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CONTENTS

FOREWORD

The five papers and two discussions comprising this RECORD will be of interest to a variety of traffic and safety specialists. There is information of value to designers, enforcement officials, traffic operations engineers, and administrators of traffic and safety programs.

Two Ohio Department of Transportation researchers, Foody and Culp, studied the safety benefits of raised medians versus depressed medians in similar sections of Interstate highways. They looked at accident data from 1969 through 1971 on about 130 miles of each design and concluded that the 84-ft width of either cross-sectional design provided a generally adequate recovery area for encroaching vehicles. They decided that the depressed design offered more opportunity for operators to regain control and return to their roadway than did the raised design, and that both designs were comparable in relation to vehicle roll-over, severity of accidents, and primary path of the vehicle. Discussions by Anderson and Glennon challenge some of the authors' conclusions. In their closure, the authors concede that their conclusion regarding recovery capability in depressed medians is not fully supported and also offer clarification on other questions raised by the discussers.

Vaswani used the case study method to evaluate wrong-way entry incidents as related to intersection geometry, markings, and signs. He presents five such studies and offers suggestions for improvement through improved channelization and markings and through signs conveying intersection geometric information to drivers.

In research supported by UMTA, Hemphill and Surti studied a corridor in South Denver to determine if a reversible lane was justified on a major arterial. They compared costs and benefits with and without the reversible lane and concluded that the lane was justified; they suggest that their method of analysis could be applied elsewhere.

Improvement of safety at rail-highway grade crossings is often constrained by legal parameters that divide the responsibility for action between the public agency and the private company. Hopkins presents some technological aspects of the public responsibility related to train detection techniques different from the time-worn track circuit systems; some of the new approaches appear feasible and attractive.

Okechuku and Lambe describe their use of a model to design more efficient urban transportation systems, with special emphasis on the management of parking prices in a way that will minimize total driving and driver walking distances. Data from a study in Vancouver, British Columbia, were used to validate their procedures.

A COMPARISON OF THE SAFETY POTENTIAL OF THE RAISED VERSUS DEPRESSED MEDIAN DESIGN

Thomas J. Foody and Thomas B. Culp, Bureau of Traffic Control, Ohio Department of Transportation

This paper examines the safety benefits of the mound (raised) median design as compared to the swale (depressed) median design for Interstate highway medians having an 84-ft (25.6-m) design width. The effects of each median design on the frequency and severity of median-involved single-vehicle accidents, on the path of the encroaching vehicle, and on the vehicle's tendency to overturn were studied. Approximately 130 miles (209 km) of 4-lane, divided highway with each median design were studied, and the accident experience from 1969 through 1971 was analyzed. The results indicated that the 84-ft median of either cross-sectional design provides a generally adequate recovery area for encroaching vehicles, although the swale median appears to provide more opportunity for encroaching vehicles to regain control and return to their roadway. The use of either cross-sectional design for medians of this width has no effect on the primary path of the vehicle, on the vehicle's tendency to overturn, or on the resulting severity of the accident when a median encroachment results in a reported accident.

THE current emphasis on highway safety as evidenced by the Interstate Upgrading Program and the Spot Improvement Program has once again raised the question concerning the safety benefits of the raised (mound) median design when compared to the depressed (swale) median design. There are, of course, other factors, primarily economic, that may influence the final selection of the median design. Although the effects or potential benefits of these economic factors are easily predicted, the potential safety benefits of the median design are not so easily predicted.

Ohio is in the unique position of being able to determine empirically the safety benefits of the mound median design and the swale median design because it constructed a significant portion of its Interstate System with the mound design. Ohio also has a computerized accident record system, which makes the determination of safety benefits a feasible undertaking.

PURPOSE AND SCOPE

The objective of this study was the determination of the safety potential of the mound (raised) median design versus the swale (depressed) median design. The safety potential of each design was determined by the frequency and the severity of median-involved single-vehicle accidents occurring on sections of Interstate highway with each type of median design.

The study involves 125 miles (200 km) of Interstate with the mound median design and 135 miles (217 km) of Interstate with the swale median design. Both types of median have a design width of 84 ft (25.6 m) (Figure 1). Three sections with the mound median design were selected for study. These were I-75 near Toledo, I-71 between Cincinnati and Columbus, and I-70 from the Indiana border to Dayton and

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Figure 1. Typical median cross sections.

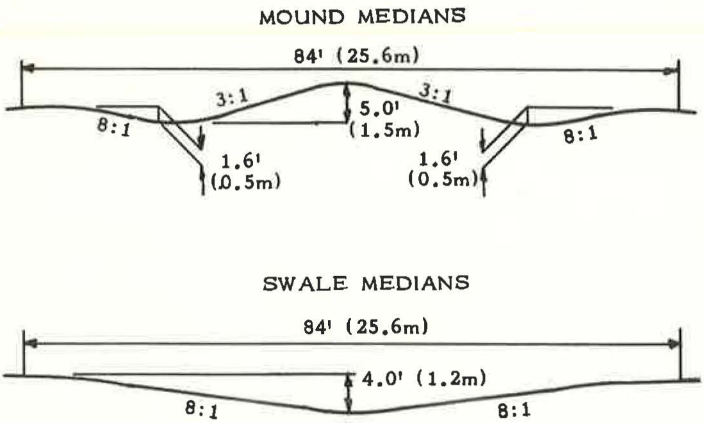
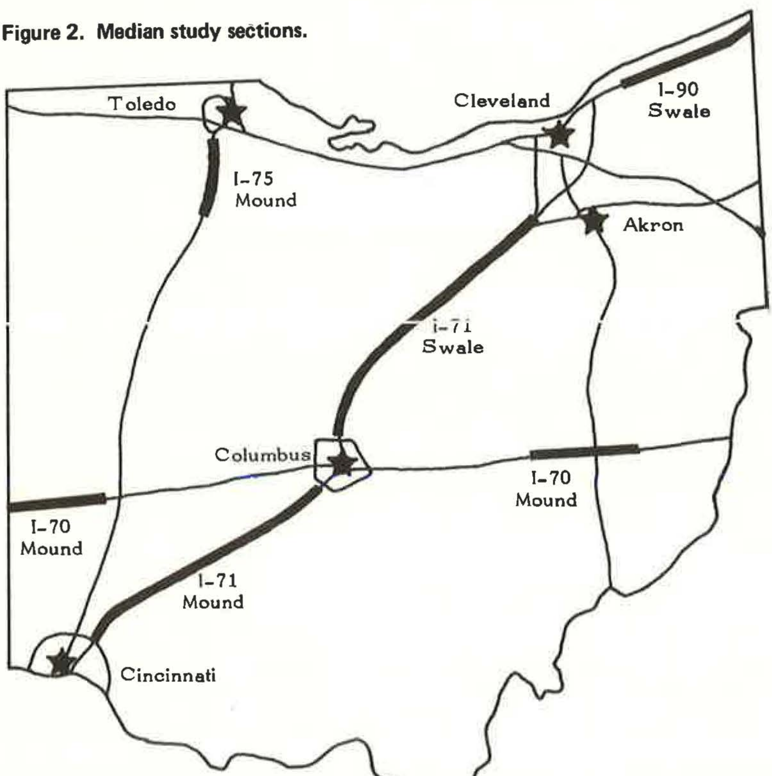


Figure 2. Median study sections.



from Zanesville to Cambridge. Two sections with the swale median design were selected for study. These were I-71 between Columbus and Medina and I-90 east of Cleveland (Figure 2). All sections selected for study were 4-lane, divided highways. Accident records for 3 years, 1969, 1970, and 1971, were used in the investigation.

PROCEDURE

To make a determination of the safety potential of each median design, it was felt that the following questions would have to be answered:

1. Does the median design affect the frequency of reported traffic accidents involving the median?
2. Does the median design affect the severity of accidents involving the median?
3. Does the median design affect the path of the vehicle after it enters the median during an accident?
4. Does the median design affect the roll-over tendencies of vehicles after they enter a median during an accident?

The sections of Interstate under study were field-inventoried to determine the location of all interchanges, structures, and median abnormalities (catch basins, roadways of unequal elevation, crossovers, etc.), and to verify the widths shown in the Road Inventory File prepared by the Bureau of Transportation Technical Services. The finite lengths of roadway to be included in the study were then formulated by eliminating all lengths of roadway between interchange terminals, all roadway 0.01 mile (0.16 km) either side of a structure, and all roadway 0.01 mile on either side of an abnormality.

These finite lengths of roadway were then matched with the computerized accident records in order to determine the accident frequency subdivided by the following types:

1. Median-involved single-vehicle accidents,
2. Non-median-involved single-vehicle accidents, and
3. All multivehicle accidents.

Once the frequencies were obtained, copies of the police investigation reports for those accidents involving the median were obtained for detailed analysis. These reports were reviewed manually to determine the path of the vehicle involved in the accident, the type of vehicle involved in each accident, and whether or not the vehicle rolled over during the accident.

ANALYSIS AND RESULTS

The analysis was structured to answer each of the four questions given previously. The first question concerned whether or not the median design affected the frequency of reported accidents. The term "reported" is used to emphasize the fact that the number of accident reports may not accurately indicate the number of incidents in which a vehicle leaves the roadway but, more accurately, will show the number of times in which the result of leaving the roadway was severe enough to be reported to the police. One valid measure of the safety potential of a given median design would be the number of times that "incidents" involving vehicles entering the median did <u>not</u> become "reported accidents". However, since figures such as these are not currently available, it is necessary to interpret reported accidents only.

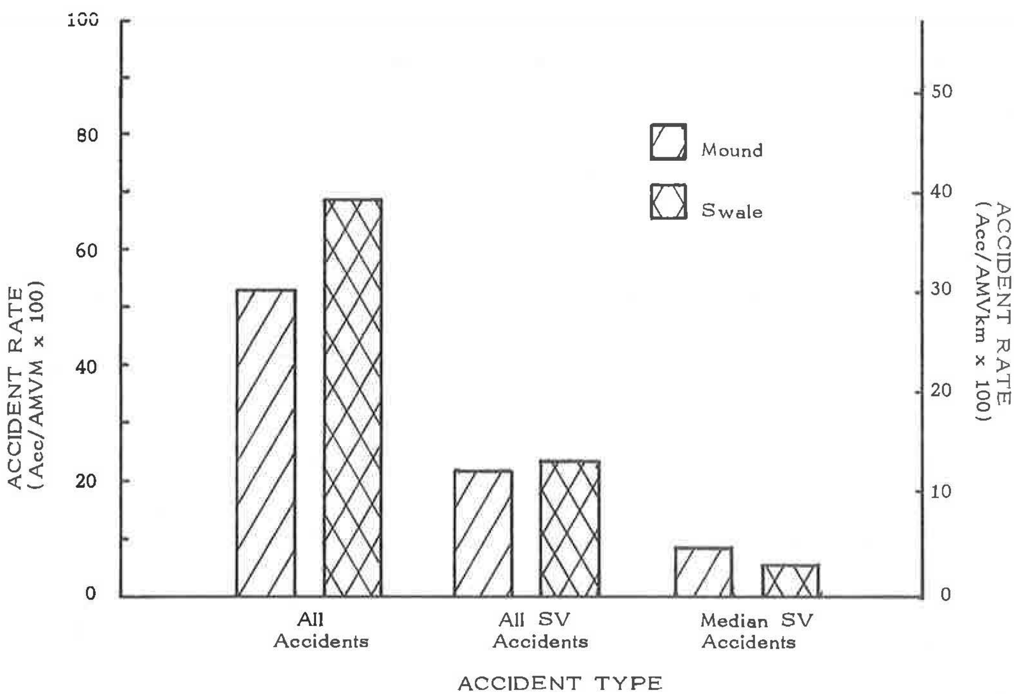
In this analysis, it is assumed that the frequency of vehicles leaving the roadway is primarily proportional to the volume of traffic, since all other design features are similar except the median design. It is then assumed that any difference in the reported accident frequency, after taking volume into account, would be the result of the median design. This assumption is substantiated by previous research for volumes in excess of 6,000 average daily traffic (1, 2, 3). Table 1 gives a summary of the accident statistics for each median design and includes a general description of the volume characteristics of each type of design. It can be noted that the average daily traffic for the two types of median design differs by approximately 1,600 vehicles (11 percent), with the higher volume being carried by the sections with the swale

Table 1. Summary of accident statistics for study sections.

Category	Median Design: Mound	Median Design: Swale
Routes		
Sections	I-71 S. of Columbus I-70 I-75	I-71 N. of Columbus I-90
Length, miles	124.19	135.23
AMVM, 3 years	1,838.62	2,409.85
ADT (average)	14,011	15,617
Total accidents		
Number	952	1,604
Fatal accidents	12	29
Severity index	0.41	0.32
Accidents/mile	7.67	11.86
Accidents/AMVM	0.52	0.67
All single-vehicle accidents		
Number	378	541
Fatal accidents	2	13
Severity index	0.43	0.33
Accidents/mile	3.04	4.00
Accidents/AMVM	0.21	0.22
Median single-vehicle accidents		
Number	125	122
Fatal Accidents	1	4
Severity index	0.40	0.43
Accidents/mile	0.99	0.90
Accidents/AMVM	0.07	0.05
Other single-vehicle accidents		
Number	253	419
Fatal accidents	1	9
Severity index	0.39	0.33
Accidents/mile	2.05	3.10
Accidents/AMVM	0.14	0.17

Note: Severity index = Fatal accidents + Injury accidents ÷ (Total accidents).

Figure 3. Comparison of accident rates.



median design. Figure 3 shows the difference in the accident frequency between the median designs, expressed as the number of accidents per 100 annual million vehicle-miles of travel. It can be seen that the accident rate for the swale design is higher for total accidents and for single-vehicle accidents. However, when considering median-involved single-vehicle accidents, the accident rate is higher for the mound median design. The significance of this difference in accident frequency was ascertained by employing a chi-square contingency test (Table 2). In this test, the number of median-involved single-vehicle accidents was compared with the number of non-median involved single-vehicle accidents and the multivehicle accidents. The underlying hypothesis in this test is that if the median design had no effect on the frequency of reported accidents, the distribution of accidents for all accident types for each median design would be equal. The results of this test indicate that the difference in the number of reported accidents between the median designs is significant.

The significantly lower number of reported median-involved single-vehicle accidents occurring on sections with the swale median design implies that more vehicles encroaching into the median are able to regain control and return to the proper roadway on sections with the swale median design than on sections with the mound median design.

The second question to be answered concerned the effect of median design on the severity of reported accidents involving the median. Figure 4 shows the percentage of median-involved accidents by severity for both the mound median design and the swale median design. In order to test for a statistical difference in the number of injury-producing accidents, a chi-square contingency test was used (Table 3). The results of this test indicate that there is no difference in the number of injury-producing median-involved accidents for the two median designs.

The third question concerned the path of vehicles after entering the median. For the purposes of this analysis, the following vehicle paths were defined:

1. Vehicle entered the median, traveled across the median, entered the opposing roadway, and came to rest either on or off the opposing roadway;
2. Vehicle entered the median, traveled along the median, and came to rest in the median; and
3. Vehicle entered the median, was redirected by the median, reentered the original roadway, and came to rest either on or off the original roadway.

These paths were titled "crossover", "median", and "redirect" respectively. All head-on multivehicle accidents that involved the median were included in the crossover category.

Figure 5 shows the percentage of median-involved accidents by vehicle path for both the mound and swale median designs. Approximately 81 percent of the vehicles entering the median of either design remained in the median. The proportion of crossover accidents was approximately equal for both median designs, as was the proportion of redirect accidents. Table 4 gives the results of the chi-square contingency test conducted to ascertain the significance of the slight differences in the number of accidents of each type between the two median designs. The results indicate that there is no difference between the two median designs in the number of accidents for each vehicle path.

The next step in the analysis was to determine the effect of vehicle path on the resulting severity of median-involved accidents. Figure 6 shows the severity index (ratio of injury-producing—including death—accidents to total accidents) by vehicle path for both median designs. A chi-square contingency test was employed to test the difference in the number of injury-producing accidents between the two median designs for each of the median paths individually. The results of these tests, at the 5 percent significance level, are as follows:

1. For the median path (81 percent of the median-involved accidents), there is no difference in the number of injury-producing accidents between the designs.
2. For the crossover path (11.5 percent of the median-involved accidents), there is a significant difference in the number of injury-producing accidents between the

Table 2. Chi-square test for difference in number of reported accidents.

Accident Type	Median Design	
	Mound	Swale
Multivehicle	574 (609)	1,063 (1,027)
Non-median single-vehicle	253 (250)	419 (422)
Median single-vehicle	125 (92)	122 (155)

Note: ( ) = Expected cell frequency.

Hypothesis: There is no difference in the number of reported accidents between the two median designs.

$\chi^2 = 22.193$ $\chi^2_{0.05} = 5.991$

Result: Reject hypothesis.

Figure 4. Severity of median accidents.

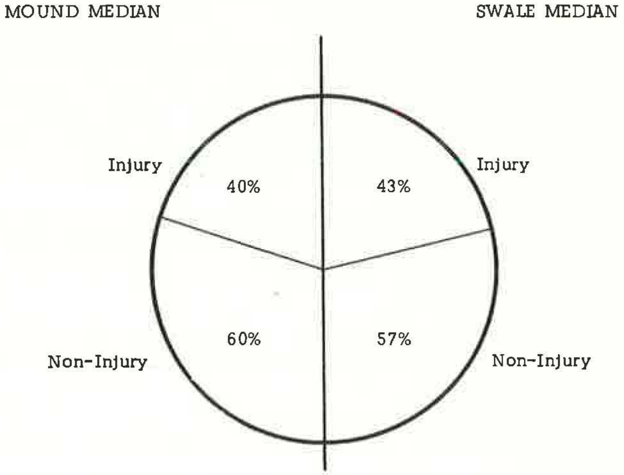


Table 3. Chi-square test for difference in number of injury-producing accidents.

Accident Type	Median Design	
	Mound	Swale
Injury	50 (52)	52 (50)
Non-injury	75 (73)	70 (72)

Note: ( ) = Expected cell frequency.

Hypothesis: There is no difference in the number of injury-producing accidents between the two median designs.

$\chi^2 = 0.96$ $\chi^2_{0.05} = 3.84$

Result: Accept hypothesis.

Figure 5. Path of vehicle in median accidents.

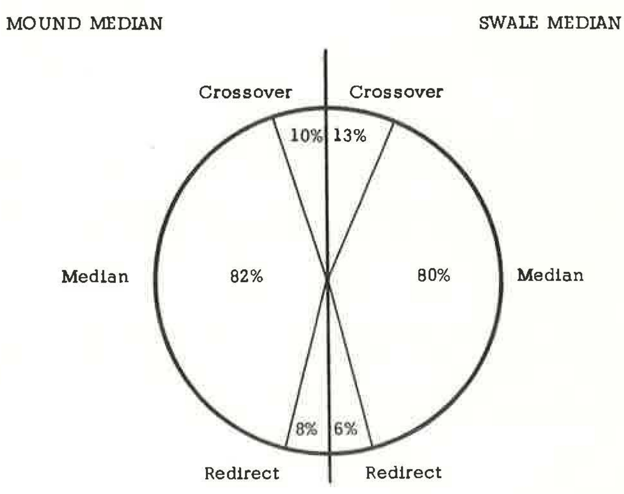


Table 4. Chi-square test for difference in number of accidents for each vehicle path.

	Median Design	
Vehicle Path	Mound	Swale
Crossover	12 (14)	16 (14)
Median	103 (102)	98 (99)
Redirect	10 (9)	8 (8)

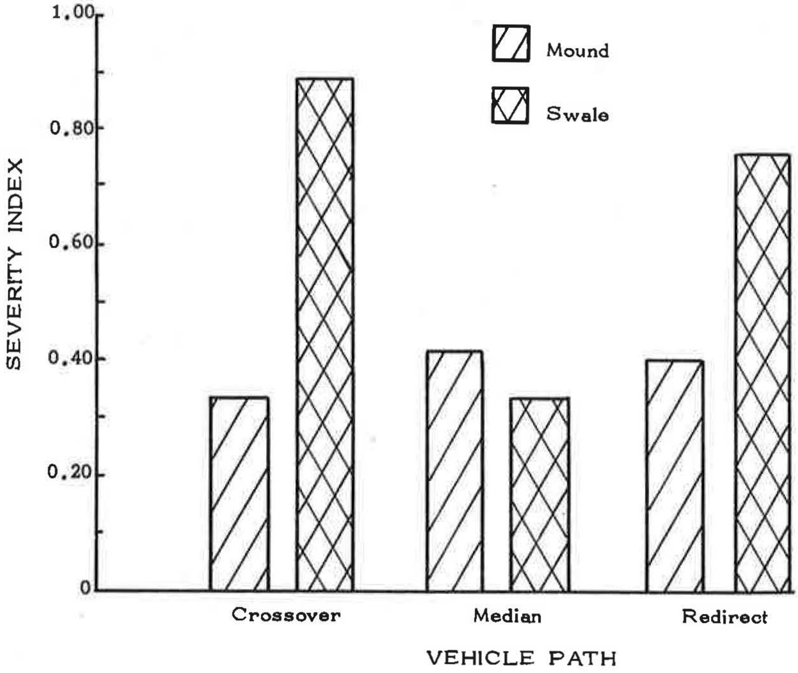
Note: ( ) = Expected cell frequency.

Hypothesis: There is no difference in the number of accidents for each vehicle path between the two median designs.

$\chi^2 = 0.814 \quad \chi^2_{0.05} = 5.991$

Result: Accept hypothesis.

Figure 6. Comparison of severity index for vehicle paths.



designs. There is a disproportionately higher number of injury-producing accidents on sections with the swale median design than on sections with the mound median design for this path.

3. For the redirect path (7.5 percent of the median-involved accidents), there is no difference in the number of injury-producing accidents between the designs.

When these results are compared with the severity indexes shown in Figure 6, it can be seen that the severity for the paths in which the vehicle leaves the median are generally higher for the swale median design, although the only significant difference is for the crossover path.

To investigate the reason for this difference between the two median designs in accident severity for the crossover path, the accidents included in the crossover category were examined further. Although examination of the single-vehicle crossover accidents revealed no apparent reason for this difference in severity, it was noted that six head-on median-involved accidents occurred on sections with the swale median design, all of them involving injury, while no head-on median-involved accidents occurred on sections with the mound median design.

The effect of these head-on median-involved accidents on the safety potential of the median designs can be determined by an examination of several factors regarding the nature of the occurrence of head-on median-involved crossover accidents. Of primary consideration is the fact that a median-involved crossover accident becomes a head-on accident only if another vehicle is present in the opposing lanes and is struck during the crossover. The presence of a vehicle in the opposing lanes is a chance occurrence on which the median design has no effect. A second consideration is the effect of volume on the chance of a vehicle being present in the opposing lanes at the time of the occurrence of a crossover accident. During the examination of crossover accidents, it was noted that the swale median section on which the six head-on median-involved accidents occurred (I-71 between Columbus and Medina) experienced an average volume of 17,500 ADT over the study period. This volume is higher than that experienced on the other swale median section (14,033 ADT) or on the mound median sections (14,011 ADT). Although beyond the scope of the study to verify, this increase in volume and the resulting increase in head-on median-involved accidents imply that, as the volume on sections with the swale median design increases, the proportion of crossover accidents that result in head-on collisions also increases.

When the study data were being reduced, it was observed that many vehicles that crossed over the median also collided with a fixed object such as a guardrail, ditch, or fence when they came to rest off the roadway. On sections with the mound median design, the presence of the mound may tend to reduce the speed of the vehicle as it traverses the mound. This reduction in speed may result in a less severe accident if the vehicle collides with a fixed object before coming to rest. This is supported by the severity indexes shown in Figure 6, in which the severity index for the mound design is less than that for the swale design for the crossover path.

The relative safety potential of each median design must, however, be based on the largest possible portion of the accident phenomenon. Thus, for 89 percent of all median-involved accidents (the median and redirect paths), no difference exists between the two median designs in the number of injury-producing accidents. On this basis, the safety potential of the two median designs is equal in terms of the effects of median design and vehicle path on the number of injury-producing accidents.

The final question to be answered concerned the effects of median design on the roll-over tendencies of vehicles entering the median. Figure 7 shows the percentage of median-involved accidents by vehicle action for both median designs. A chi-square contingency test was again employed to determine if the proportion of median-involved accidents involving roll-over was different for the two median designs. The results of this test indicate that there is no difference between the two median designs in the number of roll-over accidents at the 5 percent significance level.

Because certain vehicle types may be more prone to roll-over than others—e.g., tractor-trailers versus passenger cars—the frequency of roll-over was computed for the various vehicle types, as given in Table 5. A chi-square contingency test was

Figure 7. Action of vehicles in median accidents.

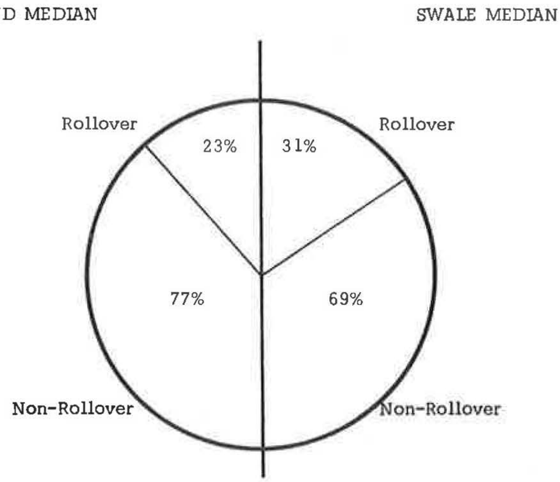


Table 5. Summary of vehicle path statistics for median-involved single-vehicle accidents.

Category	Crossover					Median					Redirect				
	PC	TT	C	O	TV	PC	TT	C	O	TV	PC	TT	C	O	TV
Mound															
Frequency															
No. by vehicle type	10	1	0	0	1	57	34	4	0	8	5	3	1	0	1
No. by path	12 →					103 →					10 →				
Percent of median type	9.6 →					82.4 →					8.0 →				
Injury accident[a]															
No. by vehicle type	4	0	0	0	0	28	10	3	0	1	3	0	1	0	0
No. by path	4 →					42 →					4 →				
Percent of median type	8.0 →					84.0 →					8.0 →				
Rollover															
No. by vehicle type	0	0	0	0	0	17	2	3	0	5	0	0	1	0	1
No. by path	0 →					27 →					2 →				
Percent of median type	0.0 →					93.1 →					6.9 →				
Swale															
Frequency															
No. by vehicle type	13	2	0	0	1	50	32	2	4	10	7	0	0	0	1
No. by path	16 →					98 →					8 →				
Percent of median type	13.1 →					80.3 →					6.6 →				
Injury accident[a]															
No. by vehicle type	11	2	0	0	1	20	7	2	2	1	5	0	0	0	1
No. by path	14 →					32 →					6 →				
Percent of median type	26.9 →					61.5 →					11.6 →				
Rollover															
No. by vehicle type	1	1	0	0	1	18	4	2	3	6	1	0	0	0	1
No. by path	3 →					33 →					2 →				
Percent of median type	7.9 →					86.8 →					5.3 →				

Notes: Vehicle Type: PC = Passenger car; TT = Tractor-trailer; C = Commercial (bus); O = Other (motorcycle); TV = Towed vehicle.
Vehicle Path: Crossover = Vehicle crossed median into opposing roadway; Median = Vehicle remained in median; Redirect = Vehicle redirected from median into orginal roadway.

[a]Injury accident category includes fatal accidents.

again employed to ascertain the significance of the difference in the number of roll-overs in each vehicle category between the two median designs. Low frequencies in certain vehicle categories required that all vehicles except passenger cars be combined into one group labeled "other". The results show no difference between the two median designs in the number of roll-overs for each vehicle category at the 5 percent significance level.

SUMMARY OF RESULTS

The results of the analysis can be summarized as follows:

1. In answer to question 1, there is a significant difference in the number of reported median-involved accidents between the two median designs. A disproportionately high number of single-vehicle median-involved accidents occur on sections with the mound median design. This implies that a greater number of nonreported (i.e., less severe) "incidents" of median encroachment occur on sections with the swale median design. However, documentation of this implication is not currently possible.
2. In answer to question 2, there is no difference between the two median designs in the number of injury-producing accidents.
3. In answer to question 3, first, there is no difference between the two median designs in the number of accidents for each vehicle path (crossover, median, or redirect); second, for both median designs, in approximately 81 percent of the median-involved single-vehicle accidents, the vehicle remained in the median; and third, when the numbers of injury-producing accidents for each vehicle path are examined, in 89 percent of the median-involved accidents (median and redirect paths), there is no difference between the two median designs in the number of injury-producing accidents.
4. In answer to question 4, first, there is no difference in the number of roll-over accidents between the two median designs; and second, there is no difference in the number of roll-over accidents for each vehicle type between the two median designs.

These results indicate a difference in the safety potential between the mound median design and the swale median design only in the area of the frequency of reported accidents. The difference in the number of head-on accidents cannot be used as a measure of safety potential since the number of crossover accidents was statistically equal for both median designs and since the element of chance determines the crossover collision occurrence. For all other factors analyzed, no difference between the two median designs exists.

CONCLUSIONS

On the basis of the results, the following conclusions concerning the safety potential of 84-ft (25.6-m) medians of the mound and swale designs can be drawn:

1. The 84-ft median of either cross-sectional design provides a generally adequate recovery area for encroaching vehicles.
2. Based on the disproportionately low frequency of reported accidents, the swale median design on 84-ft medians appears to provide more opportunity for encroaching vehicles to regain control and return to their roadway.
3. The use of either cross-sectional design with an 84-ft median does not have any effect on the primary path of the vehicle, on the vehicle's tendency to roll over, or on the resulting severity of the accident where the median encroachment results in a reported accident.

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DISCUSSION

H. L. Anderson, Office of Development, Federal Highway Administration

The report by Foody and Culp provides some interesting statistics and information to be considered in freeway designs. I have a number of reservations that may warrant review.

In the section on analysis and results the authors state that, since the geometrics of the sections of Interstate highways analyzed are similar except for the median design, it can be assumed that any difference in accident frequency would be the result of the median design. This assumption, although substantiated by references, cannot in my opinion be logically made when you consider the extremely large differences in accident rates on the sections involved. A rate of 11.86 accidents per mile for the swale design compared to 7.67 for the mound and a record of 29 fatal accidents for the swale compared to 12 for the mound and 13 fatal single-vehicle accidents compared to only 2 for the mound lead me to conclusions that are quite contrary to the authors'. Nor can the 10 percent additional ADT on the swale design in itself result in a significantly larger fatality rate and accident rate for the swale design. I believe the sections require a much more detailed analysis to account for the relative safety in the mound freeway sections. Certainly the percentage of trucks or the mix of traffic has a bearing on accident rates, and certainly vehicle speeds have a large bearing on severity indexes. An inventory and accounting of fixed objects, drainage ditches, guardrail, and other hazards should be studied. All of these must be considered in addition to median design and traffic volume, and therefore I think the assumption that median design is the only variable is generally an improper or incomplete one.

From Table 1 there is no basis for picking the very minor differences that exist in the median single-vehicle accidents and arriving at the conclusion that one is safer than the other, because the difference in accidents per million vehicle-miles is not significant. A difference of 0.02 accidents per million vehicle-miles is so small that a very few additional accidents one way or the other would have a radical effect on the accident rate. One snowstorm or icy condition can alter a difference this small. Of significance, however, is the total rate of 0.52 accidents per million vehicle-miles for the mound compared to 0.67 for the swale, a difference of 0.15. This leads me again to believe that a more thorough analysis of the differences between comparative sections is required. There must be a difference in the character of either the traffic or the roadway itself other than just the median. This was noticed in passing and discarded when the authors stated that most of the crossover accidents resulted in some injuries due to the striking of guardrails or other fixed objects when the vehicles did not strike each other.

Table 4 states that for the median path there is no difference in the number of injury-producing accidents between the two median designs. With this I agree—the differences are insignificant—if we can forget fatalities or equate them to being no worse than a broken finger or a bumped forehead. In my opinion the one advantage of wide medians or even alternative median designs is the elimination or reduction of fatalities. The swale design evidently produced 4 times as many fatalities within the median as did the mound design.

The authors state that there is no difference between the two designs in the number of injury-producing accidents for the redirected vehicle. Figure 6, however, indicates that the severity index for the redirected vehicle again is almost twice as high for the swale design as it is for the mound design. Thus, I cannot understand the conclusion drawn by the authors when they state that the only significant difference in the severity indexes is for the crossover path. Figure 6 indicates that the severity indexes for both the crossover and the redirected types of accident for the swale median were almost double those in the mound design. This again is borne out by the fatal accidents that occurred, where 29 people were killed in the swale median and only 12 in the mound median. It is again my opinion that much more analysis must be made on these significant differences. I agree with the authors when they state that the presence of a vehicle in the opposing lanes is a chance occurrence and is proportionate to the volumes of traffic in those lanes. I do not agree, however, that the chances of hitting another car in other lanes is a function of only the traffic volume. It is also a function of the speed of the vehicle and the angle at which the vehicle is crossing the path of the opposing traffic. It is entirely possible that the swale median might tend to flatten the angle at which the vehicle leaves the median and enters the opposing lane of traffic, whereas the mound design may have an opposite effect, thereby reducing the chances of head-on collision in the opposing lane. It should also be noted that in 13 percent of all accidents in the swale design the vehicle did cross the median, whereas only 10 percent crossed it in the mound design. This is a 30 percent increase in the crossover type and, except for the small sample size, is significant and again is discounted by the authors.

The purpose of the report was to answer four questions; however, three of the four questions were, in my opinion, either not answered or only partially answered. To question 2, Does the median design affect the severity of accidents involving the median?, the authors reply that there is no difference between the two designs in the number of injury-producing accidents. This is not a responsive answer to the question since there is a radical difference; the swale design severity and fatalities are considerably higher than those of the mound. Reference should be made to Figure 6.

Again, question 3, Does the median design affect the path of the vehicle after it enters a median during an accident?, is not answered, and the answer supplied for some of the accident information is evasive and incorrect. If in fact vehicle path was studied, it was not commented on in the report except to the degree that a vehicle crossed the median or was redirected.

To question 4, Does the median design affect the roll-over tendencies?, the authors' negative answer does not agree with their statistics in Figure 7 showing that 31 percent of vehicles entering the swale design median and 23 percent in the mound design rolled over. These percentage differences are significant.

In substance, I cannot agree with many of the conclusions of the authors or with much of the analysis and reasoning used. Ohio has more mileage and experience with these median configurations than any other highway organization. A vast amount of valuable information has undoubtedly been compiled in this study that is not available from any other source. I believe further review, analysis, and reporting would be worthy of consideration.

John C. Glennon, Traffic Safety Center, Midwest Research Institute

I would like to commend the authors on preparing this paper. They obviously put in a lot of effort.

My remarks are very brief. I do, however, have one major constructive criticism.

First, let me say that I find their conclusions 1 and 3 substantiated by the data. These conclusions say, first, that both median designs provide an adequate recovery area and, second, that both median designs exhibit similar characteristics of vehicle path, tendency for roll-over, and accident severity. One caution to the reader on the recovery conclusion, however, deals with their definition of recovery. What the conclusion implies is that, with an 84-ft median, very few vehicles encroach on the opposing lanes.

Their second conclusion is the one I question. This conclusion, based on a chi-square contingency test, touts the swale median design as providing the better opportunity for an encroaching driver to regain control. This conclusion may be incorrect because of the possible invalidity of their basic implicit assumption for the contingency test. This assumption is that the comparison samples are identical in every way except for median design and ADT.

If this assumption is valid, then a much more important conclusion has been overlooked. That is, the mound median is significantly better than the swale median. This conclusion would be based on the significant difference in total accident rates for the comparison samples as substantiated by the data. The accident rate was 0.52 for the mound median and 0.67 for the swale median.

I suspect the more likely possibility is that the basic implicit assumption is invalid.

Even though the comparison samples have significantly different rates, this difference is probably due to parameters other than median design. My questions are

1. Are the percentages of truck traffic similar?
2. Are the frequency and rigidity of roadside fixed objects similar?
3. Are the peaking characteristics on the sections making up the comparison samples similar?
4. Are the distributions of ADT within each comparison sample similar?
5. Are the combinations of other geometric features similar?

What I suggest is that the authors need to answer these questions and others to prove or disprove the validity of their basic implicit assumption. If, in fact, the assumption is valid, then the very important conclusion is that the mound median is significantly better than the swale median.

AUTHORS' CLOSURE

The basic questions raised by Anderson's and Glennon's reviews of this paper can be summarized as follows:

1. Given the differences in the total accident rate and the overall fatal accident frequency (Table 1), is the assumption correct that the two study groups (mound median sections and swale median sections) are equivalent except for the median design and the ADT? If this assumption is not correct, then is the conclusion correct that there are more unreported median encroachments in sections with the swale median design, thereby indicating that it is more conducive to driver-vehicle recovery?
2. Given the severity index figures (Figure 6) and the fatal accident frequency for median-involved single-vehicle accidents (Table 1), is the conclusion correct that the injury-producing potential of the two designs is not different?
3. Given the distribution of median-involved accidents by vehicle path (Figure 5), is the conclusion correct that there is no difference in the effect that each median design has on the vehicle path?
4. Given the percentage of roll-overs for each median design (Figure 7), is the conclusion correct that there is no difference in the effect that each median design has on the tendency for vehicles to roll over during a median-involved accident?

The objective of this study was to draw conclusions about the safety potential of each median design based on the analysis of the frequency and the severity of median-involved, single-vehicle accidents. The first question raised in the discussion refers to the analysis of the frequency of median-involved accidents. In analyzing the frequency of accidents involving the median, we did not merely compare the number of reported accidents per million vehicle-miles of travel for the two study groups for several reasons. First, it was known that the volume levels were different for the two groups. Second, it was possible that other unknown differences (such as percentage of truck traffic and frequency of roadside objects off of the right-hand side of the road) also existed. Third, it was known from the previously cited research by Hutchinson et al. that the number of vehicles leaving the roadway for a section of highway is proportional to the volume but not equal to the number of reported accidents. Given this fact, it was assumed that, if the two median designs had different effects on the vehicle encroaching the median, then the ratio of encroachments to reported accidents could be affected. Our intention was to obtain a measure of the effect of all accident-causative factors existing on the mileage within each study group. Therefore, in comparing the reported accident frequency distribution by accident type for the two study groups, it was assumed that the known factors (ADT) and unknown factors (percent truck traffic, etc.) had different order effects on the two study groups (mound versus swale), but that the effect within each study group was the same for the three types of accidents: multivehicle accidents (occurring on the roadway), non-median-involved single-vehicle accidents (occurring to the right of the roadway), and median-involved single-vehicle accidents (occurring to the left of the roadway). Implicit with this approach was the assumption that the 84-ft-wide median made the effect of oncoming traffic negligible as a primary factor in a multivehicle accident or in a vehicle initially leaving the roadway. Therefore, in comparing the distributions of reported accidents by accident type, it was assumed that differences between the distributions would be a direct indication of the effect of the median design since all the mileage included in the two study groups was constructed to Interstate standards, with the median cross section being the only pronounced difference. The validity of this approach would require that the cross section of the right-of-way beyond the right-hand shoulder and the frequency of fixed objects be shown to be no different. Information of this nature was not and is not available, and thus the validity of this assumption cannot be tested. Therefore, we agree with both discussants that the results of the analysis of the reported accident frequency by accident type do not clearly establish the validity of the conclusion that the use of the swale median design provides more opportunity for encroaching vehicles to regain control and return to the roadway.

However, we must also disagree with the implication by both discussants that the mound median design is superior to the swale median design based on subjective comparisons of the total accident rate and fatal accident frequency. The interpretation of the data in this manner requires the assumption that the median design has a major effect on the frequency and severity of the non-median-involved accidents, both multivehicle and single-vehicle. Previous research by Kilhburg and Tharp (4) established that the total accident rate increases with increasing ADT, as the result of a large increase in the frequency of multivehicle accidents and a leveling off of single-vehicle accident frequency. These research results tend to explain the difference in the multivehicle accident rate, and, since no other research results were found that supported the assumption that an 84-ft median has a major effect on the total accident rate, we question the validity of drawing conclusions about the effect of median design based on the total accident rate.

Our reply to the second question raised in the discussion is directed to Anderson's comments regarding the severity of median-involved accidents. Anderson states that he "... cannot understand the conclusion ... that the only significant difference in the severity indexes is for the crossover path" and that the severity indexes shown in Figure 6 indicate a difference between the two designs for the redirect path as well. Anderson apparently arrived at this deduction through a subjective evaluation of the data presented in Figure 6 and an inappropriate reference to the total number of fatal accidents.

In the study, the conclusion that there was no difference between the two designs with respect to the effect of median design on the severity of median-involved accidents was based on the results of two separate, objective analyses. In the first analysis (Table 3), it was found that there was no difference in the proportion of accidents resulting in injury between the two designs for all reported accidents involving the median. This test was then repeated for the data in each of the three vehicle path categories, once it was established that there was no difference in the proportion of accidents for each vehicle path between the two designs (Table 4). Tables containing the raw data used in these three tests were not given in the report to conserve space but can be generated by multiplying the severity indexes shown in Figure 6 by the total accident figures for each vehicle path shown in Table 4. The results of these tests indicated that only for the crossover path was there a difference in median-involved accident severity between the two designs. This difference (involving only 11.5 percent of all median-involved accidents) must be balanced against the fact that no difference in severity exists for the remaining 88.5 percent of the median-involved accidents (median and redirect categories). Therefore, it is our opinion that the conclusion of the study concerning the severity of accidents involving the median is valid since it is based on objective, mathematical analyses.

The third question raised in the discussion refers to the analysis of the path of the vehicle after entering the median during an accident. Our response is directed to Anderson's comments questioning whether or not vehicle path was actually studied and stating that the results of the vehicle path analysis were not properly interpreted. As stated in the section on procedure, each police report for all median-involved accidents was manually reviewed and was classified into one of the three vehicle paths explicitly defined in the text of the report. The distribution of vehicle paths for all median-involved accidents for both designs was compared mathematically (Table 4) and found to be the same. The calculation that there was a 30 percent difference in the percentage of crossovers between the mound design (10 percent) and the swale design (13 percent) is not valid in that percentages taken from two different bases cannot be manipulated to form a valid third percentage. Therefore, it is our opinion that the conclusion reached in the study regarding the effect of median design on the path of vehicles entering the median during an accident is complete and correct since it was based on valid, mathematical analysis.

Although not specifically directed to a conclusion of the study, we feel that it is appropriate to respond to Anderson's comment regarding the occurrence of a head-on median-involved crossover accident. Anderson agrees with our statement that the presence of a vehicle in the opposing lanes is a chance occurrence that is unaffected by median design. He also implies that we reduced this chance occurrence to a function of only traffic volume. We did, however, offer traffic volume as one factor that may influence the crossover collision, realizing that an analysis of vehicle speed and encroachment angle, as suggested by Anderson, was far beyond the scope of this study and the data available to us.

The final question raised in the discussion refers to the analysis of vehicle roll-over frequency for median-involved accidents. Anderson's question concerns the evaluation of the proportion of roll-overs for the two median designs as shown in Figure 7. He states that the difference between the percentage of roll-overs for the mound design (23 percent) and the swale design (31 percent) is significant, but offers no mathematical justification to support this statement. However, the objective, mathematical analyses contained in the report indicate that not only is there no difference in the proportion of roll-over accidents between the two designs but that there is no difference in the proportion of roll-overs by vehicle type between the two median designs as well. It is therefore our opinion that the conclusion reached in the study with respect to the effect of median design on the roll-over tendency of vehicles in median-involved accidents is correct since it was based on established analytical procedures.

In summary, we agree with the discussants that the conclusion based on the analysis of the frequency of occurrence of median-involved accidents is not fully supported, since the assumption that the roadside design (to the right of the roadway) is the same for the two study groups cannot be documented. It is our opinion, however, that this assumption is less strenuous than the alternate assumption offered by the discussants: that the 84-ft median has a major effect on the total accident rate. With respect to the questions regarding the differences in the effect of the two median designs on the severity, vehicle path, and vehicle roll-over tendency for median-involved accidents, it is our opinion that the conclusions presented and supported in the report are correct.

We commend Anderson and Glennon for the thoroughness of their review efforts. We appreciate this opportunity to clarify those portions of our report that were subject to misunderstanding.

REFERENCE

4. Kihlberg, J. K., and Tharp, K. J. Accident Rates as Related to Design Elements of Rural Highways. NCHRP Rept. 47, Highway Research Board, 1968.

CASE STUDIES OF WRONG-WAY ENTRIES AT HIGHWAY INTERCHANGES IN VIRGINIA

N. K. Vaswani, Virginia Highway Research Council

An evaluation of a 2-year survey of wrong-way driving led to on-site investigations of a number of intersections and interchanges. The investigations showed a consistent pattern in wrong-way entry incidents that related to road geometrics, markings, and signs. Based on the findings of the investigations, five case studies were developed to show the effects of these variables. This paper discusses the results of the survey, some case studies, and measures for preventing wrong-way entries at selected interchanges. Some of the recommendations made are as follows: Channelize the left lane of the exit ramp and remove the left end flare; investigate the effectiveness of stop lines, the continuation of pavement edge lines across exit ramps, and the use of continuous double yellow lines; through the use of signs, provide intersection geometry information for drivers entering a 4-lane divided highway; and provide additional pavement marking and spotlighting to supplement signs.

•THE object of this investigation was to determine means for alleviating the problem of wrong-way driving on 4-lane divided highways. The information considered in the investigation was obtained from a 25-month survey of incidents of wrong-way driving on Virginia's divided highways and investigations of the physical aspects of sites at which wrong-way incidents had occurred within the past 3 years.

EXTENT OF PROBLEM

Table 1 gives a comparison of accidents involving wrong-way driving with total accidents for the period covered by the 25-month survey. These data show that the accidents involving wrong-way driving are only 0.1 percent of the total accidents. But for each wrong-way accident on Interstate highways, 0.47 and 1.18 persons were killed and injured respectively; these figures represent 27.4 (2,740 percent) and 2.81 (281 percent) times the deaths and injuries resulting from other types of accidents. These facts emphasize the need for highway improvements that are not very expensive and do not impede motorists other than the 0.1 percent wrong-way drivers.

Wrong-way driving surveys performed in California (1), Michigan (2), Missouri (3), and Texas (4) have observed the same trend as found in Virginia.

EVALUATION OF THE WRONG-WAY DRIVING SURVEY

The parameters determined in the wrong-way driving survey in Virginia up to December 31, 1972, were examined in detail. These parameters were driver's age, time of day, day of week, weather, lighting condition (daylight or darkness), and location of the wrong-way entry. The most important observations from the data are discussed in the following.

Drunkenness and Darkness

Darkness combined with drunkenness of the driver accounted for 2.3 to 4 times the

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number of incidents that occurred during the daytime (Table 2). This is contrary to the pattern obtained in the case of non-drunken drivers, where the daytime incidents exceed the nighttime incidents (Table 3).

Partial Interchanges

The evaluation showed that on Interstate highways most of the wrong-way entries occurred at partial interchanges through an exit ramp from the Interstate onto the crossroad. [In a partial interchange (e.g., a diamond type with four ramps), although the cross-traffic at grade is eliminated, all or some of the left-turn movements cross the path of other vehicles, as compared to no such crossing on a full interchange (e.g., cloverleaf).] No report in the survey showed that a wrong-way entry was made through the entry ramp, i.e., the ramp leading from the crossroad to the Interstate road.

The incidents of wrong-way driving on Interstate highways were broken down into four major categories, as given in Table 4. This table shows that 47 percent of the cases of wrong-way driving resulted from entries at interchanges, the origins of 37 percent were unknown, 15 percent resulted from U-turns, and very few (none in the last survey period of 1972) originated at crossovers.

The reason that exit ramps on partial interchanges generate wrong-way entries is that these ramps, unlike the ones on non-partial interchanges that converge with right-hand traffic, meet the crossroad at about 90 degrees to accommodate both left and right turns (5, 6). Because of this design, the three wrong-way entries shown in Figure 1 are possible.

Intersections With 4-Lane Divided Highways

The evaluation of the survey data on divided arterial and primary roads showed that 45 percent of the wrong-way entries were at their intersection with exit ramps or secondary roads. All such wrong-way entries were due to left-turning vehicles making an early left turn rather than turning around the nose of the median.

The data showed that, of 19 incidents at the intersections with exit ramps, 18 involved non-drunken drivers. For secondary roads, also, the non-drunken driver rate was higher than for drunken drivers. This finding stresses the need for improvement of highways rather than drivers.

Repeatability

The survey showed that repeated wrong-way entries from a given ramp are very rare, so it seems that any partial interchange is as prone to a wrong-way entry as any other. Therefore, the preventive techniques adopted should be sufficiently economical that they could be used for all interchanges.

CASE STUDIES

In Virginia the longest and most heavily trafficked Interstate routes are I-95 and I-81. I-95 has more than 5 times the traffic of I-81 but has a history of fewer wrong-way driving incidents. Since the beginning of the wrong-way driving survey, more attention has been paid to reducing wrong-way entries on I-95 and I-81. The wrong-way entries on I-95 have varied between 6 and 8 for each 6-month period since 1970. On I-81 they have been reduced from 22 in 1970 to 14 in 1972 on a semiannual basis.

For this presentation, five interchanges on these two Interstate routes were chosen for case studies. These interchanges illustrate most of the design drawbacks noted on the various other interchanges for which wrong-way entries were reported. As a result of the on-site investigations of some interchanges for which no wrong-way incidents have been reported and some that have been modified since a wrong-way incident was reported, certain suggestions and recommendations for improvement have been made.

CASE STUDY 1: INTERCHANGE 43 ON I-81

Interchange 43 is of the diamond type and intersects a 2-lane crossroad. The cross-

Table 1. Total accidents compared with wrong-way driving accidents.

Category	No. of Accidents	Fatalities Per Accident	Injuries Per Accident
Interstate Roads			
All accidents, 1970 and 1971 (24 months)	14,862	0.016	0.42
Wrong-way accidents			
25-month survey	55	0.47	1.18
Percent of all accidents	0.4	2,740	281
All Roads			
All accidents, 1970 and 1971 (24 months)	133,065	0.014	0.41
Wrong-way accidents			
25-month survey	138	0.22	1.03
Percent of all accidents	0.1	1,570	250

Source: Summary of Accident Data: State's Highway Systems, 1970 and 1971, Virginia Department of Highways, and unpublished data compiled by the Department of Highways and Department of State Police.

Table 2. Day and night wrong-way incidents by drunken drivers.

	Interstate		Divided Arterial and Primary	
Time	No.	Ratio	No.	Ratio
Daylight	22	1.0	23	1.0
Darkness	51	2.3	93	4.0

Table 3. Day and night wrong-way incidents by non-drunken drivers.

	Interstate		Divided Arterial and Primary	
Time	No.	Ratio	No.	Ratio
Daylight	24	1.0	112	1.0
Darkness	19	0.79	59	0.53

Table 4. Places of wrong-way entries on Interstates.

Place of Entry	No.	Percent
U-turn	21	15
Crossover	2	1
Interchange	66	47
Unknown	51	37
Total	140	100

Figure 1. Wrong-way entry and egress on left lane of exit ramps.

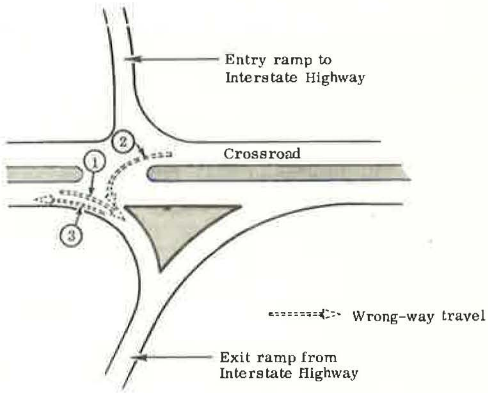


Figure 2. View from crossroad, with exit ramp on right, where wrong-way entry took place.

road carries about 6,000 vpd. All necessary "one way", "do not enter", "wrong way", and "no right turn" signs are provided on the exit ramps from I-81. Yet the wrong-way entry was made through an exit ramp and by a sober driver.

This interchange was chosen for study because it is typical of others for which wrong-way entries have been reported. It is similar to many others with respect to geometrics, signs, markings, and construction practices. The intersection of the exit ramp with the crossroad is shown in Figure 2. This photograph was taken from the crossroad and shows the left lane of the exit ramp (which the wrong-way driver entered) meeting the crossroad at a right angle.

As a result of inspections of this interchange and others like it, it is believed that they could be improved by three fundamental changes as discussed below.

Elimination of Unnecessary Flares

During this study it was observed that on almost all interchanges on which wrong-way entries had been made into the exit ramp or from the exit ramp into the crossroad, the left corner of the left lane of the exit ramp flared into the right pavement edge of the crossroad. An example of a flared end is shown in Figure 2.

Such a flared end (termed "flare" hereafter) provides for a very easy but incorrect right-hand turn. It is therefore possible that it would induce a driver to make a wrong-way entry from the crossroad into the exit lane. For a sharp, right-angled junction, the driver would have to reduce his speed and almost come to a stop before maneuvering into the exit lane.

Similarly, a driver coming upon the left flare from the exit ramp could be encouraged to make an improper left turn into the wrong lane of the crossroad. Again, a sharp, right-angled bend would not permit an easy left turn.

The site inspections showed that, where the flare is not provided and the left lane of the exit ramp and the passage through the median are channelized, no wrong-way entry to or egress from exit ramps has been reported. It was also noticed that, on most interchanges with 4-lane divided crossroads that included flares, the flares had collected dust, which indicated their disuse by properly maneuvering drivers. (A good example of this is given in case study 3 where channelization to prevent wrong-way driving is discussed.)

These flares may have been provided either as a matter of construction expediency or the design requirement for a left-turn curve from the exit ramp to the crossroad. It is recommended that the designs be checked and the flares be removed or their use be prevented when they have been provided to satisfy the left-turn curve requirements.

To discourage this type of wrong-way entry, pavement marking at the corner of the left lane of the exit ramp could be provided as shown in Figures 3 and 4. To make this turn difficult to negotiate, or to prevent the use of the shoulder, a physical barrier could be provided along lines AB and BC in Figures 3 and 4.

Stop Line

The exit ramp has one-way traffic and on all partial interchanges the traffic must stop at a stop sign and/or a stop line before entering the crossroad. During the site investigations it was observed that many of the exit ramps involved in wrong-way entries onto the crossroad or the Interstate highway did not have stop lines at the junctions.

The stop line probably has the following two advantages: First, more drivers tend to stop for a stop line and a stop sign than for a stop sign only; while stopped, the driver is likely to observe the signing and road layout before entering the crossroad. Second, the stop line also may discourage a driver from the crossroad from entering the exit ramp. It is recognized that these two observations do not provide conclusive evidence that the provision of stop lines would discourage wrong-way entries and that further consideration of this subject is needed.

During the investigation it was found that at two intersections the stop line was closer to the edge of the crossroad than the minimum distance specified in the Virginia Manual (7). This is an improvement because the line is clearly visible at a considerable dis-

Figure 3. Suggested improvements of exit ramp shown in Figure 2 by marking pavement in flared corner, providing a stop line, and continuing the pavement edge line across the exit ramp junction.

Figure 4. Suggested improvements of exit ramp shown in Figure 2 by marking pavement in flared corner and providing a very thick line—minimum of 24 in. (0.6 m)—with its outer edge in line with the pavement edge line.

Figure 5. View of crossroad from exit ramp from which six wrong-way entries were made onto the crossroad.

Figure 6. Suggested improvements on exit ramp and crossroad shown in Figure 5 by improving sight distance for left turns, providing intersection geometry sign at X, moving stop line closer to edge of crossroad, marking left corner flare, decreasing width of crossover, providing median nose delineators, and bringing signs on median closer to nose.

Figure 7. Recommended striped median.

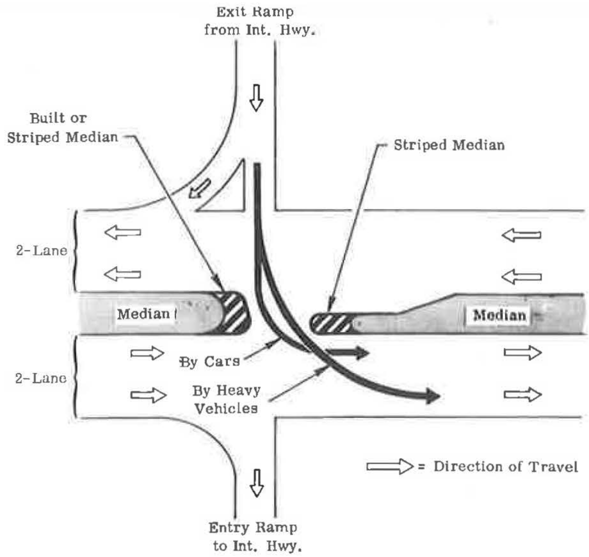


Figure 8. Suggested improvement on exit ramp shown in Figure 5 by use of intersection geometry sign.

tance from both lanes of the crossroad. If the stop line is brought up to the edge of the crossroad and in line with the edge line of the crossroad (as discussed below), it might completely deter drivers from entering the exit ramp. If such a stop line is provided, it should be at least 24 in. (0.6 m) wide with a stop marking to provide enough clearance between vehicles on the crossroad and vehicles stopped at the line.

Continuation of Pavement Edge Line Across Exit Ramp Junction

Drivers have now become so accustomed to pavement edge lines that they subconsciously use them as a guide. It is felt that if the edge line were continued across the junction of the crossroad with the exit ramp it would make the exit ramp less conspicuous to a driver on the crossroad for the following reasons: First, a person whose attention to the driving task is impaired might, as a matter of habit, still use the edge line for guidance and thus not cross it for a wrong-way entry onto the exit ramp. In fact, it is possible that if the edge line follows the flare into the exit ramp, as is sometimes the case, an impaired driver would follow the edge line so scrupulously that he would turn with it into the exit ramp. A normal driver will have less chance of doubling his mistake by crossing the edge line and getting into the exit ramp. Second, if it is true that the stop line discourages drivers from entering the exit ramp from the crossroad, the continuation of the pavement edge line would prove to be more effective because it would be nearer the driver.

Hilton (8) also recommends continuing the edge marking across intersections adjacent to bridges, even though the traffic is both ways across such intersections.

It is therefore recommended that continuation of the pavement edge line—as shown in Figure 3—be tried. Continuation of the pavement edge line of the crossroad across the exit ramp would conform with the principle followed in continuing it across the deceleration and turning lanes from primary, arterial, and Interstate highways.

CASE STUDY 2: INTERCHANGE OF I-95 SOUTH AND ROUTE 1

The junction of I-95 South and Route 1 is a diamond interchange linking an Interstate highway with a 4-lane divided crossroad that has two additional left-turning lanes. The crossroad carries about 5,000 vpd.

This interchange was chosen because it is the only one in Virginia that had as high as six wrong-way entries onto the crossroad reported in the survey. Two of these six incidents happened between 10:30 and 11:30 a.m., and the other four during hours of darkness. All the entries were made by sober drivers, all were made from the same exit ramp, and all resulted from the drivers making their turns before rounding the nose of the crossroad median. As mentioned before, this type of maneuver accounts for 45 percent of the wrong-way entries in Virginia. The photograph of this intersection, shown in Figure 5, was taken from the exit ramp meeting the crossroad at its right.

In addition to the many geometric features that might be involved, it is possible that low visibility and a restricted sight distance may have contributed to the numerous wrong-way entries. The crossroad has a curve on the left of the exit ramp and the exit ramp is in a deep cut. To increase the sight distance up the crossroad from the exit ramp, the stop line should be brought closer to the edge of the crossroad, as recommended in case study 1. Figure 6 shows a suggested revised location of the stop line and the marking of the flared end at this junction.

The "do not enter" and "wrong way" signs, as can be seen in Figure 6, are placed very far from the ends of the medians of the 4-lane divided crossroad and are not easily read from the junction. Figure 6 also shows the setback of the nose of the medians (not considering the extension marked in white) from the exit ramp. This setback, and hence the width of the crossover, seems to be too long and should be reduced to minimum requirements. If this width cannot be reduced, pavement nose markings as shown in Figure 6 (by a white mark) and in Figure 7 would help. The nose markings should be applied to provide the minimum width of crossover needed for lighter vehicles, which form a large percentage of the total traffic.

The information-decision-action (IDA) sequence developed by Taylor and McGhee (9) shows that, for a left turn, nine actions are needed. In order to execute these actions the driver needs the following information: (a) destination/direction, (b) advance warning of intersection, and (c) intersection geometry. Preferably this information should be given to the driver during his first action, i.e., in the "approach vicinity of the intersection". In the present system of signing, drivers are unaware of the intersection geometry, and while taking the third action, i.e., "entering the appropriate lane", some make a faulty maneuver and enter the wrong lane. It is therefore necessary that the driver be supplied information on the intersection geometry before he takes the third action. Figure 8 shows one example of an intersection information sign to replace the direction sign shown in Figure 5. An enlarged view of this intersection information sign is shown in Figure 9. An alternative arrangement is to provide a sign such as shown in Figure 9 at the corner of the exit ramp shown by X in Figure 6.

The possibility of spot illumination of the far lane to help drivers make a left turn could also be considered. An example of such spotlighting is shown in Figure 10. This spotlighting would also illuminate the entry ramp junction and thus make it more conspicuous so as to reduce the likelihood of its being missed by normal as well as impaired drivers.

CASE STUDY 3: INTERCHANGE 53 ON I-81 SOUTH

Interchange 53 is of the diamond type and connects I-81 with a 4-lane divided crossroad. The crossroad carries about 2,000 vpd on the north side and about 4,000 vpd on the south side. Two wrong-way entries (both by sober drivers) have been made onto the crossroad from the exit ramp by drivers turning too early rather than turning around the nose of the median.

The interchange was chosen mainly to emphasize the need for channelization on the left lane of the exit ramp. In this case, as shown in Figure 11, "do not enter", "one way", and "wrong way" signs—to discourage wrong-way entry from the exit ramp for left-turning vehicles—are not provided on the median.

Channelization to prevent wrong-way entries involves four elements: (a) elimination of flares, (b) minimum width of the left lane of the exit ramp, (c) minimum width of the junction of the left lane of the exit ramp with the crossroad, and (d) physical barriers along the pavement edge.

Elimination of Flares

The disadvantages of flares were discussed in case study 1. The present case study shows that on 4-lane divided crossroads the flares are not in use and have been found to collect dust. Figure 11 shows at A the left lane of the I-81 exit ramp. A dark patch in the flared corner shows the collection of dust, which exemplifies its disuse.

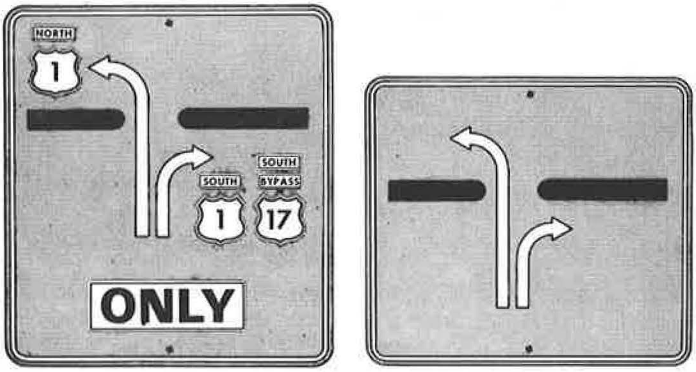
Minimum Width of Left Lane of Exit Ramp

Generous widths of the exit ramp at its junction with the crossroad make wrong-way entry onto or egress from the exit ramp easy; narrow pavement widths will discourage such entries. Figure 12 shows an excess width by the dark patch on the right side of the lane. This patch has collected dust, which indicates its disuse. Such excessive widths could be striped to discourage their use for wrong-way entries.

Minimum Width of Junction of Left Lane of Ramp With Crossroad

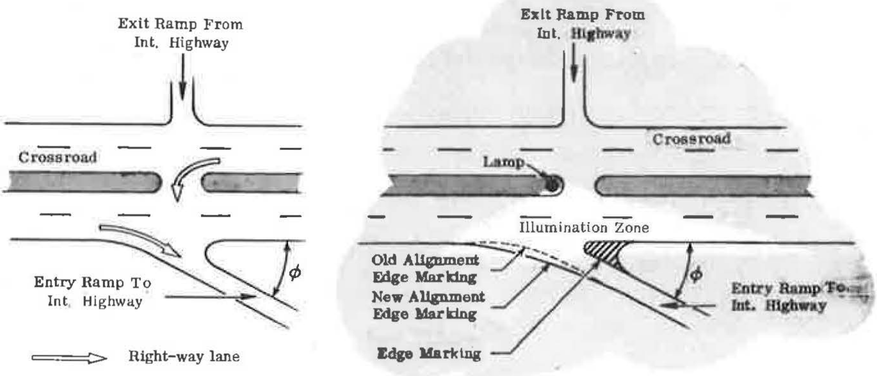
A right-angled junction of the left lane of the exit ramp with the crossroad, without a flare, would reduce wrong-way entries and exits. This design would provide a minimum width of the left lane of the exit ramp and make it difficult for a driver from the right lane of the crossroad to maneuver onto the ramp. Most of the left lanes are at right angles with the crossroads; hence, after the flare is removed, the minimum width would automatically be obtained. An example is shown in Figure 12.

Figure 9. Recommended geometry signs for installation on exit ramps to 4-lane divided crossroads; provide sign (b) on left corner as shown by X in Figure 6.



(a) (b)

Figure 10. Recommended improvement of an entry ramp.



(A) Before improvement. (B) After improvement.

Figure 11. View of exit ramp with its left lane and junction with crossroad, marked A. Note the dark patches of unused pavement at flare and left edge and absence of one-way, do not enter, and wrong-way signs for the crossroad.

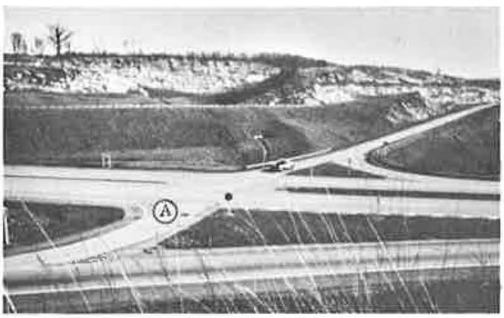
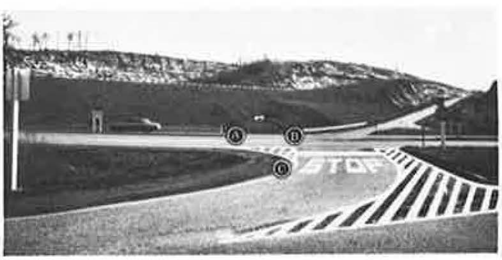


Figure 12. Suggested improvements of left lane shown in Figure 11 by channelizing left lane by marking or by providing physical barrier along ABC and reducing pavement width, providing stop line, continuing pavement edge line of crossroad across exit ramp, providing missing signs, and providing geometry sign shown in Figure 9(b).



CASE STUDY 4: INTERCHANGE 49 ON I-81 SOUTH

Case study 4 involves a diamond interchange over a T-junction. The crossroad carries about 2,000 vpd. No marking is provided to divide the two lanes of the crossroad. At the time of the wrong-way entry, the crossroad had a stop sign for vehicles turning into the entry ramp and the exit ramp did not have a stop sign at its junction with the crossroad, as shown in Figure 13. This stop sign system is unusual.

This interchange was chosen for study for two reasons: First, placement of the stop sign on the crossroad instead of the exit lane is unusual, and second, provision of the flare encourages a right-hand turn for a wrong-way entry from the crossroad into the exit ramp. (It is noted that in the accident resulting from the single wrong-way incident at this intersection, two people were killed and five injured.)

Figure 13(a) shows the plan of the T-junction at the time of the accident. At that time there was no pavement marking and the "no right turn" sign was so low that it could be hidden by a car coming from the exit ramp on the right across the field of vision of the driver intending to turn left onto the entry ramp for the Interstate highway.

The signing system has since been changed and pavement marking provided as shown in Figure 13(b). This junction is now less likely to be the scene of wrong-way entries.

Provision of arrows is recommended as an improvement on the modified marking system as shown in Figure 13. The recommended improvement of the modified system also includes the provision of the "no right turn" sign within the cone of the driver's vision and in such a place that the driver sees it at the time he can most effectively utilize the information it imparts. There appears to be a need for a revision of the specifications for the location of signs in the Virginia Manual (7). The revised specifications should be based on the cone of vision and the size and effectiveness of the sign.

CASE STUDY 5: INTERCHANGE 33 ON I-81

Interchange 33 is also of the diamond type. It connects I-81 with a 2-lane crossroad that carries about 1,700 vpd. The lanes on the crossroad are separated by double yellow lines that have openings for left turns and for through traffic from the exit ramps to the entry ramps. The crossroad and the exit ramps are fully furnished with the necessary "one way", "do not enter", "wrong way", "no right turn", and "no left turn" signs. Both the exit ramps and entry ramps are divided into right and left lanes by islands at their junctions with the crossroad. Except for the signs, the details, including pavement markings, are shown in Figure 14.

This interchange was chosen for two reasons: (a) the need for modifications in the use of double yellow lines and (b) the need for emphasizing stop lines. It should be noted that in the accident resulting from the single wrong-way incident at this intersection, six people were killed and one was seriously injured.

Figure 14 shows a photograph of the junction of the right-hand lane of the exit ramp and the crossroad, with the opening between the yellow lines. A drunken driver coming from a gas station went through this opening into the right lane of the exit ramp as shown by the arrow superimposed on the photograph.

If there had been no gap in these yellow lines the driver may not have crossed them. Further, if a stop line were provided at the junction of the right lane of the exit ramp and the crossroad it might have further discouraged this driver from entering the exit lane. The yellow line and white line are shown in Figure 15.

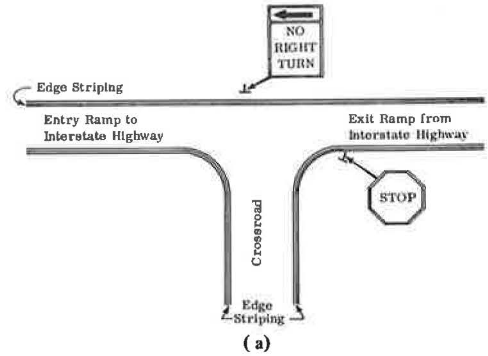
The following improvements are recommended.

Double Yellow Lines on 2-Lane Undivided Crossroad

Many 2-lane undivided crossroads at interchanges have been provided with double yellow lines to separate the lanes. Whenever these lines have been provided on the crossroad of an interchange, openings in the lines like the one shown in Figure 14 have also been provided to guide turning or crossing vehicles. It seems that when providing these openings the possibilities of wrong-way entries were not considered.

A scheme for the use of double yellow lines to discourage wrong-way entries by a left turn from the crossroad into the exit lane is shown in Figure 16, where only two entries

Figure 13. Original and modified marking and signing at Interchange 49: (a) at the time of the accident; (b) after the accident; (c) recommended improvement in the modified marking and signs.



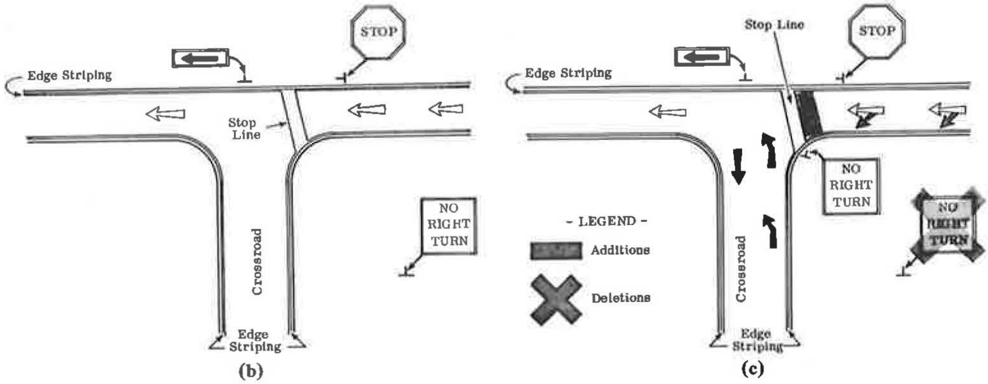


Figure 14. The driver went through the opening in the double yellow lines and entered the wrong way through the right lane of the exit ramp as shown by the arrow.

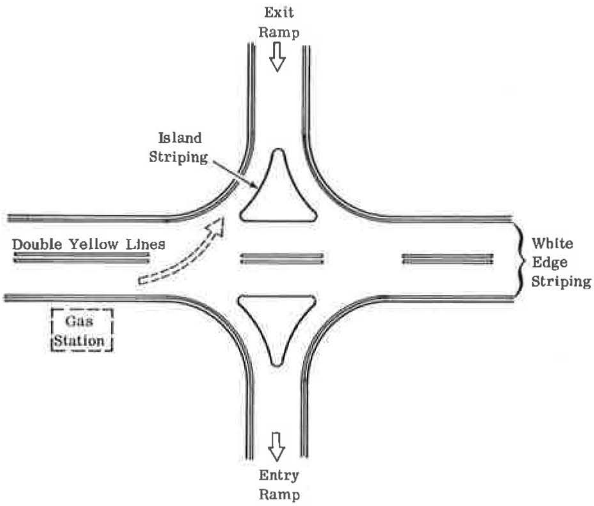


Figure 15. Suggested improvement on the exit ramp shown in Figure 14 by continuing the yellow lines and providing about a 24-in.-wide stop line. The island at the entry ramp is unnecessary.

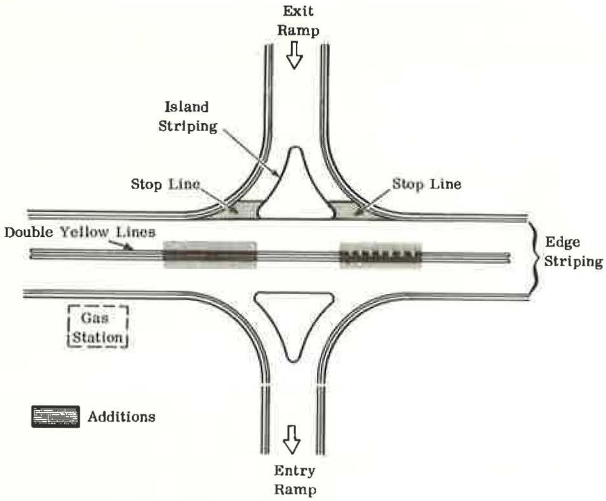
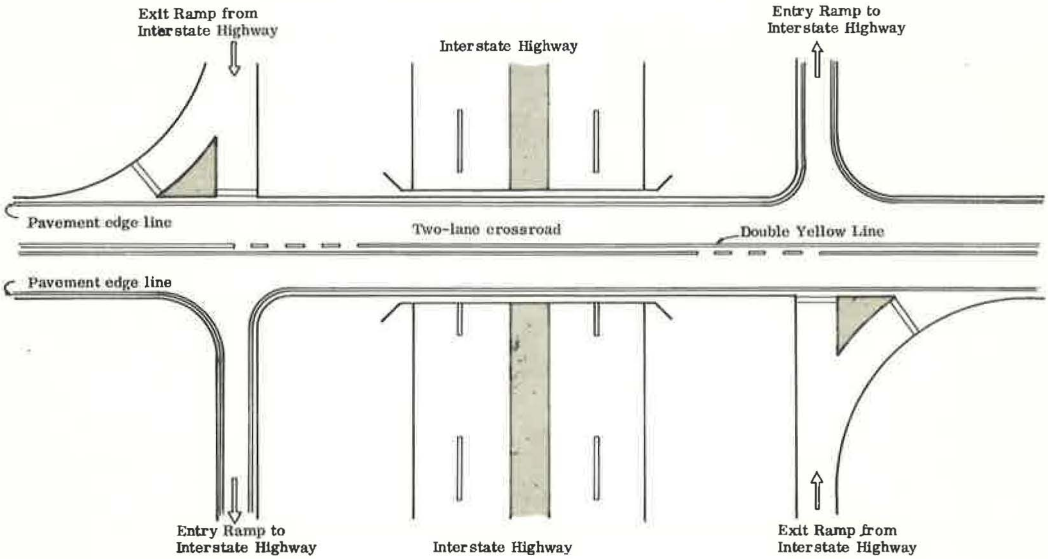


Figure 16. Recommended marking on undivided crossroad.



are provided for left turns. The openings made for these entries extend up to a point facing the center of the left lane of the exit ramp. In the opening provided, only one line is broken for the left turn while the second line is solid. Yellow lines thicker than the normal width might increase the effectiveness of the markings. If provision is to be made in the yellow lines for direct connection between the exit and the entry ramp across the crossroad, a slight adjustment in the position of the broken yellow line might sometimes be necessary.

For new designs, the need for a dividing island at the junction of the entry ramp with the crossroad should be carefully examined.

FINDINGS

The following findings are based on visual observations and evaluations of several interchanges and intersections on highways in Virginia:

1. Aids in addition to signs are necessary on crossroads and their junctions with exit and entry ramps.
2. Since non-drunken drivers were involved in most of the wrong-way entries by left turns from the exit ramps onto the divided highway, improvements at such junctions are needed to guide normal drivers.
3. Use of flared corners at the junction of the left side of the left lane of the exit ramp with the right edge of the crossroad should be discouraged or prevented.
4. Evaluations should be made of the effectiveness of stop lines and continuation of the pavement edge lines across exit ramps. In fact, the provision of a very wide (24-in. minimum) stop line with its edge on the side of the crossroad in line with the crossroad pavement edge line is recommended for evaluation.
5. "Intersection geometry" signs might considerably help drivers maneuver around the nose of the median when making a left turn from an exit ramp into a 4-lane divided highway.
6. At interchanges, spotlighting at night could be used as a driver aid.
7. The left lane of the exit ramp should be channelized by (a) providing a minimum-width left lane and (b) providing a minimum width at the junction of the left lane and the crossroad.
8. Specifications for the location of signs based on their size and the cone of vision should be developed and incorporated in the Virginia Manual.
9. Continuous vigilance should be maintained to ensure that all signs are provided.
10. For 2-lane crossroads, the use of double yellow lines without openings to divide the lanes seems to be necessary.
11. Crossovers could be channelized or made narrow and provided with nose markings and delineators to make them more conspicuous. Some of the crossovers with very wide widths could be modified by simple methods given in this paper.

ACKNOWLEDGMENTS

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A FEASIBILITY STUDY OF A REVERSIBLE-LANE FACILITY FOR A DENVER STREET CORRIDOR

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ABRIDGMENT

•TRAFFIC volumes and rush-hour congestion in and around many metropolitan areas are increasing yearly. Several years ago a national system of freeways was seen as the panacea for the congested streets and highways of the United States, but these wide-lane, limited-access facilities require space that is often not available in urban areas. Recently the emphasis has switched to grade-separated rapid transit systems for urban areas, but these systems are costly to develop and operate. Both of these systems have merit and could help in solving the chaotic transportation scene in urban areas if time and money were made available. Because of the high volumes of traffic on urban streets and the difficulty urban areas have in obtaining large amounts of capital funds, more economically attractive and easily implemented systems for increasing street capacities and traffic volumes need to be investigated. One such system that could help alleviate some of the congestion by increasing traffic volumes is a reversible-lane facility.

A street corridor southeast of the Denver central business district was selected as a study area. This area demonstrated directional flow characteristics on its boundary one-way arterial streets, Sixth and Eighth Avenues. Seventh Avenue, a little-used two-way residential street between the boundary one-way arterials, was believed to be capable of relieving some of the rush-hour congestion within the study corridor by the application of traffic engineering techniques.

The objective of this study is to determine the feasibility of the installation of a reversible-lane facility to reduce the congestion on a pair of one-way streets during rush hours. In this study it is hypothesized that, based on rush-period operation within the study corridor, a reversible-lane facility will increase capacity and reduce travel time with a minimum of disruption to the neighborhood.

Throughout the 1½-mile length of the study corridor Seventh Avenue changes from a 48-ft-wide street with business activity to a 30-ft-wide residential street and then to a 6-lane divided parkway through a residential area. By eliminating curb parking along the 30-ft-wide portion of Seventh Avenue, adequate width for a reversible center lane can be maintained. Transitions at both ends of the study corridor and for each of the changes in the street characteristics can also be provided.

The implementation of a reversible center lane and the resulting increase in capacity would be of little value if signal progression for the favored direction of flow could not be developed. Using a modification of the summation of offsets, new morning and afternoon rush-hour offsets were calculated that provide the favored directions with the same progression as the boundary arterials.

The signing, signaling, and pavement marking of a reversible center lane facility are perhaps the most important aspects for its safe and efficient operation. Without clear and legible lane-control devices a reversible-lane facility can be hazardous to drivers. Many different types of lane controls have been used in the past on reversible-lane systems. However, recently the Manual on Uniform Traffic Control Devices has defined standard lane controls to be used when implementing a reversible lane.

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Before any system can be evaluated, some estimate of the number of people who will use the system must be made. There are several methods for determining traffic diverted to a new or improved traffic facility. A modification of the capacity restraint model, which determines the least travel time between two nodes, was developed for use in this study.

Travel times throughout the study corridor were established using the relationship between volumes, capacity, and speed. The relationships shown in Figure 1 are adaptations of Figure 10.3 of the Highway Capacity Manual. The figures illustrate that, as volume increases, average speed of the vehicles decreases. The v/c ratio is the actual volume divided by the capacity of the facility.

The two curves in Figure 1 are of the same family of curves but represent different types of streets. Curb I was calibrated for the 2 one-way boundary arterials. Curve II was developed for conditions on Seventh Avenue after the installation of a reversible center lane.

The traffic volumes were derived for Sixth and Eighth Avenues by projecting traffic counts obtained from the Colorado State Highway Department. There were no records of traffic volumes on Seventh Avenue but observations showed these volumes to be negligible.

If the capacity and volume of these streets are known, the travel time over these links can be calculated. By varying the volumes carried by the one-way arterials and the reversible center lane facility, optimum travel times throughout the study corridor can be determined. The speeds that can be maintained within the study corridor for various volumes were determined and plotted in Figure 2. This plot of speed versus volume can be used to determine the optimum speed a given total volume should maintain and the resulting volume on each street. Curve I represents the cumulative volume, curve II the volume on Sixth or Eighth Avenues, and curve III the volume on Seventh Avenue in the direction of favored flow.

Observations made during the peak period of the peak hour indicate that traffic volumes on Sixth Avenue are at or near its capacity of 2,200 vehicles per hour. Seventh Avenue runs approximately 10 percent of this volume during the same peak period. The total westbound volume of 2,420 vehicles per hour travels through the study corridor at an average speed of 13 mph under present conditions.

Assuming that, if a reversible center lane were installed on Seventh Avenue, drivers would choose the route providing the shortest travel time, the volumes and speeds of traffic using both Seventh Avenue and Sixth or Eighth Avenue can be determined by Figure 2. This figure indicates that the same volume of 2,420 vehicles per hour could travel through the study corridor at an average speed of 28 mph, with 1,710 vehicles using Sixth Avenue and 710 vehicles on Seventh Avenue.

To determine if the installation of a reversible center lane on Seventh Avenue is economically feasible, the total transportation costs with and without such a facility need to be determined. At present the transportation costs consist of vehicle and travel time costs. With the installation of a reversible center lane on Seventh Avenue the additional costs of installation and maintenance must be included.

Several factors need to be known in order to determine transportation costs within the study corridor. These factors include the hourly traffic volumes during rush hours, the average speed of these volumes, the cost of vehicle operation, the cost of travel time, and the vehicle occupancy rate.

After determining these factors, transportation costs were computed within the study corridor for the years 1973 through 1975. For the purpose of comparison, the costs were derived for the weekday rush periods because a reversible center lane would operate only during these times.

All transportation costs were computed on a yearly cost basis. The yearly transportation costs with and without the installation of a reversible center lane on Seventh Avenue and the yearly savings and present worth of that savings were calculated for 3 years. Over a 3-year period a savings of $697,400, in 1973 dollars at 6 percent interest, in transportation costs will be recognized by the installation of a reversible center lane on Seventh Avenue. Each year the savings in total transportation cost is greater than that of the preceding year. Since the expected life of a reversible center

Figure 1.

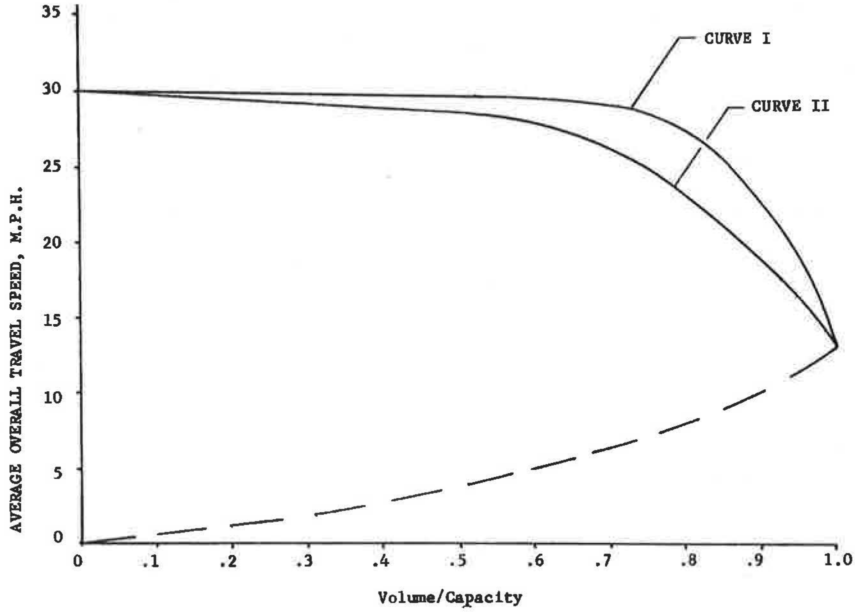
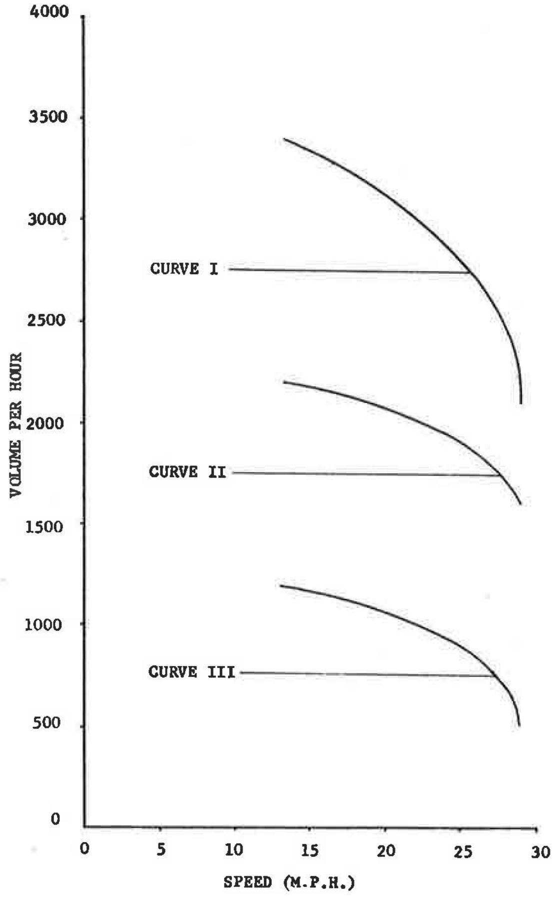


Figure 2.



lane facility should be at least 15 years, the actual transportation costs savings over the life of the project should far exceed the average present worth yearly gain of $233,000.

Before completing any study of a proposed traffic facility improvement, there are environmental aspects that should be considered. When crowded street conditions force traffic to start and stop often during rush hours, automobile emissions are increased. By increasing traffic progression, reducing congestion, and shortening the length of the rush period, air pollution and noise pollution can be reduced. Another environmental aspect, the aesthetics, may also be disrupted by the use of overhead lane signals and other devices used to implement a reversible lane system, but, when compared to widening projects or other alternatives for increasing street capacities, these devices seem less disruptive.

The results of this study show that a small initial investment for the installation of a reversible center lane facility could have a significant effect in reducing transportation costs over a short period of time within the study corridor.

Therefore, after an engineering analysis, the recommendation of this study is that a reversible center lane be installed on Seventh Avenue. This economically attractive and easily implemented system provides a simple solution for decreasing traffic congestion on the overburdened arterial streets.

As in any study of this kind, the results and recommendations are only a tool to be used by the decision-makers. Other aspects, such as citizen participation and reaction, would be vital inputs into the decision process before implementation could be considered.

Although this study concerns the feasibility of installing a reversible-lane facility for a Denver street corridor, the method of analysis is applicable in other areas of other cities. The probability of these same conditions existing elsewhere is very small, but the existence of overburdened arterial streets is a common problem for many urbanized areas. A properly engineered reversible-lane facility could be a boon in relieving some of the ever-increasing rush-hour congestion.

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TECHNOLOGICAL ASPECTS OF PUBLIC RESPONSIBILITY FOR GRADE CROSSING PROTECTION

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Recent interest in improvement of safety at railroad-highway grade crossings has been accompanied by a growing involvement of government at all levels. Public responsibility typically has been confined to providing funding, developing information, planning, and regulating; the design, installation, and maintenance of automatic protection has been exclusively a railroad activity. This paper examines the technical limitations that constrain public authorities from taking total responsibility for crossing protection devices, which are the only highway traffic control devices that are not the responsibility of highway officials. Research directed toward removal of those limitations is described. A review of the legal history and current role of governmental units precedes a description of conventional technology in terms of impact on a wider public role. Means of train detection and motorist warnings are discussed; the conclusion drawn is that the principal technological impediment to non-railroad responsibility for crossing protection is the present dependence on track circuit techniques for determination of train presence. Recent research directed at removing this constraint is presented. Analysis of system requirements and available technology has identified a discrete train detector-microwave communication link concept, and the results of field testing indicate a number of attractive features and general feasibility.

•IN recent years there has been a significant increase in the attention directed toward improvement of safety at railroad-highway grade crossings. Examples of this awakening—particularly at all levels of government—include the Highway and Railroad Safety Acts of 1970 and the resulting two-part FRA-FHWA Report to Congress (1, 2); aggressive and comprehensive information-gathering and protection implementation programs in a number of states; formation of Department of Transportation and Highway Research Board committees; and convocation of four national conferences. Federal and state funding legislation, development of improved governmental structures, and an improved information base for policy formulation and implementation have been accompanied by steadily increasing assumption of both capital and maintenance costs by public bodies. In 1972 a new FHWA policy eliminated completely the requirement for any railroad contribution to the cost of installation of automatic protection on federal-aid projects. At least 17 states now have special crossing improvement funds, and 11 share to some degree in maintenance expense—100 percent under certain circumstances in one state.

This growth of public involvement might not seem noteworthy to the casual observer. The basic function of crossing protection is, after all, to alert the motorist to a possible hazard—a responsibility normally assumed by governmental bodies for virtually all other potential dangers on highways. However, historical, technical, and legal considerations have traditionally lodged the primary burden of protection on the railroads.

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The movement away from that arrangement has arisen from a number of factors, which include the great increase in highway traffic, the diminished role of railroads as the predominant transportation mode, the impediment to efficient implementation of protection programs caused by diffusion of functions among numerous public and private bodies, and the ever-greater degree to which public funds are involved.

It is the objective of this paper to explore the subject of direct involvement by public agencies in the actual installation and maintenance of automatic crossing protection, including the possibility of complete independence from the railroads. A description of the general background and context of grade crossing protection matters is followed by a review of relevant present technology and both the practical and inherent limitations thereby imposed. Attention is then given to the nature and benefits of activities that could be undertaken by governmental bodies either within conventional techniques or through application of recent technical developments. The latter discussion is based primarily on research carried out over the last 3 years concerning alternatives to track-circuits for actuation of motorist warnings.

BACKGROUND

The "grade crossing problem" began almost with the first railroad and became a significant concern as railroads expanded in the late 19th century. The legal history of the subject has been examined by FRA (1) and is only briefly summarized here. In the 1890s several court decisions held that assignment of the crossing protection responsibility to the railroads was both within the inherent police powers of the states (to ensure public safety) and justified as an obligation naturally associated with the railroad's acceptance of a franchise. Although this basic view prevailed until the 1930s, the dramatic increase of motor vehicle traffic and highway improvements soon raised the problem to a serious level, causing reconsideration. The early ventures into federal financing of highway construction permitted, in 1916, the use of such funds for reduction of hazards at railroad-highway crossings; usually a substantial railroad contribution was required. However, the primary responsibility for crossing protection quite clearly remained with the railroads. During the depression, financial difficulties for the railroads were accompanied by major federal-aid highway construction programs, creating many additional crossings on improved highways. This was an important change from the 19th century, when new tracks were generally cutting across existing highways.

At this time both governmental policy decisions and several landmark court cases established a marked turn toward increased public responsibility. At the federal level, the basic guideline to emerge from the 1930s (widely, but not universally, accepted) was that costs should be assessed in proportion to benefits received. One indication of the result is the observation that during the period from 1934 to 1972, for those crossing protection projects involving the use of federal funds, such monies comprised 83 percent of the total $3.5 billion expended. The next major turning point was the extensive study undertaken by the ICC in 1961 and concluded in 1964. An important finding was that "The cost of installing and maintaining such separations and protective devices is a public responsibility and should be financed with public funds the same as highway traffic devices" (3).

The acceptance of major federal responsibility was underscored in 1970 by passage of the Federal Railroad Safety Act, the Highway Safety Act, and the Federal-Aid Highway Act, all of which address grade crossing safety in a substantive way, and later by the Highway Safety Act of 1973, which provides for specific funding for automatic protection and funding (for the first time) for installations off the federal-aid system. Similarly, a number of states have undertaken coordinated and comprehensive programs in problem definition, policy formulation, and installation of protection and have established special state funds for both capital and maintenance costs.

Grade separations, being extremely expensive, have typically accounted for the major part of resources expended (94 percent in the period 1967-1970) and are generally motivated and justified more on grounds of motorist convenience and reduced delay than on safety, since nearly as great a level of protection is possible with automatic devices at a fraction of the price. Indeed, a conclusion of the report to Congress (1, 2) is that the most effective and beneficial expenditure of available resources in terms of safety is a program of installation of new protection and improvement of that already existing at approximately 30,000 public crossings. Thus, it is this topic—implementation of active protection—that has generally received major attention and that forms the focus of this paper. Both conventional and innovative technology are considered, with special attention given to those aspects of particular relevance to public responsibility.

NATURE AND IMPLICATIONS OF CONVENTIONAL TECHNOLOGY

Discussion of grade crossing technology is facilitated by delineation of two quite separate functions: (a) detection of actual or imminent train presence at the crossing and (b) presentation of appropriate warnings to the motorist. It is sometimes useful to consider as separate the interface circuitry that connects the basic train detection equipment to the warnings. However, that function is often physically a part of the system that determines train presence and is so treated here. The basic principles of conventional techniques are easily stated, since practices are well standardized. During the fluctuations in funding and other responsibilities described earlier, one factor has remained constant: The railroads have always been responsible for design, installation, and maintenance of crossing protection. Thus, the hardware and concepts associated with automatic protection arise directly from railroad signal technology and practices and have been controlled exclusively through establishment of industry (AAR) standards, specifications, and requisites.

Train Detection

A brief review of the history and state of the art of such systems has been given elsewhere (4) and will not be repeated here. However, certain critical aspects deserve emphasis.

The most fundamental and universal characteristic of active protection is use of the track circuit for train detection. Invented for general railroad signal purposes in 1872, it forms the basis of block signal technology and was first applied to grade crossings in 1914. The basic concept is shown in Figure 1. The principle of operation is quite elegant. The battery at one end of a section of track—electrically isolated at both ends—is connected to a relay at the other end, using the rails as electrical conductors; the normally closed relay is held in an open position. A train between the battery and the relay short-circuits the relay, which, upon losing current, closes, thereby activating any desired warning, such as a bell, light, or gate. Several features are particularly noteworthy. Any open circuit (break) in the rails or connections, or any short circuit across the rails, or failure of the power source (battery) causes the gravity-operated relay to close, actuating the warnings. Thus, with respect to all primary failure modes, the system is fail-safe, in the sense that malfunction causes the most restrictive signal aspect—a fundamental criterion for all railroad signaling. Actual achievement of a protective system approximating truly fail-safe operation requires careful attention to many details, particularly in the more complex designs and installations now used. Many years of evolutionary improvement have been required to provide the high level of performance now available. Such a system, unless equipped with overriding devices, provides continuous detection, in that a train is detected constantly while in the block.

The most basic crossing protection system, then, entails a track circuit on either side of the crossing ("approach circuit"), with a third covering the region where the tracks actually cross the highway ("island circuit"). The length of the approach circuits must be sufficient to provide 20 to 30 seconds of warning for the fastest train speeds allowed—approximately ½ mile (0.8 km) for a 60-mph (97-kmph) train speed limit. Modern modified installations utilizing audio frequency signals rather than direct current, with solid-state logic, have proved advantageous in many locations, but a number of constraints to this approach remain. The track segments involved must have electrical integrity throughout their length and isolation at each end. A substantial quantity of power is required at the "battery" end (whether DC, AC, or audio frequency)—at least several watts—and this must be provided via special cables or existing track-side power lines. In addition, all active elements must have emergency power—batteries—available in the event of power or fuse failures. The challenging nature of the railroad operating environment—weather, temperature extremes, vandalism—should need little elaboration, but it is appropriate to note the less obvious difficulties, such as vulnerability to lightning and other power surges and variation of the electrical impedance of the ballast between the rails.

In recent years, a new class of devices has been developed that also use the rails as conductors and detect the trains from the shunting effect of the train wheels and axles. However, there are significant differences and new functional capabilities, compared to the basic track circuit. The concept is shown in Figure 2 and is dependent on measurement at the crossing of the electrical impedance between the rails. Although the rails have a very low resistance, it is not zero, so that as a short circuit (a train, for example) moves toward the crossing the measured impedance decreases. Thus, it is possible to determine not only that the block is occupied but also whether the vehicle is moving and the direction of motion, toward or away from the crossing. In the simpler applications of this concept, such devices serve as motion detectors, eliminating unnecessary actuations when trains stop near a crossing or move away from it after stopping and reversing. The more sophisticated forms can actually measure range and closing rate with sufficient accuracy to activate warnings a fixed time interval prior to train arrival, regardless of train speed. This constant warning time feature appears to be highly desirable. In part, it reduces unnecessary motorist delay, but, more importantly, it also provides a far more precise, and thus more credible, warning, and motor vehicle operators appear more likely to obey signals that experience shows to be truthful. Such devices require power only at the crossing, with a passive termination at the end of the block, but the more complex version for constant warning time also demands substantial power—tens of watts.

In summary, the track circuit approach is well proved, effective, and reliable, but it is also relatively labor-intensive in both installation and maintenance and is therefore not inexpensive. Although largely fail-safe, system malfunction is generally not easily distinguished from train presence, which leads to an undesirably high false-alarm rate, with unfortunate impact on system credibility and motorist response. However, the most important weakness in terms of this discussion is the inherent inseparability of track circuits from railroad involvement and responsibility for operation. It is clear that this technique—as effective as it has proved for the railroads—is totally inappropriate to implementation by any non-railroad body. Thus, total public responsibility for crossing protection can be achieved (if desired) only through alternative technology, for which there has previously been no demand. This topic will be explored at a later point.

Motorist Warnings

Given a reliable and accurate means of train detection, the heart of the protective system is the means by which the train presence is displayed to the motorist. If it is to be effective, virtually all drivers must see the warnings, understand their meanings, and be motivated to act accordingly. The fact that nearly 40 percent of crossing fatalities occur at railroad-highway intersections that have some form of active protection suggests that this sequence fails all too often. Unfortunately, the statistical data are inadequate at present to identify specific weaknesses. "Active protection" as used here includes a wide variety of hardware and crossings, and it may well be that the best of present-day systems, properly installed and maintained, can demonstrate a far better record than the average for all active protection. Indeed, figures reported by the California PUC (5) suggest very high effectiveness for well-engineered gate installations, which generally include constant-warning-time train detection. It is noteworthy that California, in strongly emphasizing gates, reduced crossing accidents by 49 percent from 1965 to 1972, while the remainder of the nation showed only a 7 percent decline (6).

Figure 1. Basic track circuit.

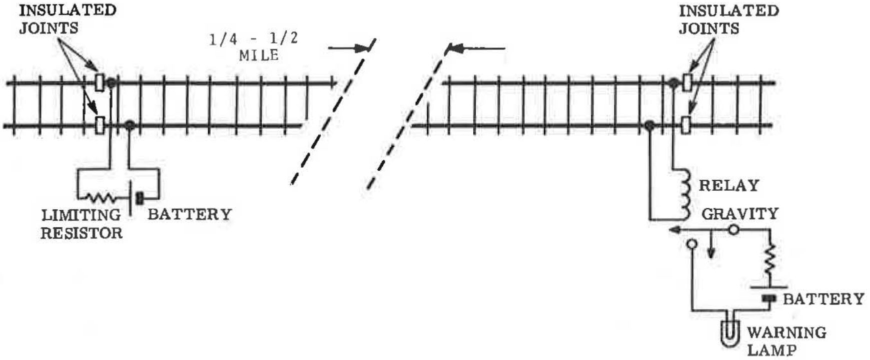

Figure 2. Basic impedance-measurement train-motion detector.

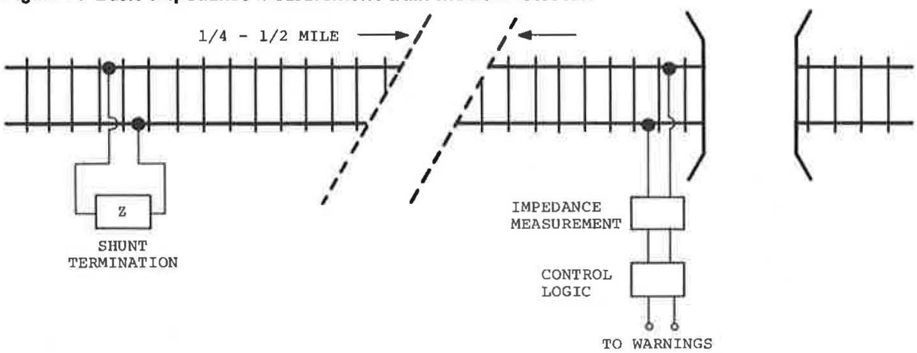

Figure 3. Basic telemetry system.

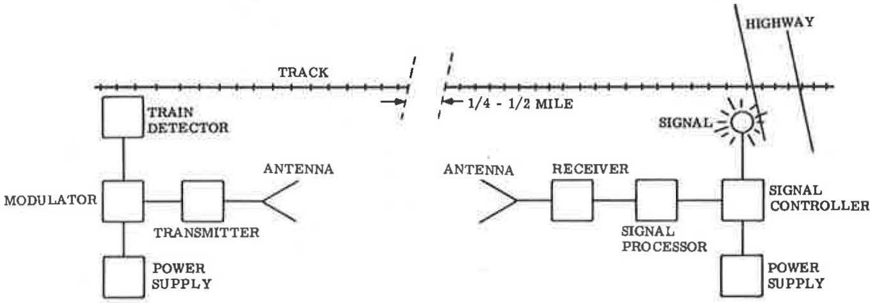

Figure 4. Telemetry system during field test (transmitter location, solar panel in use).

Regardless of statistics, an informed observer may question whether present active warnings represent the best that can be achieved. Although many variations exist, the two basic devices used throughout the United States are flashing lights or flashing lights plus automatic gates. The lights, which are used alone at 80 percent of crossings with automatic protection, have been developed by the railroad signal community rather than highway signing engineers and scientists, and this has led to certain characteristics. The flash rate (35 to 40 cycles per minute) is modeled after the rate at which a man customarily swings a lantern. The shade of red commonly used is substantially deeper than general highway use demands, determined in part by the basic railroad concern that an engineer might mistake a red block signal for amber. This was unfortunate, since light intensity was reduced more than necessary by the dark lens. Some recent installations of flashing lights have used a lighter red such as the ITE shade, although further improvement is possible. Intensity is a serious concern with grade crossing flashing lights, since the requirement for a 1- to 3-day back-up battery power supply dictates minimal power consumption. The bulbs have generally been 11 or 18 watts; 25-watt units are now coming into use. Sufficient brightness is obtained through utilization of narrow-beam focusing lenses and high-quality reflectors. This requires precise alignment, achieved only through frequent maintenance and very sturdy (and expensive) mounting structures, which are quite impressive in size when lights are mounted over the highway on cantilevers.

Criticism of these devices is not the point of this discussion. However, it is not unreasonable to examine alternatives with the goal of beneficial impact on both cost and effectiveness. There are no serious technical barriers to such experimentation, either by railroads or public bodies. (Many years of dealing with the problems of interconnecting crossing protection signals and nearby highway traffic lights have established procedures by which railroad-owned train detection systems can be used to operate non-railroad devices with no danger of creating malfunctions for which the railroad is not responsible.) The principal difficulties in this area are legal and institutional. Railroad companies are bound both by strong concerns for liability—"experimental" devices may be ill-received by a jury—and by standards established by trade organizations and state regulatory bodies. Public authorities appear to have been loath to attempt to complicate further the task of achieving installation at a particular crossing by seeking some new, non-standard warning. In addition, these constraints have served to limit interest by others in development of improved, innovative devices.

This situation is unfortunate, for it not only prevents innovation in general but also has tended to exclude those most knowledgeable in the subject of motorist warnings from involvement in this key element in crossing protection. (For example, the Manual on Uniform Traffic Control Devices merely refers to Association of American Railroads standards.) However, as noted, the situation is not without hope; alternatives can be tried if the railroad and the state are willing. Experimental systems can be in addition to standard equipment if suitable, although regulatory waivers and liability insurance may be required. The point to be emphasized is that it is physically possible for public authorities (most probably highway departments) to install and maintain innovative (or conventional) warnings, and in many cases this may be feasible—if not easy—within the institutional constraints as well. A current example of such an effort is the installation of strobe lights on gates on a high-speed rural highway carried out by the State of Indiana and the Norfolk and Western Railroad.

Advance Warnings

In the case of advance warnings—those installed before the crossing merely to alert the motorist to the impending potential hazard—much greater freedom exists, although it has been little utilized. In addition to a less rigid relationship to liability and regulatory aspects, such warnings are in most cases already the responsibility of highway authorities. This aspect of crossing protection has generally received very limited attention, although recent state and FHWA research projects auger well for improvement. The present standard warning has a limited ability to attract attention, particularly if poorly maintained, and provides only the barest information concerning the imminent hazard. The motorist is not told whether the protection provided is active or passive, although his surveillance activities should be dependent on this. The number of tracks, angle of the crossing, possibility of obscured sight lines, and nature of the rail traffic are all ignored. Such information could, of course, be readily provided.

The subject of active advance warnings is particularly interesting. Given the major investment associated with automatic protection, it is clearly desirable to maximize the effectiveness obtained. As mentioned in connection with crossing-located motorist warnings, there is no major technical problem involved in obtaining train presence information from railroad-operated train detection apparatus and using it to activate advance warning devices. In special situations, particularly those characterized by blind approaches, both states and railroads have used such devices. However, more widespread application could carry significant benefits, and implementation poses no major problems other than the ever-present question of availability of funds. It should be noted also that new active warning devices can be tested first as advance warnings and then be considered for installation at the crossing if found to be effective.

In summary, current technology and practices are such that only standard passive advance warnings are the responsibility of public officials. The basic concept underlying conventional train detection—the track circuit—virtually excludes non-railroad operation of that element of the system. However, the possibility of more extensive public concern with active, crossing-located warning devices appears to be limited more by tradition and legal and institutional factors than by technology and offers the opportunity for greater experimentation than has been the case to date. Improvement of advance warnings, particularly through the use of active devices, has received attention in some states but appears to remain a promising area for substantial public involvement, in terms of both ease of entry and potential benefits.

RELEVANCE OF TECHNOLOGICAL INNOVATION TO PUBLIC INVOLVEMENT

The foregoing discussion has shown that the primary technical limitation on full public ownership and operation of grade crossing protection is associated with the task of timely actuation of motorist warnings. Although trains do indeed make their presence known in a wide variety of ways, the demands made on crossing protection systems are severe and not easily met. First and foremost, all trains must be detected adequately in advance of arrival—typically 20 to 30 seconds. All system failures must result either in activation of warnings or unmistakable indication of the malfunction. The operating environment is severe, and both practicality and safety demand extremely durable equipment with a long service life and limited maintenance needs. Costs must not be extreme—certainly no greater than for conventional equipment.

Low power consumption is desirable in general to reduce the required investment and maintenance associated with an emergency supply and is particularly important away from the crossing, where provision for line power can add significant expense and vulnerability to lightning damage. Power drain of less than $\frac{1}{2}$ watt is desirable. Finally, for total public responsibility, there must be a high independence of railroad property and systems. Of the many alternative techniques that might be considered, most can quickly be rejected through application of the above criteria. Several, which have been found to merit further consideration, are discussed in the following sections.

Train Presence Detectors

Most potential alternatives to track circuits explictly separate the train detection and communication functions. This approach involves specifically checking trains in and out of critical regions rather than noting presence continually, as do track circuits. One then requires specific detectors of train presence at a particular point. Rail vehicle presence detectors are used in a variety of applications, generally not vital (safety-related), and several types exist. Other concepts, some drawn from related fields, could be developed for the grade crossing case. The "perfect" sensor, which probably does not exist, would be characterized by very low (or zero) power consumption; fail-safe operation; no electrical or mechanical attachment to the rails; high resistance to weather and vandalism; indication of train direction and velocity; sensing of stationary trains; and low cost. A brief review of the state of the art follows.

Wheel Detectors—The most common type of detector in general railroad use is the wheel detector, which bolts to a rail and detects passing wheel flanges either magnetically or inductively. Both active and passive methodologies are available; active devices consume significant power but offer better possibilities for fail-safe operation. Such devices are subject to damage by dragging equipment, plows, and vandals, and prices range from approximately $200 to $800. Physical connection to the rail implies railroad involvement, but there is no inherent link to the signal system, nor dependence on electrical characteristics of the rails. Speed and direction measurement is possible at significant increases in cost and power.

Inductive Loops—A commonly used highway vehicle detector is the inductive loop, which is also produced in a form suitable for railroad use. Relatively high power consumption is a weakness. They must be installed in close proximity to the tracks, over a relatively large area, so that cost, durability, and vandal resistance can suffer. Velocity and direction information are not easily obtained.

Magnetometers—Since all rail vehicles are composed partly of large masses of iron (for example, the wheels), magnetic detection is natural to consider. A commercial traffic detection magnetometer was tested, buried 1 ft (30 cm) below the track level. Results were highly satisfactory, although power consumption was higher than desired. Multiple units are required for velocity and direction discrimination, doubling cost and power consumption.

Beam Interruption—A common means of detection of moving objects is interruption of a beam, typically of visible or infrared light. Difficulties associated with fog, dirt, and malicious activation appear solvable for this application with careful design, but power consumption, cost, and multiple-track situations all represent complicating problems. Speed and direction can be determined from dual-beam systems with moderately increased complexity.

Pressure—A natural possibility is detection of rail flexing or other pressure related effects. However, no obvious realizations or available devices that meet the criteria have been identified.

Mechanical—A rail-mounted treadle switch, activated by the wheel flange and used widely in Europe for other applications, was tested. However, unsafe failure modes, vulnerability to accidental and malicious damage, and maintenance needs make it an unpromising approach.

Sonar—Ultrasonic sonar, mounted above roadways, has been used successfully for vehicle detection. However, cost, vulnerability to weather (ice in particular), and high power consumption are substantial drawbacks.

Radar—Short-range radar, using compact antennas and solid-state oscillators, appears promising, although achievement of fail-safe operation is challenging. Complete independence from rail operation is possible.

In summary, there are a number of potentially feasible means of presence detection, each with certain strengths and weaknesses. Although no ideal detector is available, it appears that a satisfactory compromise is possible in most cases. The choice will depend on the relative importance of particular constraints—speed information, power consumption, railroad independence, etc.

Communication of Train Presence Information to the Crossing

The communication task may be simply defined. The basic requirement is transmission of information, at a very low data rate, over a distance typically less than 3,000 ft (914 m). The constraints described earlier must be met. One can easily imagine a number of possible approaches, but most have serious limitations. For example, the cost of underground or pole-mounted cable, including installation and maintenance, is quite expensive. Of the electromagnetic approaches, optical devices are too vulnerable to the environment for the range considered—dust, snow, mud, fog, ice, and vegetation could all drastically interfere with proper operation.

On the other hand, radio techniques are quite suitable. Radio communications can be carried out using readily available apparatus in the frequency range of fractions to tens of thousands of megahertz. Efficiency, reduction of electromagnetic interference problems, and low vulnerability to extraneous signals strongly suggest the desirability of a focused, line-of-sight system in which signals are either absorbed by obstacles or pass through the ionosphere with no reflection. High frequencies are also desirable in that wider, less crowded bands are available and antenna size—determined by wavelength—can be smaller and thus more convenient. An important weakness of low frequencies (below 1 GHz) is the lack of durable, small, highly directional antennas; use of a narrow beam can increase system efficiency by a factor of 10^3 to 10^6 with both transmission and reception are considered. Economical microwave sources and compact, highly directional antennas are best obtained in the frequency range of 10 to 20 GHz. Significantly higher frequencies (above 30 GHz) would increase cost substantially, as both oscillators and other components would require closer manufacturing tolerances. In addition, above 30 GHz, attenuation from heavy rainfall can have a significant effect on propagation distances. On the other hand, at 10 GHz no severe problems occur for rainfall of less than 5 to 10 in. (12 to 25 cm) per hour, a rate at which motor vehicle traffic would presumably be at a standstill.

Considerations of this type lead to the conclusion that the most practical means of realizing the communication function is in the form of a simple microwave telemetry link, in which the short range and low information rate required make possible a simple, highly reliable, low-cost system. A basic communication link has been designed according to these guidelines, constructed, and tested in order to explore the feasibility of such an approach. A block diagram of the system is shown in Figure 3. Technical details of the effort are available elsewhere (7, 8) and are merely summarized here. A solid-state microwave transmitter, operating at 10.5 GHz, is placed at the down-track train detection point, with a receiver at the crossing. The normal (train absent) condition is with the transmitter on, with pulse modulation of low enough duty cycle to provide minimal power consumption. At the receiver, this signal is detected and rectified, giving an output voltage as long as a signal is received. In the absence of such a signal, for whatever reason, there will be no output, and malfunction or motorist warnings are activated to provide fail-safe operation. It is highly desirable that there be a detectable difference between system failure and train presence, so the latter case is indicated by a change in the modulation waveform rather than total absence of signal. The receiver also has an input from a train detector at the crossing, so that it is reset to the train-absent state after a train moves across the crossing. As is the case for track circuits, appropriate logic is necessary to account properly for train presence, direction, etc., particularly in multiple-track situations.

Pulses are transmitted at a rate of 2 to 3 per second, so the system responds to train presence in approximately 1 second. The power consumption of the transmitter is approximately 100 mW, or 1 kW-h per year, and this can be reduced still further. Charging from solar panels 1 ft^2 (930 cm^2) in area is entirely feasible and not excessively expensive. Use of sealed batteries can reduce periodic maintenance needs to annual servicing. An installed prototype system, utilizing solar panels, is shown in Figure 4. Six such installations, in several variations, have undergone extended field testing under realistic conditions of operation over periods of 6 to ten months. The tests were carried out at grade crossings with conventional active protection in place. Both the existing track circuits and the experimental units activated strip-chart event recorders, providing a clear indication of the reliability and accuracy of the new systems. A variety of train detectors was used, with primary reliance on magnetic flange detectors and magnetometers. The sites were located on Boston and Maine Railroad mainline track within 25 miles (40 km) of Boston.

Results of the field tests were highly encouraging and clearly demonstrate basic feasibility. Difficulties that occurred are typical of first-stage field testing of prototypes and generally involved peripheral hardware. The basic system concept has proved completely satisfactory. The transmitter and receiver have sufficient margin that performance can be degraded very markedly—over 20 db—before malfunction occurs, and this will be in a fail-safe mode, with a malfunction indication generated.

The cost associated with this approach is clearly a very important factor in ultimate viability. The exploratory nature of this work prevents quotation of exact prices. However, the basic circuitry is of approximately the same complexity as found for track circuit systems, and it appears that installation and maintenance requirements can be significantly reduced. Expenses associated with provision of power and surge protection should be lower, and in multiple-track situations one telemetry system, with additional sensors and logic, can replace several track circuits. Thus it appears that cost reductions of 20 to 30 percent are realizable, although the principal benefit of this approach is felt to lie with the potential it offers for public operation of crossing protection.

CONCLUSIONS

In terms of both technical and legal considerations, public authority for crossing protection is most readily assumed in operation of active advance warnings, acting to supplement existing crossing-located railroad equipment. Simple means of actuation are possible that allow complete isolation from railroad circuits. This area has been addressed in several states and locations but appears to warrant greater attention. There is less risk in experimentation with advance warnings, so that trials of new types of signals are not so severely constrained. Devices that show high effectiveness then become candidates for installation at crossings.

The technical constraints on public responsibility for active warnings—but not train detection—at the crossing are no greater, but legal questions present an obstacle. In the event of any failure, there is a possibility of extended controversy over whether it occurred in the detection system or the warning devices. Railroads might naturally hesitate to enter into such an arrangement for fear of becoming embroiled in the failures of another party. Relevant standards of regulatory bodies might also require waiver. Matters could be facilitated through purchase of liability insurance; such a strategy is uncommon but not unknown.

Full public responsibility for the total protection system, including train detection, poses a severe technical problem at present, since conventional technology in this country is universally based on track circuit techniques. Research at the Transportation Systems Center has demonstrated the feasibility of a non-track-circuit concept, although significant product development, field testing, and refinement are necessary before such a system would be acceptable. Also, there is an additional practical constraint on implementation. Such systems appear to offer significant cost savings, but that estimate is based on production volume comparable to that for conventional hardware. However, railroads are naturally reluctant to introduce a system totally unrelated to present techniques, since this complicates inventory and labor matters. Thus, reasonable production—and attractive costs—are likely only if a number of states and localities actively choose to follow such a course.

A decision of that nature will not be easy. The advantages of simplified implementation of crossing protection—lower cost and more direct control—are offset by the need to establish the appropriate facilities and labor force and (perhaps more restricting) to face the potential lawsuits in the event of accidents. Liability is not the subject of this paper and will not be addressed here, but it appears that the overall legal constraints and responsibilities involved cannot be completely spelled out in advance but rather will evolve as various precedents are applied to a succession of cases. There appear to be major benefits associated with a decision to accept this challenge—improved protection and enhanced capability to implement a comprehensive, coordinated program—and history shows a steadily increasing public involvement that may ultimately include total responsibility.

A first significant step could be taken if a state or other public authority assumed responsibility for installation, operation, and maintenance of some active motorist warning devices at the crossing, with the railroad continuing its traditional responsibility for the train detection track circuits, terminating them in a junction box in the vicinity of the crossing in which the state would make connections leading to the warning devices.

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PARKING PATTERNS AND PRICES IN THE CBD

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This paper shows how the flow of automobile traffic from residential areas is allocated among downtown parking facilities by a pattern of prices that acts to minimize the total driving and subsequent walking costs for the drivers as a group. These prices also provide the maximum revenue that can be collected by each parking facility when competing freely. A set of data from a central business district with more than 10,000 parking spaces demonstrates the validity of the analysis and shows that the parking patterns and prices can be determined inexpensively by computer. The model should be useful to traffic engineers and urban planners in their design of more efficient urban transportation systems.

•THE allocation of demand for parking space to the available supply customarily is regulated in large cities by a system of user fees. The automobile driver searching for a parking space selects a location that he feels minimizes some combination of driving time, cost of parking, and walking distance to his ultimate destination. Drivers who value their time most highly will tend to select locations close to their destination, while others will save money by parking in peripheral lots and walking longer distances. The parking lot manager, on the other hand, in attempting to maximize his revenue, sets his fees at the highest level that competition will permit without significant loss of patronage. The parking price therefore acts to ensure that virtually all spaces are used and that they are allocated to those parkers who value them most.

The foregoing principles represent the extension of a model by Brown and Lambe (1) to include the effects of driving distance; they belong to a growing body of literature on parking models (2-5). There are two advantages from this extension. The first is that the inclusion of driving distances provides a clearer view of the flow of traffic from the suburbs to the central business district (CBD) and back. The second is that the presence of this secondary cost factor improves the accuracy of the model in its prediction of parking prices.

The effect of driving distance on the choice of parking location can be illustrated by a simple example of two persons destined for the same office building, where one person lives to the east and the other to the west of the building. Clearly, if there only are parking spaces available at 1,000 ft from the building in each direction, the person from the westerly suburb should use the west one and the other person should use the east one. However, if the westerly parking facility was 3,000 ft away from the office building, the first person probably would prefer to drive an extra 4,000 ft to the eastern parking facility to save 2,000 ft of walking if both lots were free. If he valued his time and traveling expenses at \$0.05 per 1,000 ft for driving and \$0.20 per 1,000 ft for walking, he would theoretically prefer the eastern lot (to save \$0.20 per trip), all other things being equal.

The foregoing example can also demonstrate the effect of driving distance on the maximum price that each person is willing to pay for a parking space at his office building. The person from the east would be willing to pay \$0.15 to save 1,000 ft of walking by driving the extra 1,000 ft to his office, while the person from the western suburb (and parking in the eastern facility) would pay \$0.25 because of the additional

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saving in driving. On a daily basis, each would be willing to pay double the figures because of the savings on the return trip to his home. Clearly, if only one space were available at the office building, the westerly person theoretically would get it in a free market by being willing to pay a higher price. Furthermore, he should get the space if total driving and walking cost is to be minimized for this combination of drivers. The value of an additional space for the eastern driver, incidentally, would be twice \$0.15 per day. Consequently, the manager of a parking facility at the office building could charge twice \$0.25 per day if he had one space available, but only twice \$0.15 per day per space if he had two spaces available and could not charge the customers different amounts.

Finally, the example can illustrate the effect of parking duration on the value and choice of parking facility. If the person from the eastern suburb went home for lunch, a parking space at the office building would save him four walking trips per day, and consequently would be worth \$0.60 per day to him. Therefore, if only one space was available at the office building, he would be able to bid a higher price than the westerly person who stayed at his office all day. The value of an additional space (for the westerly person) would be \$0.50 per day. Additional spaces at either outlying facility obviously would yield no revenue because the facility already has ample capacity that is free.

These examples conform to the classical transportation problem (6), where the object is to allocate a set of demand quantities (the parkers) to another set of supply capacities (the parking spaces) in such a manner as to minimize the total transfer (driving and walking) cost. The advantage of this representation of the problem lies in a very efficient mathematical procedure that not only determines the allocation of parkers to minimize total driving plus walking costs in the city but also determines the optimal set of parking fees to achieve this end. Furthermore, an extremely fast computer program has been developed for finding these solutions (7).

The algebraic representation of the transportation problem determines the specific (non-negative) number of drivers X_{ijkp} who drive from a point (i) to a parking facility (j), then walk to a building (k), and repeat the trip a specific number of times (p) per day. The transportation cost C_{ijkp} for each of these drivers depends on their driving and walking distances per day. The solution obviously cannot assign more people to a parking facility (j) than its total spaces S_j. It also must satisfy the demand D_{ikp} of people traveling from point (i) to destination (k) with frequency (p). The optimal solution therefore is

$$\text{Minimize} \sum_{ijkp} X_{ijkp} C_{ijkp} \text{ by adjusting } X_{ijkp}$$

subject to

$$\sum_{ikp} X_{ijkp} \le S_j \text{ for all j}$$

$$\sum_{j} X_{ijkp} = D_{ikp} \text{ for all i, k, p}$$

$$X_{ijkp} \ge 0 \text{ for all i, j, k, p}$$

As an illustration, the last version of the previous example consists of two drivers, one from the west (i = 1) and the other from the east (i = 2). Both of them have the same destination (k = 1), but one makes one round trip per day (p = 1) and the other makes two round trips per day (p = 2). Therefore $D_{111} = D_{212} = 1$, and $D_{211} = D_{112} = 0$. Each person has a choice of three parking facilities, a large free one having, say, 100 spaces located 3,000 ft to the west (j = 1), a single space at the destination (j = 2), and another large free one located 1,000 ft to the east (j = 3). Therefore S_1 and S_3 present no capacity constraint, but $S_2 = 1$. If each person's home is 10,000 ft from the office, the transportation costs in dollars per day are $C_{1111} = 1.90$, $C_{1211} = 1.00$, $C_{1311} = 1.50$, $C_{2112} = 5.00$, $C_{2212} = 2.00$, $C_{2312} = 2.60$. The algebraic representation of the problem finds the combination of non-negative values for all of the X_{ijkp} to minimize

$$(1.90X_{1111} + 1.00X_{1211} + 1.50X_{1311} + 5.00X_{2112} + 2.00X_{2212} + 2.60X_{2312})$$

subject to

$$X_{1111} + X_{2112} \leq 100$$
$$X_{1211} + X_{2212} \leq 1$$
$$X_{1311} + X_{2312} \leq 100$$
$$X_{1111} + X_{1211} + X_{1311} = 1$$
$$X_{2112} + X_{2212} + X_{2312} = 1$$

By inspection, the solution is $X_{1311} = 1$, $X_{2212} = 1$, and $X_{1111} = X_{1211} = X_{2112} = X_{2312} = 0$. In practical terms, the first variable states that a person drives from the western suburbs ($i = 1$), parks at the eastern facility ($j = 3$), walks to his office ($k = 1$) one round trip per day ($p = 1$). The second variable states that another person drives from the eastern suburb ($i = 2$) directly to a parking lot ($j = 2$) at his office ($k = 1$) two round trips per day ($p = 2$). The remaining variables confirm that no one else drives and parks elsewhere. The correspondence between the theoretical and the practical observations (for the idealized model) indicates the usefulness of the theory for predicting the basic flow of traffic in a large city from a multitude of possible trips.

The maximum daily rental R_j that the manager of a parking facility (j) can charge without losing customers is given by the dual formulation of the transportation problem. This version states that the maximum amount P_{ikp} per day that each person making (p) trips from origin (i) to destination (k) is willing to pay for any space is the smallest of all available combinations of driving, walking, and parking fee. The algebraic representation finds P_{ikp} and R_j to

$$\text{Maximize} \left[\sum_{ikp} P_{ikp} D_{ikp} - \sum_{j} R_j S_j \right]$$

subject to

$$P_{ikp} \leq R_j + C_{ijkp} \text{ for all } i,\ j,\ k,\ p$$
$$P_{ikp} \geq 0 \text{ for all } i,\ k,\ p$$
$$R_j \geq 0 \text{ for all } j$$

The dual version of the previous example selects non-negative values for P_{111}, P_{212}, R_1, R_2, and R_3 to maximize

$$(P_{111} + P_{212} - 100R_1 - R_2 - 100R_3)$$

subject to

$$P_{111} \leq R_1 + 1.90$$
$$P_{111} \leq R_2 + 1.00$$
$$P_{111} \leq R_3 + 1.50$$
$$P_{212} \leq R_1 + 5.00$$
$$P_{212} \leq R_2 + 2.00$$
$$P_{212} \leq R_3 + 2.60$$

By inspection, the optimal solution is $P_{111} = 1.50$, $P_{212} = 2.60$, $R_2 = 0.50$ and $R_1 = R_3 = 0$. The practical implication of the first number is that an additional person traveling from the western suburb to the same building in the CBD and back per day would have a total daily outlay of \$1.50 for the cost of time and travel expenses. These consist of \$1.10 for driving 11,000 ft each way and \$0.40 for walking 1,000 ft each way from the eastern parking facility. The second number states that an additional person from the eastern suburb would pay \$2.60 per day for his two round trips while using the eastern parking facility. The remaining three numbers repeat the earlier conclusion that an additional parking space at the destination building would rent for \$0.50 per day, while additional spaces at either outlying parking facility would yield no revenue because the facility already had extra spaces that were free. Thus the solutions R_j to the dual formulation of the transportation problem correspond to the fees that can be charged for extra spaces, and consequently they also should correspond to the rentals at large public facilities where everyone pays the same price for the same parking service.

APPLICATION OF THEORY TO DATA

In May 1962 the City of Vancouver, British Columbia, carried out a survey of the existing parking situation in order to plan for future space requirements (8). The survey consisted of a compilation of the available parking spaces and their use. The survey encompassed less than 1 square mile of the CBD. Closely following the procedures outlined by the U.S. Bureau of Public Roads (9), the team of 75 men involved in the survey collected data from different city blocks each day and recorded, among other things, the location, size, and fee schedule for all parking facilities. They also recorded the arrival and departure times of all the commuters parking in public facilities between 8:00 a.m. and 6:00 p.m. each day, their home address or last stop, their walking destination, and whether they paid by the hour, the day, or the month. Included in the survey also were parkers at unrestricted curb spaces and public parking facilities in areas adjoining the CBD who were destined for the downtown area.

The survey showed that of the 17,000 spaces located in the area in 1962, 14,000 were available to the general public. These 14,000 spaces divide into two groups, according to the period they were available for continuous use. The first group, called curb, are 2,000 spaces that have time restrictions of 2 hours or less and charge 10 cents per hour. The second group, called commercial, can be used for any length of time. They include unrestricted curb spaces and all off-street lots that are rented by the hour, day, or month. The remaining 3,000 spaces that are not available to the general public are excluded from the subsequent analysis. Also excluded are the users of these spaces because they have a special parking privilege.

The demand for parking space varies throughout the day. In general, there is a heavy flow of people into the spaces as commuters arrive at work between 8:00 a.m. and 9:00 a.m. Then shoppers and people making business calls come and go throughout the middle of the day. Finally, the spaces start to be emptied as commuters journey homeward between 5:00 p.m. and 6:00 p.m. Table 1 shows that total demand for the 14,000 public parking spaces increases rapidly until 10:00 a.m. and continues at slightly less than 10,000 cars until 4:00 p.m.

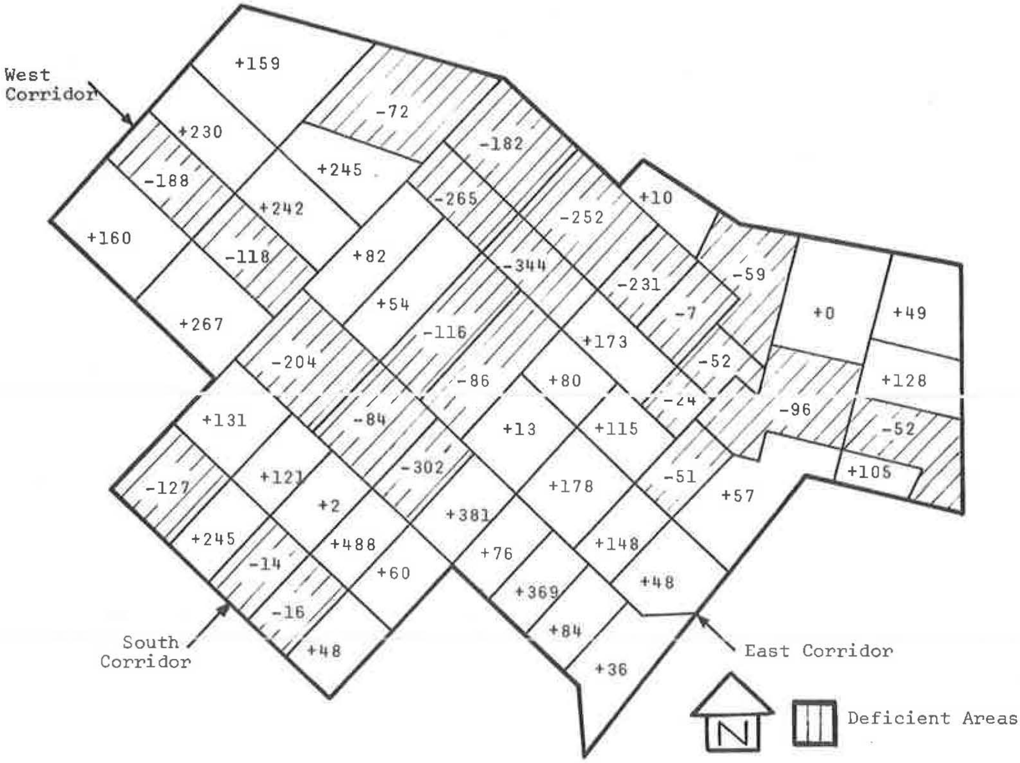
The length of time a person stays at his destination influences his choice of parking facility. Some park for less than 2 hours and legally could use curb spaces. Others stay between 2 and 4 hours and generally pay on a daily basis. The remainder park for more than 4 hours and usually rent parking spaces by the month. The first two groups usually comprise shoppers and people on business calls, while the third group consists of employees and other downtown business people.

The drivers also differ in terms of their home address, their ultimate destination in the CBD, and where they park. Because virtually all traffic reaches the Vancouver CBD through essentially three major corridors (a harbor blocks access from the fourth direction), the classification of home addresses can be considerably simplified by allocating the drivers accordingly. Furthermore, the point where each corridor touches the boundary of the CBD serves as a convenient common origin in determining the relative driving distances to the various parking facilities within the CBD. The classification of ultimate destination also is simplified by dividing the CBD into a number of zones that roughly correspond to 2 city blocks. The same pattern of zones also designates the actual choices of parking location. However, because there are two types of facilities per zone, an additional index (q) is needed to designate curb capacity S_{j1} and commercial capacity S_{j2}. This index also must be added to driver choices X_{ijkpq} and transfer costs C_{ijkpq}.

Table 1. Demand for public parking in the CBD.

Time of Day	No. of Parkers
8:00 a.m.	2,300
9:00 a.m.	7,000
10:00 a.m.	9,200
11:00 a.m.	9,900
12:00 noon	9,900
1:00 p.m.	9,800
2:00 p.m.	9,900
3:00 p.m.	9,900
4:00 p.m.	9,200
5:00 p.m.	7,000
6:00 p.m.	3,300

Figure 1. Supply less demand for parking space.



A comparison of the total demand and supply of parking space per zone shows that there is a severe deficiency near the center of the CBD. This shortage occurs in both short-term and long-term facilities. Figure 1 shows the distribution of the net supply of public parking facilities at 11:00 a.m. after demand and an allowance for the minimum time to change vehicles have been deducted. Table 2 gives the distribution of this demand D_{ikp} by assumed access corridor (i), destination zone (k), and duration class (p). The comparable list for the supply S_{jq} of parking facilities appears in the paper by Brown and Lambe (1). It should be noted that curb and commercial capacities have been reduced by 10 and 20 percent respectively to allow for the normal vacancy rate that occurs in an area of high demand.

The final step in linking the data to the theoretical structure of the previous section is to establish the transfer costs C_{ijkpq}. These depend on the one-way driving and walking distances, their value per foot, their frequency per day, and the direct cost per space for operating a parking facility. The latter is estimated to be \$0.60 per day for maintenance and fee collection at curb and hourly commercial service and \$0.30 per day for monthly commercial services that do not require meters nor parking attendants ($p = 3$, $q = 2$).

Because of the grid-like arrangement of the city streets, the average driving distance from each corridor entry point to parking facilities in each zone is equal to the sum of the absolute differences between the location coordinates of the entry points and zone centroids when measured along axes parallel to the street alignment. The same procedure determines the average walking distance between zones. Thus, if E_i and N_i are the east-west and north-south coordinates of corridor entry point (i), and Y_j and Z_j are east-west and north-south coordinates of parking zone (j), the one-way driving distance is

$$V_{ij} = |E_i - Y_j| + |N_i - Z_j|$$

A similar formula gives the one-way walking distance to destination zone (k),

$$W_{jk} = |Y_j - Y_k| + |Z_j - Z_k|$$

These relationships greatly reduce the computer storage requirements in the next section. In terms of the location of the zones (as given in the paper by Brown and Lambe), $(E_1, E_2, E_3) = (257, 500, 715)$ and $(N_1, N_2, N_3) = (500, 330, 530)$ in 10-ft units.

The criterion for the choice of one parking facility over another is the value of the commuter's leisure by parking at a conveniently located facility as opposed to more money saved by parking at a cheaper facility. In other words, the driving and walking distances involved in parking at, and walking from, each alternative facility must be assigned values in order to facilitate comparison between the various alternatives. Using data on the prices that people are willing to pay to park closer to their destinations in order to reduce walking distance, Lambe (10) has shown that the difference between driving and walking was valued at \$0.15 per 1,000 ft in 1962 for distances under 4,000 ft. Driving can be valued at \$0.05 per 1,000 ft on the basis of an average driving speed of 20 mph in the CBD, plus maintenance, gas, and depreciation costs of \$0.10 per mile. Consequently, the implicit cost of walking is \$0.20 per 1,000 ft.

The average number of one-way trips per day depends on the parking duration of the driver. Spaces that are occupied by people parking for less than 2 hours at a time ($p = 1$) tend to have three users during the 6-hour period between 10:00 a.m. and 4:00 p.m., and consequently they generate six one-way driving (and walking) trips. In a similar manner, spaces occupied by people parking between 2 and 4 hours ($p = 2$) generate three one-way trips per day on average. Finally, spaces occupied for more than 4 hours ($p = 3$) generate two one-way trips. When combined with the previous data on walking distances and cost rates, the trip frequencies give the following set of transfer costs:

$C_{ijk11} = 0.30V_{ij} + 1.20W_{jk} + 0.60$ for 2-hour curb users
$C_{ijk12} = 0.30V_{ij} + 1.20W_{jk} + 0.60$ for 2-hour commercial users

Table 2. Parking demand and prices.

	Demand D_{ikp} by Entry Corridor, Duration, and Destination (in spaces)											
	West Corridor (i = 1)			South Corridor (i = 2)			East Corridor (i = 3)			Theoretical Price		
Zone (j or k)	0 - 2 (p = 1)	2 - 4 (p = 2)	4 + (p = 3)	0 - 2 (p = 1)	2 - 4 (p = 2)	4 + (p = 3)	0 - 2 (p = 1)	2 - 4 (p = 2)	4 + (p = 3)	Curb ($/hour)	Daily ($/day)	Monthly ($/month)
910	12	1	18	20	2	49	8	4	24	0.10	0.60	6.00
911	6	1	15	11	3	26	2	4	13	0.10	0.60	6.00
912	6	2	99	13	14	133	12	6	82	0.10	–	9.00
913	11	4	109	24	8	171	17	12	87	0.13	0.75	9.00
914	4	1	39	4	8	82	4	1	29	0.10	–	6.00
915	1	0	25	5	0	41	0	2	39	0.10	0.65	6.90
916	5	7	72	2	10	115	11	8	82	0.10	0.60	6.00
917	3	0	18	3	0	18	0	0	7	0.10	0.75	9.00
918	12	4	111	14	11	192	10	7	135	0.10	0.90	12.00
920	9	9	89	17	19	142	18	9	58	0.15	1.11	16.20
921	16	4	75	21	12	167	21	4	70	0.23	–	22.10
922	4	1	24	12	12	45	1	5	42	0.15	1.13	16.60
923	1	1	5	6	3	21	3	4	13	0.10	1.04	14.80
924	11	10	102	19	18	192	21	3	97	0.21	1.23	18.60
925	9	5	120	30	13	209	15	11	75	0.26	1.57	25.40
926	37	28	46	90	63	119	69	56	80	0.23	1.37	–
927	2	1	5	5	0	8	0	2	15	0.17	–	–
928	2	2	23	9	2	20	6	0	14	0.14	–	–
929	2	1	20	9	2	38	6	2	33	–	1.15	16.90
930	17	5	22	45	16	58	21	16	37	0.20	–	–
931	12	5	34	31	14	117	18	11	43	0.18	1.06	–
932	3	4	51	8	6	155	3	1	150	0.12	0.87	11.40
933	0	0	1	3	1	3	4	0	3	0.10	0.70	7.90
934	15	4	79	30	13	133	12	21	52	0.15	0.87	11.40
935	2	5	26	3	16	71	12	8	37	0.17	0.99	13.80
936	7	4	48	12	18	94	7	5	38	0.13	1.17	17.30
937	32	34	79	121	61	132	64	42	88	0.20	1.19	17.70
938	3	2	9	3	3	30	8	4	16	0.10	0.90	12.00
939	2	1	10	5	6	30	2	0	27	0.10	1.05	–
940	4	3	35	2	10	58	11	3	62	0.10	–	10.50
941	1	0	6	2	0	15	5	1	23	0.10	0.60	6.00
942	6	0	3	6	6	16	3	1	5	0.10	–	9.00
943	0	1	1	5	0	8	2	0	11	0.10	–	–
944	1	1	8	1	0	11	3	2	13	0.10	0.60	6.00
945	0	0	5	6	0	10	3	0	13	0.10	0.71	8.10
946	9	2	15	13	7	12	4	4	10	0.10	0.60	6.00
947	8	1	10	14	1	21	5	0	20	0.10	–	–
948	0	0	3	5	0	10	2	1	8	0.10	0.60	6.00
949	2	0	3	2	0	14	5	0	0	0.10	0.60	6.00
970	0	0	1	1	0	18	0	0	17	0.10	0.61	6.10
971	0	1	3	3	1	9	0	1	11	0.10	0.60	6.00
972	0	0	0	1	0	1	0	2	4	0.10	–	6.00
973	4	1	11	6	2	19	0	1	44	0.10	–	6.00
974	0	0	2	2	0	13	5	0	20	0.10	0.76	9.10
975	2	3	15	10	5	30	2	3	16	0.10	0.62	6.34
976	1	0	0	1	0	5	0	0	2	–	0.60	6.00
977	6	1	16	13	3	40	6	4	40	0.10	0.91	12.10
978	0	1	9	4	2	26	4	5	34	0.10	0.89	–
979	6	0	10	12	2	57	16	6	36	0.10	0.77	9.30
980	30	21	67	128	66	140	130	72	192	0.17	1.01	–
981	0	0	0	0	0	1	1	0	6	0.10	0.71	8.20
983	3	0	3	15	7	25	18	2	25	0.10	–	–
984	1	1	14	14	14	32	7	1	13	0.10	–	–
985	3	0	7	9	3	22	3	0	16	0.10	–	–
986	3	0	1	0	0	10	3	2	7	0.10	0.60	6.00
987	2	0	2	0	0	18	0	0	19	0.10	–	6.00
Total	338	182	1,624	850	483	3,252	613	359	2,153			

C_{ijk21} = infinity, because not allowed
$C_{ijk22} = 0.15V_{ij} + 0.60W_{jk} + 0.60$ for 2- to 4-hour commercial users
C_{ijk31} = infinity, because not allowed
$C_{ijk32} = 0.10V_{ij} + 0.40W_{jk} + 0.30$ for >4-hour commercial users

RESULTS FROM THEORY AND DATA

An extremely fast computer program written by Thompson and Srinivasan (7) finds the values of P_{ikp}, R_{jq} and X_{ijkpq} for the foregoing transportation problem in 0.32 minutes of processing time on the IBM 370-165 at a cost of less than \$20. Although essentially the same solutions can be obtained in 0.1 minute by pre-assigning the short- and long-term demands when there is sufficient suitable capacity for all of the demand at the destination zone and by combining some of the low demands by corridor, the saving may not be worth the trouble nor the possibility of error. Pre-assignment incidentally reduced the problem from 504 demand and 112 supply equations to 133 and 97 respectively.

The only significant modification to the original computer program was the generation of C_{ijkpq} as needed from E_i, N_i, Y_j, Z_j, Y_k, and Z_k in order to keep computer storage requirements to a manageable level. Minor modifications to the input and output routines consisted of adjustments for receiving the data by zones and subtracting the row and column multipliers from their largest value to produce the basic measurements of driver expenses P_{ikp} and parking fees R_{jq}. The theoretical parking rate for each zone (j) is then equal to the sum of the net parking charge for that zone and the overhead expense, as follows:

$0.167(R_{j1} + 0.60)$ for hourly curb
$(R_{j2} + 0.60)$ for daily commercial
$20(R_{j2} + 0.30)$ for monthly commercial

The resulting optimal assignment of the 504 types of driver to the 112 types of parking facility agreed reasonably well with observed parking patterns. For those who walked, the average walking distance from the parking zone to the destination was 513 ft for people parking less than 2 hours, in comparison with the observed average distance of 1,227 ft. From theory only 155, compared with an observed 776 (out of 1,795), walked, while the rest parked in their destination zones. For people parking between 2 and 4 hours the corresponding averages were 628 and 1,270 ft, and from theory only 116, compared with an observed 543 (out of 1,025), walked. For people parking more than 4 hours the averages were 855 and 1,360 ft respectively, and from theory only 3,031, compared with an observed 4,718 (out of 7,028), walked. The trend to longer walking distances with increases in parking duration confirms the theoretical assumption that people are willing to walk to save larger sums of money that are charged for parking long periods. Observed walking distances generally are 60 percent larger than theoretical ones because other factors (besides price and distance) influence the drivers' choices of parking facilities.

The effect of the access corridor on the walking direction supports the assumption that driving distance also influences the choice of parking location. For those who walked, long-term parkers from the west in theory walk an average of 652 ft east to their ultimate destination, in comparison with an observed eastward distance of 855 ft. In a similar manner, drivers from the east walk 785 ft west in theory, in comparison with the 1,310 ft observed. Finally, drivers from the south walk 918 and 1,408 ft north respectively. Again, the effect of factors other than driving distance tends to increase the observed walking distance.

The corresponding optimal values for parking prices also agreed reasonably well with observed rates. The standard deviation is \$0.13 for the differences between the observed daily rates ranging from \$0.50 to \$1.50 and theoretical rates ranging from \$0.60 to \$1.57. The standard deviation is \$2.15 for the differences between the observed monthly rates ranging from \$5.70 to \$25.00 and theoretical rates ranging from \$6.00 to \$25.40. Because the curb meter rate of \$0.10 per hour is not determined purely by the interaction of supply and demand, a statistical comparison between the observed and theoretical is not meaningful. Table 2 gives the theoretical rates by

Figure 2. Theoretical minus observed curb rates.

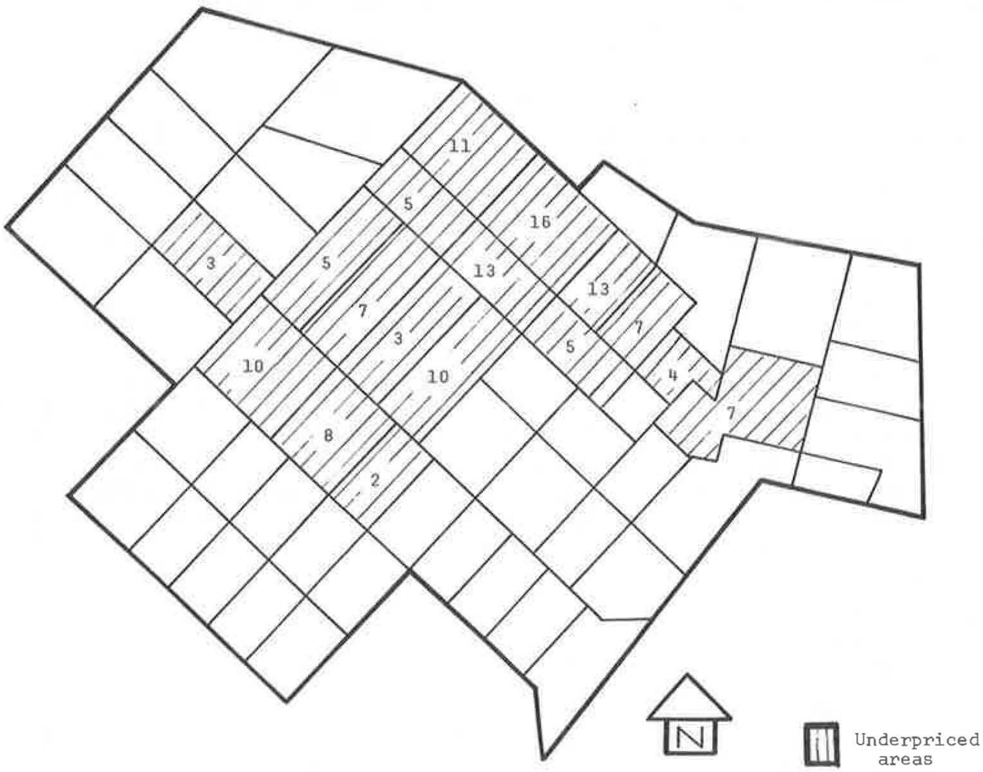


Table 3. Curb price and space utilization.

Theoretical Price ($/hour)	Observed Utilization (fraction)	Number of Zones
>0.25	0.97	1
0.20-0.25	0.92	3
0.16-0.20	0.78	6
0.11-0.15	0.75	7
0.10	0.62	37

zone, and the paper by Brown and Lambe (1) gives the observed rates as weighted averages based on the capacity of the facilities.

Part of the difference between theoretical and observed parking fees may have resulted from the shape and location of the zone boundaries. For example, a person may park at the edge of a zone and walk across the street to a store in a different zone. His theoretical walking distance from the center of the parking zone to the center of the destination zone consequently is much greater than the actual walking distance, thereby exaggerating the gradient between theoretical parking prices. In principle, the zones should be kept as small as one city block, with the boundaries passing down the middle of the blocks instead of the streets.

Figure 2 shows that the theoretical curb rates are higher than the present $0.10 per hour in the central 17 of the 54 CBD zones with curb parking. The local government deliberately sets these low prices and at the same time maintains a maximum 2-hour time restriction in an effort to attract business to downtown stores. Consequently, the demand for these spaces is very high as parkers take advantage of this underpricing feature. A fair amount of overcrowding thus results, causing the average utilization of curb spaces in high-demand areas to rise above the average observed across the entire CBD. By classifying the 54 CBD zones with curb parking according to their theoretical rates and their observed utilization, Table 3 clearly shows that underpricing results in exceedingly high use of these spaces.

From a supply-and-demand point of view, it is obvious that the operators of curb spaces (usually the city) are not charging the most that competition will permit and, as such, are not maximizing their revenue. If the 2-hour limits were removed and curb rates were allowed to rise to the levels determined by the interaction of supply and demand, these rates would approximate the going prices at nearby commercial facilities. In general, the central spaces still would be used by short-term parkers, but they would have to pay more for the privilege. Long-term parkers would not be so attracted to the currently illegal practice of adding coins to the meter every 2 hours, even though this practice would then be legal. Finally, street congestion would be reduced because spaces always would be available to those who are willing to pay for them and there would not be the current financial incentive to hunt for a scarce but cheap curb space.

In conclusion, the use of the transportation model to link theory to observation will help city and other transportation planners to understand the behavior of the average parker and to anticipate changes in the pattern of parking with changes in supply and demand. The most useful aspect of this work to such authorities is the systematic way it links the flow of traffic from the major corridors of the downtown area to the optimal traffic parking pattern and the associated optimal parking rates. Future road networks and parking facilities therefore can be planned with greater accuracy.

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