mound locations were noninjury. Accidents involving guardrails at bridge piers indicated that end treatment of guardrails continues to be a problem with this method of diverting vehicles away from bridge piers. Of the 11 accidents involving guardrails at bridge piers, there were one fatal and six injury accidents. In the fatal accident, the vehicle mounted the approach end of the guardrail, became airborne, and impacted the shoulder pier. In another accident involving a severe injury, the vehicle became airborne and hit the center pier. In two other accidents, vehicles hit

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the shoulder pier after first mounting the end of the guardrail and then vaulting it.

2. Gap between bridge openings—Of five accidents involving a wall built to close the gap (SI = 8.30), there were three fatal (resulting in eight fatalities) and two injury accidents. Bushes had been planted in front of the wall to retard encroaching vehicles at two of the locations, but one of the two accidents at those locations still resulted in a fatality.

Guardrails ahead of the gap were found to be only partially effective; the newer and longer rails are much better than the short sections previously used. Of 15 accidents involving guardrails, six were fatal (11 fatalities). In five of the fatal accidents, the vehicle went over the guardrail; in one instance, the vehicle went around the guardrail. The guardrail completely stopped a vehicle from going through the gap in only five cases, and these cases involved the newer design.

3. Collision with entrance posts and wing walls—Of 29 accidents (SI = 6.67), there were 9 fatal (nine fatalities), 16 injury, and 4 noninjury accidents. Twelve of the accidents involved collision with the right entrance. At all of these locations, the shoulder narrowed at the bridge. Only two of the remaining 17 accidents, which involved the left entrance, involved a bridge that had a full-width shoulder. Light and visibility conditions appeared to be a contributing factor. Only 9 of the 29 accidents (and one of the nine fatal accidents) occurred during daylight. Three of the nighttime fatal accidents were attributed to the driver's going to sleep. In the majority of locations where guardrail was provided, it was not attached to the bridge to prevent pocketing. In newer installations, the guardrail is attached and should reduce the severity of these accidents.

4. Collision with bridge railing or curb—Collision with the bridge railing or pier was the most frequent type of accident and was a low-severity type (SI = 2.16). The majority of these accidents (61 percent) occurred during inclement weather. The railing design appeared structurally adequate; only three accidents (one fatality) involved a vehicle going through or over the railing. These three accidents (2 percent of the total of this type) involved a semitrailer, bus, and sedan. The curb and safety walk combination, formerly a design standard, did not provide good redirectional qualities.

5. Collision with bridge railing and guardrail—A high percentage of these accidents occurred during icy or wet conditions (43 percent). The average severity was not high (SI = 2.85). There was only one fatality, which resulted when the driver was thrown from his vehicle when it overturned after striking a guardrail.

6. Collision with guardrail—In most of collisions with the guardrail, the driver lost control of his vehicle on an icy bridge and then struck a guardrail. Icy or wet conditions were a factor in 80 percent of the accidents. In three accidents, the driver lost control after hitting the bump at the end of the bridge.

7. Collision with another vehicle—Inclement weather was a factor in 58 percent of this type of accident. Wet road conditions were the cause of the only fatal accident. Lack of room was mentioned on some of these accident reports; the driver could not avoid another vehicle because the bridge was narrow.

8. No contact with bridge, guardrail, or vehicle—In accidents in which no contact was made with bridges, guardrails, or vehicles, drivers lost control and proceeded into the median or off the shoulder. Icy conditions existed in 79 percent of these accidents.

Roadway and Environmental Conditions

The percentages of each accident type were compared to that of all accidents on the Interstate and parkway system (3). The percentage of accidents related to road character was very similar to that found for the total system. However, differences were found for road surface and light conditions. The percentage of accidents during snowy or icy conditions (46 percent) was considerably higher than that for the total system (17 percent). Also, the percentage of nondaylight accidents (54 percent) was higher than the corresponding percentage for the entire system (40 percent). The percentage of nondaylight accidents that involved icy conditions (65 percent) was more than that of all bridge-related accidents, indicating that the problem of ice-related accidents is greater at night.

Attempts to alleviate the hazards from ice on bridge decks with warning signs have been moderately successful. Investigation of three locations where ICE ON BRIDGE signs were placed indicated some accident reduction. The year before the signs were placed seven ice-related accidents occurred; the year after, only two. However, ice-related accidents have continued to occur at two of the locations since placement of the signs. Flashing ICE ON BRIDGE signs, activated by detectors in the bridge decks, have been installed at two locations. Problems with the detectors have made operation undependable; one is now being activated manually. Accidents during icy conditions have continued to occur at these locations in spite of the flashing signs.

Primary and Secondary Highways

Number of Accidents

The percentage of bridge-related accidents was considerably less on primary and secondary highways than on Interstates and parkways. This seems to be related to the smaller number of bridges per kilometer on the primary and secondary system (about 0.19 bridge/km or 0.3 bridge/mile) as compared to the Interstate and parkway system (about 0.43 bridge/km or 0.7 bridge/mile). Bridges were involved in 3 percent of all accidents and 4 percent of fatal accidents. Accidents involving bridges resulted in about 4 percent of all fatalities and of all injuries. As on Interstates and parkways, the severity of bridge-related accidents was shown to be high: SI = 3.26 compared to SI = 2.86 for all accidents. The severity of bridge-related accidents on primary and secondary highways (SI = 3.26) was almost identical to that of the bridge-related accidents on the Interstate and parkway system (SI = 3.24).

Discussions of each type of accident follow.

1. Collision with bridge pier—There were only nine reported collisions with bridge piers. Four were fatal accidents. The pier had no guardrail in seven of the accidents, three of which were fatal accidents. In the other fatal accident, the vehicle hit the approach terminal of the guardrail, became airborne, and turned over.

2. Collision with bridge entrance post or wing wall—Collisions with entrance posts or wing walls were the most severe accidents (SI = 5.65). The high severity resulted from direct collision with entrance posts or wing walls; none of the 27 fatal accidents of this type involved guardrail protection. A very high percentage of these accidents occurred at night (61 percent).

3. Collision with bridge railing or curb—As on Interstates and parkways, collisions with bridge railing or curb were the most frequent type of accident. Many of these accidents (44 percent) occurred during inclement weather, particularly icy conditions. This type of accident was not usually severe (SI = 2.64). The exceptions were accidents where the vehicle went through the

bridge railing or hit another vehicle on the bridge. Several fatal accidents resulted because of inadequate containment by the railing.

4. Collision with bridge railing and guardrails—Only seven accidents involving collision with bridge railing occurred where guardrail had been used in conjunction with bridges.

5. Collision with guardrails—Most of the collisions with guardrail (75 percent) involved a driver losing control of the vehicle on an icy or wet bridge and then striking a guardrail. There were two fatal accidents. One involved a vehicle jumping the guardrail; in the other, the vehicle went through the guardrail.

6. Collision with another vehicle—Collisions with another vehicle were another common type of accident. The two primary causes were icy or wet conditions (49 percent) and a narrow bridge.

7. No contact with bridge, guardrail, or vehicle—As on Interstates and parkways, icy or wet conditions were the cause of the majority of accidents involving no contact with the bridge, guardrail, or other vehicle (69 percent).

8. One-lane bridges—A number of one-lane bridges exist on the secondary systems. As would be expected, the most frequent type of accident involved two vehicles meeting on the bridge. Five fatal accidents were attributed to the absence of safety rails. Investigation of six locations where NARROW BRIDGE signs were installed showed that signing does alleviate this problem. There were 41 accidents before compared to 27 accidents after installation of the warning signs.

Roadway and Environmental Conditions

The percentages of bridge-related accidents were ordered according to road character, road surface, and light conditions. These percentages were compared to values found for all state-police-reported accidents on the primary and secondary system. The only difference found with respect to road character was the percentage of fatal accidents on curves (48 percent): It was higher than for the entire system (33 percent). The percentage of wet-weather accidents (31 percent) was slightly higher than that for the entire system (23 percent). The percentage of accidents during snowy or icy road surface conditions was only 4 for the total system compared to 20 for bridge-related accidents. The percentage of bridge-related accidents at night (43 and 55 percent) was also shown to be much higher than that for the total system (27 percent).

DISCUSSION, SUMMARY, AND RECOMMENDATIONS

1. Bridge-related accidents were a significant percentage of the total accidents on Interstates and parkways.

2. The lesser number of bridges per kilometer on the primary and secondary highway system, together with generally lower traffic volumes and speeds, appeared to be related to fewer accidents involving bridges on those systems as compared to the numbers of bridges and accidents on Interstates and parkways.

3. The severity of bridge-related accidents was generally higher than the severity of all accidents.

4. The severity of bridge-related accidents on primary and secondary highways was almost identical to that on the Interstate and parkway system.

5. Collisions with entrance posts and wing walls resulted in more fatalities than did accidents involving other features of bridges. Inadequate protection from direct collision with rigid elements at bridge entrances, particularly on primary and secondary highways, resulted in high severity. Lack of adequate shoulder width resulted in a large number of accidents. Where paved shoulders are provided, a means of alerting errant drivers by means of grooved sections or raised rumble strips on the shoulder in advance of the bridge would be desirable.

6. The small percentage of accidents on overpasses having full-width shoulders demonstrated the benefits obtained when this safety feature was added.

7. Guardrail protection at bridge piers has proved less than totally effective.

8. Openings between parallel bridges on divided highways are recognized hazards. When a wall is built to close this gap, some type of arresting barrier is necessary; shrubbery has not proved to be sufficient. Guardrail protection was found to be only partially effective, although the newer design, which involves a longer guardrail section, appears to be much more effective than previous designs.

9. The high percentage of nighttime accidents suggests a problem with visual perception of the structure ahead and the need for better delineation.

10. An exceptionally high percentage of accidents resulted from snowy or icy conditions, particularly on the Interstate and parkway system. This is attributable to icing of bridge decks. This commonly occurs on the bridge decks while the approach pavement remains ice-free.

11. Primary and secondary bridges with curved approaches deserve particular attention because of the high number of fatal accidents that occurred at this type of location. Improved delineation could reduce accidents.

12. Bridge railings were inadequate on some primary and secondary highway bridges. In some cases it consists of guardrail. Some fatal accidents resulted from the apparent absence of railing on some one-lane bridges.

13. One-lane bridges remaining on the secondary system constitute a recognized hazard. Warning signs were shown to be essential. Of course, the most effective solution is replacement of deficient bridges.

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