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Foreword

The papers presented in this Record are related by their focus on the use of traffic control devices to inform or control motorists and include a wide range of issues related to selection of the appropriate traffic control device or technique. The sponsors are TRB Committees on Traffic Control Devices, Traffic Safety in Maintenance and Construction Operations, and Railroad-Highway Grade Crossings.

McCoy and Heimann report on a study of the effectiveness of various speed control systems used in school zones in Nebraska and the relationship between actual vehicle speeds in the school zones and the school-zone speed limit. They found that vehicle speeds were influenced more by the speed characteristics and speed limits of the street than by the established school-zone speed limit.

Jones and Wilson evaluated the effectiveness of decision point signing (DPS) for use in advanced recreational signing. The authors investigated DPS sign effectiveness and appropriate site and route selection.

In the next three papers the issue of work zones is examined. Mousa et al. present a methodology for optimizing the performance of freeway work-zone traffic control through the use of microscopic simulation and linear optimization techniques. Bryden presents the results of full-scale crash tests that were conducted in New York State on typical work-zone traffic control devices to evaluate their performance on impact. Shepard investigated vehicle guidance through work zones by evaluating the effectiveness of two alternative devices for delineation—experimental reflectorized panels in place of steady burn lights and closely spaced raised pavement markers as a supplement to the existing pavement markings.

The last five papers examine traffic control at railroad-highway grade crossings. Marshall and Berg examine the railroad preemption capabilities of actuated traffic signal controllers to determine whether modern controllers allow practical and reasonable preemption design. Shortcomings in preemption logic were identified, and various preemption issues are discussed.

Heathington et al. report on the use of four-quadrant gate systems and a traffic signal system at selected grade crossings. In addition, the authors define the characteristics of crossings that would be candidates for installation of these devices.

Richards et al. report on the results of a before-and-after study of the effectiveness of train predictors at railroad crossings. The authors indicate that positive results were obtained with the installation of the train predictors.

Richards and Heathington assess the effect of length of warning time on driver behavior and safety at grade crossings that have active traffic control devices. The authors recommend guidelines for minimum, maximum, and desirable warning times.

Ryan and Carter present a model that describes the impact of railroad-highway grade crossings on emergency vehicle access and response times.

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Contents

School Speed Limits and Speeds in School Zones

PATRICK T. MCCOY AND JAMES E. HEIMANN

Previous research generally has found driver compliance with school speed limits to be poor, regardless of the type of school zone signing. The lack of compliance has raised questions as to the use of unreasonably low school speed limits to improve school zone safety. A study was conducted to evaluate the effectiveness of various speed control systems used in Nebraska school zones. One objective of this study was to determine the relationship between speeds in school zones and school speed limits. Spot speed data were collected in school zones with school speed limits from 15 to 25 mph. Multiple regression analysis of the data indicated that speeds in the school zones were influenced more by the speed characteristics and limits of the streets on which the zones were located than by the school zone speed limits. Also, on streets with normal speed limits of 35 mph, the 85th percentile speeds in zones with 25-mph school speed limits were lower than those in zones with 15- or 20-mph limits. Therefore, it was concluded that school speed limits lower than 25 mph should probably not be used on these streets.

Previous studies (*1*,*2*) have found no relationship between pedestrian accident experience and school zone speed limits. However, speed limits in school zones are sometimes established in response to the public perception that lower speed limits are a prime factor in school zone safety. Although drivers may acknowledge the lower speed limits as being safe, several studies have found driver compliance with school speed limits to be poor (*3*,*4*)—less than 20 percent. In many cases, the 85th percentile speed was more than 20 mph above the school speed limit. Attempts to increase driver compliance by improved signing and stepped-up enforcement have provided only slight increases in compliance and modest reductions in speed (*5–8*). The lack of compliance may cause such speed control efforts to be counterproductive. Therefore, researchers (*3*) have concluded that the safety of a school zone requires not only the use of effective signing and strict enforcement, but also the establishment of reasonable school zone speed limits.

Carter and Jain (*1*) studied school speed zones in West Virginia for the purpose of developing criteria for establishing reasonable speed limits resulting in safer school zones. Regression analysis was used to investigate the relationship between speeds in school zones and the traffic and physical characteristics of the zones. Approach speed limits and the distance of the school building from the roadway were found to be significant factors influencing speeds in school zones. The relationship between school-zone speeds and these factors was used to develop the necessary criteria as presented in Table 1. These criteria were used to raise the speed limits in three school zones from 15 to 20 mph in one, to 25 mph in another, and to 30 mph in the third zone. The mean speeds in all three zones were lower after the speed limits were raised. Thus, it was concluded that the 15-mph speed limits were not effective in reducing speeds and that traffic should be allowed to operate at a reasonable speed with proper regard for safety rather than be unnecessarily restricted.

The Department of Civil Engineering at the University of Nebraska-Lincoln conducted a study of school speed zones for the Nebraska Department of Roads. The study evaluated the effectiveness of various speed control systems used in Nebraska school zones. The results provided a basis for replacing ineffective systems. The study examined the relationship between school speed limits and the speeds in school zones to establish reasonable school speed limits. The procedure, findings, and conclusions are presented in this paper.

STUDY SITES

Twelve school speed zones considered to be representative of the variety of school speed limits used on urban streets in Nebraska were selected as study sites. The speed zones were on arterial and collector streets in residential areas of six cities. The characteristics of the speed zones are presented in Table 2.

Nine of the school speed zones were for elementary schools, two were for junior high schools, and one was for a high school. The normal speed limits on the zoned streets ranged from 15 to 40 mph, and the school speed limits ranged from 15 to 25 mph. In two of the school speed zones, a limit of 15 mph and a limit of 25 mph were in effect at all times. The remaining 15-mph school speed limits were only in effect when children were present, and the other 25-mph school speed limits were only in effect when yellow beacons in the school speed limit sign assembly were flashing. The one 20-mph school speed limit was only in effect on school days when children were present.

Protection at the crosswalks within the school speed zones consisted of crossing guards and pedestrian-actuated traffic signals. The school crossings in four of the zones had both forms of protection. One of the zones only had crossing guards, and five had only traffic signals. Two of the zones had neither form of protection.

The lengths of the 12 school speed zones ranged from 470 to 1,190 ft. The school buildings were within 150 ft of the street in five of the zones and were visible to approaching traffic in eight of the zones.

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TABLE 1 CRITERIA FOR ESTABLISHING SCHOOL SPEED LIMITS (1)

Distance of School Building from Roadway (ft)	Approach Speed Limit (mph)		
	25	35-45	55
0 - 55	20	20	30
56 - 100	25	25	30
Over 100	25	30	35

All of the school speed zoned streets were two-way, with widths ranging from 30 ft with two lanes to 66 ft with four lanes plus a two-way left-turn lane. Nine of the zones were on tangent, level sections of roadway. The other three were on tangent sections with moderate grades. On-street parking was allowed in only two of the zones.

SPOT SPEED STUDIES

Spot speed studies were conducted at each of the sites. Speeds were measured with a radar speed gun, which was hand-held and plugged into the cigarette lighter of the data collector's car. The car was positioned along the roadway so that the angle between the radar beam and the traffic stream was within acceptable limits. In addition, care was taken to ensure that the car was inconspicuous to approaching traffic. Data were collected for both directions of traffic only if suitable data collection positions could be found; otherwise information was collected for just one direction. Each speed measurement was taken at the same point on the roadway, which was a school crosswalk near the middle of the school speed zone.

Speeds were recorded for passenger cars, trucks, and buses; tractors, off-road vehicles, and motorcycles were excluded. Speeds were measured only for free flowing vehicles, which were those not influenced by the movements of other vehicles on the roadway. In the school speed zones that had traffic signals at the school crossing, speeds were measured only for vehicles traveling unobstructed through the signal on a green light.

Speeds were measured during the designated school crossing times when the school speed zones were in effect and at other times of the day when the school speed zones were not in effect. Whether or not children were present was noted. Children were considered to be present if they were visible to traffic approaching the crosswalk at which speeds were measured. Thus, children in an unfenced playground near the street and visible to traffic were defined as present, but if the playground was fenced off from the street, the children were considered not present.

DATA ANALYSIS

The objective of the analysis was to determine the relationship between the speeds in the school speed zones and the school speed limits. Previous research (*1*) had found that speeds in school zones were significantly influenced by the normal speed limit on the street. Therefore, the analysis considered the influence of the school speed limits on speeds in the zones not only when the school speed limits were in effect, but also when the school speed limits were not in effect. The speed characteristics considered were the mean and 85th percentile speeds in the zone when the school speed limit was not in effect, the normal speed limit on the zoned street, and the recommended normal speed limit for the street as determined by the procedure in the Institute of Transportation Engineers (ITE) *Transportation and Traffic Engineering Handbook* (*9*).

Multiple linear regression analysis was used to determine the relationship among the 85th percentile speeds in the zones, the school speed limits, and the speed characteristics of the street. Four models were examined. The dependent variable in each model was the 85th percentile speed in the zone when the school speed limit was in effect. Each model had two independent variables. One of them was the school speed limit and the other was one of the following:

- The normal speed limit on the street when the school speed limit was not in effect,
- The ITE-recommended normal speed limit for the street,
- The mean speed in the zone when the school speed limit was not in effect, or
- The 85th percentile speed in the zone when the school speed limit was not in effect.

Some school speed limits were in effect when children were present, some when beacons were flashing, and some were in effect at all times. Therefore, for the purpose of the data analysis, it was necessary to establish a common definition for determining when the various types of school speed limits were or were not in effect.

Because all of the school speed limits were in effect when children were present during the designated school crossing times, this was the primary criterion used to define when a school speed limit was in effect. A school speed limit desig-

TABLE 2 STUDY SITES

Study Site	School Speed Zone: City	School Speed Zone: School	School Speed Zone: Street	Speed Limit (mph): Street	Speed Limit (mph): School	Crossing Control: Crossing Guard	Crossing Control: Pedestrian Signal	Zone Length (ft)	School Building: Within 150 ft of Street	School Building: Visible from Street	Street Geometrics: Width (ft)	Street Geometrics: Number of Lanes	Street Geometrics: Vertical Alignment	On-Street Parking
1	Kearney	Bryant	16th Street	15	15	yes	no	835	yes[d]	yes	30	2	level	none
2	Lavista	Lavista J.H.S.	Giles Road	35	15[a]	no	no	735	no	no	30	2	level	none
3	Grand Island	Stolley Park	Stolley Park	35	15[a]	yes	yes	700	yes[d]	yes	40	2	level	both sides
4	Grand Island	R.J. Barr J.H.S.	Stolley Park	35	15[a]	no	yes	800	no	yes	40	2	level	both sides
5	Blair	Blair West	Washington	35	20[b]	yes	yes	800	yes[d]	yes	40	4	level	none
6	Beatrice	Lincoln	19th Street	35	25	no	yes	860	yes	yes	40	2	level	none
7	Kearney	Kearney High School	39th Street	35	25[c]	no	no	470	no	yes	44	4	level	none
8	Lincoln	Clinton	Holdredge	35	25[c]	yes	yes	885	yes[d]	yes	30	2[e]	level	none
9	Lincoln	Prescott	South Street	35	25[c]	no	yes	670	no	no	44	4	grade	none
10	Lincoln	May Morley	70th Street	40	25[c]	no	yes	1,190	no	yes	66	4[e]	level	none
11	Lincoln	Ruth Prytle	84th Street	40	25[c]	no	yes	740	no	no	44	4	grade	none
12	Lincoln	Zeman	56th Street	40	25[c]	yes	yes	950	no	no	48	4	grade	none

[a] When children are present.
[b] On school days when children are present.
[c] When flashing.
[d] Fence between school grounds and street.
[e] Plus a two-way left-turn lane median.

nated by a speed limit assembly with the message WHEN CHILDREN ARE PRESENT was considered to be in effect when children were present during the designated school crossing times. A school speed limit designated by a speed limit assembly with beacons and the message WHEN FLASHING was considered to be in effect when children were present during the designated school crossing times and when the beacons were flashing. A school speed limit designated by a standard speed limit sign without any qualifying message was considered to be in effect only when children were present during the designated school crossing times, even though it was legally in effect at all times.

The primary criterion used to define when a school speed limit was not in effect was when children were not present during times other than the designated school crossing times. A school speed limit designated by a speed limit assembly with the message WHEN CHILDREN ARE PRESENT was considered not to be in effect only when children were not present during times other than the designated school crossing times. A school speed limit designated by a speed limit assembly with beacons and the message WHEN FLASHING was considered not to be in effect when children were not present and the beacons were not flashing. A school speed limit designated by a standard speed limit sign without any qualifying message was considered not to be in effect when children were not present during times other than the designated school crossing times, even though it was legally in effect at all times.

FINDINGS

The spot speed data collected were analyzed to compute the speed distribution parameters used in the regression analysis. The parameters computed were, for the zones when the school speed limits were in effect, (a) the 85th percentile speeds and (b) the mean speeds, and for the zones when the school speed limits were not in effect, (c) the 85th percentile speeds. The 10-mph paces of the speeds in the zones when the school speed limits were in effect were also determined. They were used, together with the 85th percentile speeds in the zones when the school speed limits were not in effect, to determine the ITE-recommended normal speed limits for the zoned streets. The results of these computations are presented in Table 3. Spot speed data were collected in both directions of traffic in five of the zones and in only one direction in the other seven zones.

The four models obtained from the regression analysis are presented in Table 4. In each model, the 85th percentile speed in a zone when the school speed limit was in effect is expressed as a linear function of the school speed limit and as one of the following variables:

- The mean speed in the zone when the school speed limit was not in effect,
- The 85th percentile speed in the zone when the school speed limit was not in effect,
- The ITE-recommended normal speed limit for the street, or
- The existing normal speed limit on the street.

The coefficients of determination for the first two models, which are functions of the mean and 85th percentile speeds in the zone when the school speed limit was not in effect, were higher than those for the last two models, which are functions of the ITE-recommended speed limit and the normal speed limit on the street. All of the models are statistically significant at the 0.01 percent level.

The coefficient of the school speed limit term was negative in all models, indicating that higher school speed limits would result in lower 85th percentile speeds in school zones when the school speed limit was in effect. The coefficient of the other independent variable was positive in all models, which indicated that the 85th percentile speeds in school zones when the school speed limit was in effect would be higher on streets with higher mean speeds, higher 85th percentile speeds, and higher normal speed limits. Also, the absolute values of the school speed limit coefficients were smaller than those of the other independent variables. Thus, the school zone speeds were more sensitive to the speed characteristics of the street than to the school speed limits.

According to the models, 25-mph school speed limits would be the most effective, because they would result in the lowest 85th percentile speeds when the school speed limits were in effect. However, the regression coefficients of the school speed limit terms in the four models were not significantly different from zero at the 5 percent level of significance. This result indicates that the 85th percentile speeds in school zones when the school speed limits were in effect were independent of the school speed limits. Consequently, the results of a stepwise multiple regression analysis of the data conducted at the 5 percent level of significance revealed that the 85th percentile speeds in school speed zones when the school speed limits were in effect were simply a function of the speed characteristics of the streets (see Table 5). Once again, the highest coefficients of determination were for the first two models, indicating that the mean and 85th percentile speeds in the zones when the school speed limits were not in effect were better predictors of the 85th percentile speeds in the zones when the school speed limits were in effect.

A comparison of the speeds in the school zones that were on streets with normal speed limits of 35 mph is presented in Table 6. In the six school zones on streets where the normal speed limit agreed with the ITE-recommended normal speed limit, the 85th percentile speeds when the school speed limits were in effect were lower in the 25-mph zones than they were in the 15- and 20-mph zones. However, in the six school zones on streets where the ITE-recommended normal speed limit was 40 mph, the 85th percentile speeds when the school speed limits were in effect were higher in two of the three 25-mph zones than they were in the 15-mph zones. Nevertheless, the results of a t-test conducted at the 5 percent level of significance indicated that the average 85th percentile speed (35.5 mph) in the 25-mph zones was not significantly higher than the average 85th percentile speed (35.2 mph) in the 15-mph zones when the school speed limits were in effect. This comparison indicates that the 25-mph school speed limits were more effective than the 15- or 20-mph school speed limits on streets with 35-mph normal speed limits, provided that this normal speed limit was consistent with ITE guidelines. Also, the comparison indicates that the effects of the 25-mph school speed limits were not significantly different from those of the lower speed limits on streets with 35 mph normal speed limits when the speed limit was lower than that recommended by the ITE guidelines.

TABLE 3 SPEED LIMITS AND SPEED DISTRIBUTION PARAMETERS

Study Site	Direction	Speed Limit (mph) Street	ITE	School	Not-In-Effect Mean Speed	85th Percentile Speed (mph) Not-In-Effect	In-Effect	Sample Size Not-In-Effect	In-Effect
1	EB	15	30	15	26.1	31.1	25.4	58	55
	WB	15	30	15	26.3	30.2	23.4	74	35
2	EB	35	40	15[a]	36.9	40.8	36.5	63	88
	WB	35	40	15[a]	34.8	39.5	35.2	91	155
3	EB	35	40	15[a]	35.0	37.6	34.2	124	80
	WB	35	35	15[a]	34.1	37.4	34.2	128	70
4	EB	35	35	15[b]	35.0	37.4	33.4	96	107
5	EB	35	35	20[b]	30.6	34.9	31.7	89	56
	WB	35	35	20[b]	31.8	35.4	33.5	64	80
6	NB	35	35	25	31.2	35.3	29.7	90	115
7	EB	35	40	25[c]	37.3	40.7	37.0	165	60
	WB	35	40	25[c]	38.2	42.1	36.9	158	42
8	WB	35	35	25[c]	32.3	35.2	30.3	434	222
9	EB	35	40	25[c]	35.6	39.4	32.7	322	112
10	NB	40	40	25[c]	38.2	41.9	34.4	569	44
11	SB	40	45	25[c]	40.3	44.0	35.6	379	118
12	SB	40	45	25[c]	39.6	44.4	38.1	319	50

[a] When children are present.

[b] On school days when children are present.

[c] When flashing.

TABLE 4 RESULTS OF REGRESSION ANALYSIS

Model	Equation[a]	R^2
1	$Y = 3.1 - 0.12L_{SCH} + 0.95\bar{X}$	0.86
2	$Y = -0.86 - 0.14L_{SCH} + 0.97X_{85}$	0.85
3	$Y = 3.7 - 0.10\ L_{SCH} + 0.84L_{ITE}$	0.74
4	$Y = 19 - 0.13L_{SCH} + 0.51L_{STR}$	0.73

[a] Y = 85th percentile speed in zone when school speed limit was in effect (mph).

L_{SCH} = School speed limit (mph).

$\bar{X}$ = Mean speed in zone when school speed limit was not in effect (mph).

X_{85} = 85th percentile speed in zone when school speed limit was not in effect (mph).

L_{ITE} = ITE recommended normal speed limit for the street (mph).

L_{STR} = Normal speed limit on the street (mph).

TABLE 5 RESULTS OF STEPWISE REGRESSION

Model	Equation[a]	R^2
1	$Y = 2.9 + 0.88\bar{X}$	0.84
2	$Y = -0.63 + 0.89X_{85}$	0.82
3	$Y = 3.6 + 0.78L_{ITE}$	0.73
4	$Y = 17 + 0.47\ L_{STR}$	0.71

[a] Y = 85th percentile speed in zone when school speed limit was in effect (mph).

$\bar{X}$ = Mean speed in zone when school speed limit was not in effect (mph).

X_{85} = 85th percentile speed in zone when school speed limit was not in effect (mph).

L_{ITE} = ITE recommended normal speed limit for the street (mph).

L_{STR} = Normal speed limit on the street (mph).

TABLE 6 SPEEDS IN ZONES ON STREETS WITH 35-MPH NORMAL SPEED LIMITS

ITE-Recommended Normal Speed Limit (mph)	Speed (mph) by School Speed Limit (mph)		
	15	20	25
35	33.4	31.7	29.7
	34.2	33.5	30.3
40	34.2	—	32.7
	35.2	—	36.7
	36.2	—	37.0

NOTE: Speeds are 85th percentile speeds in zone when school speed limit was in effect.

The findings of this study are consistent with those of the West Virginia study (*1*), which found that the normal speed limit had a significant influence on speeds in school speed zones and that the raising of school speed limits actually resulted in lower speeds in school speed zones. However, the findings may be confounded by factors that may influence speeds in school speed zones but that were not accounted for in this study. For example, some studies (*3,5*) have found that school speed limits signed with flashing beacons were more effective than passive forms of school speed limit signing. In this study, most of the 25-mph school speed zones were signed with flashing beacons, but the 15- and 20-mph school speed zones were not, which may suggest the reason why the 25-mph school speed zones had lower speeds. However, based on previous experience (*8*), it was assumed that the effects of the signing differences were minimal, because the school speed limits were defined as being in effect only when children were present. Also, previous research found that increased enforcement can reduce speeds in school zones. Although this study indicates otherwise, it is possible that the 25-mph school speed zones have received more enforcement than the 15- and 20-mph school speed zones, and therefore were more effective in reducing speeds.

CONCLUSIONS

The effectiveness of school speed limits in reducing speeds is limited. Speeds in school zones are influenced more by the normal speed limits and speed characteristics of the streets on which the zones were located than by the school speed limits. Also, 25-mph school speed limits are more effective than 15- or 20-mph school speed limits on streets with a normal speed limit of 35 mph, when the normal speed is consistent with ITE guidelines. Therefore, school speed limits lower than 25 mph should probably not be used on such streets.

The scope of this study was limited to school speed limits of 15 to 25 mph on urban streets with normal speed limits up to 40 mph. Additional studies are needed to substantiate these findings before the conclusions can be recommended as school speed zone policy. The limitations of this study can be avoided by accounting for the effects of school speed zone signing and enforcement. The additional studies should include 15- and 20-mph zones on streets with normal speed limits above 35 mph and consider the safety effects of school speed zones.

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Advanced Signing for Recreational and Historical Sites

C. PAUL JONES AND EUGENE M. WILSON

In 1988 a pilot study was conducted to evaluate the effectiveness of advanced recreational signing. The central signing concept evaluated can best be described as decision point signing (DPS). This DPS concept provides information concerning a recreational or historical site far in advance of the site (40 to 60 mi) and often in advance of highway junctions. The purpose of this signing approach is to advise the traveler that he is approaching the site and provide time for him to decide whether to visit it. There are a number of unknowns associated with the DPS concept, including whether DPS has an effect, and which routes and sites should be considered for DPS. Because any sign along a highway represents a hazard to an errant motorist and costs about \$14 to \$16 per ft^2 installed, answers to the previous questions are important to the decision maker. The results of this pilot study provide insight into these questions. A brief review of signing and recreational studies associated with increasing tourism is provided in this paper. The results of the studies conducted at both Devils Tower and South Pass City are presented. Specific conclusions and recommendations are made concerning DPS for recreational and historical sites.

All highway signs are classified as regulatory, warning, or guide signs. In order for any sign to be effective, it must fulfill a need, command attention, convey a clear simple message, command respect, and be placed to allow adequate response time (*1*). Recreational signs are classified as guide signs and have a green or brown background with a white legend. Currently, no standard exists as to the effective distance for most advanced highway signs, but the *Manual on Uniform Traffic Control Devices* (MUTCD) (*1*) suggests using a distance of 1 or 2 mi for recreational guide signs. An ITE committee (*2*) reported that for advanced airport signing, a recommended signing range is 10–25 mi from the airport.

Increasing tourism is a major goal of most states and this has resulted in increased pressure on highway departments and departments of transportation to provide more tourist-oriented direction (TOD) signing. Vermont, Oregon, Minnesota, and Nebraska have reported major programs in this area (*3–6*). For example, a five-step tourism program was established in Nebraska (*6*). After the state's attractions and scenic routes were identified, an aggressive communications program was undertaken. This program involved creating a map emphasizing the attractions, establishing travel information centers, and installing signs with the name and logo of the attraction. Visitor centers were staffed with part-time college students who describe local surroundings to visitors, and additional signing was installed at these sites. Radio stations also provide travel and tourist information in spot announcements throughout the day. The last step involved television advertisements for use in and out of Nebraska. Unfortunately, no formal evaluation of the programs in Vermont, Oregon, Minnesota, or Nebraska was found in the literature, although all states indicated that they were pleased with the results of their program to increase tourism. The increased demand for roadside signing and the lack of knowledge concerning the effectiveness of signs precipitated the study reviewed in this paper. Although only a pilot study, it provides insight into the value of decision point signing (DPS) and associated visitor-use data.

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STUDY RESULTS

Devils Tower is America's first national monument and is located in northeastern Wyoming, 30 mi north of I-90. South Pass City was a major gold mining town in the late 1860s and is located on the Oregon Trail in central Wyoming, 2 mi south of State Route 28. A total of 1,159 interviews were conducted at the two sites (see Table 1). Daily visitor use of Devils Tower during the 10-day interview period averaged 570 vehicles/day (vpd). Slightly over 83 percent of the motorists surveyed were traveling along I-90. It is estimated that about 9.4 percent of the I-90 motorists visited Devils Tower during the summer interview period. Almost 100 percent of the visitors to South Pass City were interviewed during the 10-day period. An estimated 2.8 percent of the Wyoming Route 28 motorists visited South Pass City during the interview period. The type and location of signs for the before-and-after study are shown in Figures 1–3.

Table 2 presents the sources of information that motorists indicated as influencing their decision to visit the site. Indicating more than one source of information was permitted. From the Devils Tower survey, an average of 2.3 sources per respondent was indicated. For South Pass City, the average number of sources indicated by respondents was 1.2.

The role of informal information sources in repeat visitor use is readily apparent. A major difference between the two sites was the high percentage of out-of-state visitor use at Devils Tower compared with that at South Pass City (see Table 3). Of the visitors interviewed, 95 percent were from outside Wyoming compared with 56 percent of the visitors for South Pass City.

Concerning the value of advanced recreational signing, two questions associated with visitor use were of particular interest. Did the visitor plan to stop at the site and was a large percentage of the site's visitors retired persons? Table 4 pro-

TABLE 1 SITE INTERVIEWS

	Devils Tower	South Pass City
Before Advanced Signs	360	229
After Advanced Signs	365	205
Total	725	434

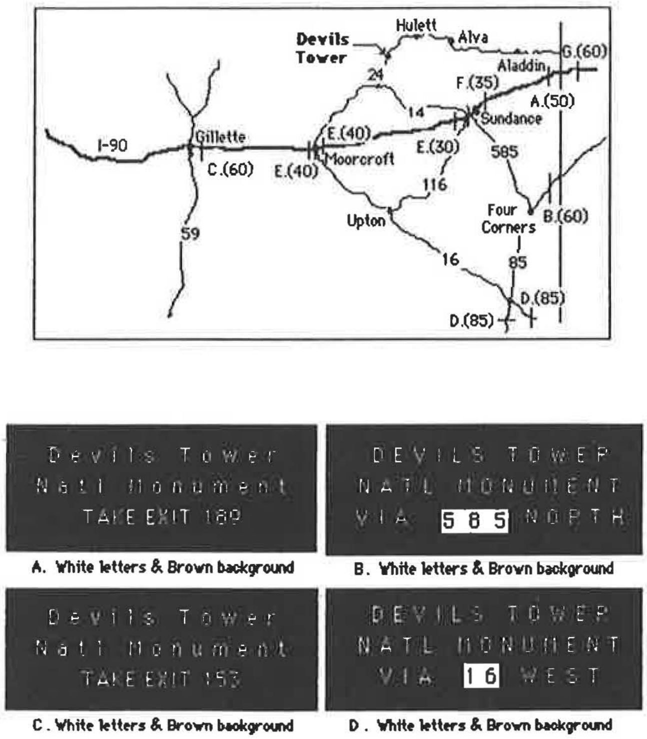

FIGURE 1 Devils Tower signing. Signs A, B, C, and D were covered during the first five-day period. A slash indicates where the sign was located. The letter indicates which sign was used at each location. The number in parentheses following the letter designates the distance in miles from the sign to Devils Tower. Signs A and C measure 6 ft × 16 ft. Sign B measures 5 ft × 10 ft. Sign D measures 5 ft × 9 ft.

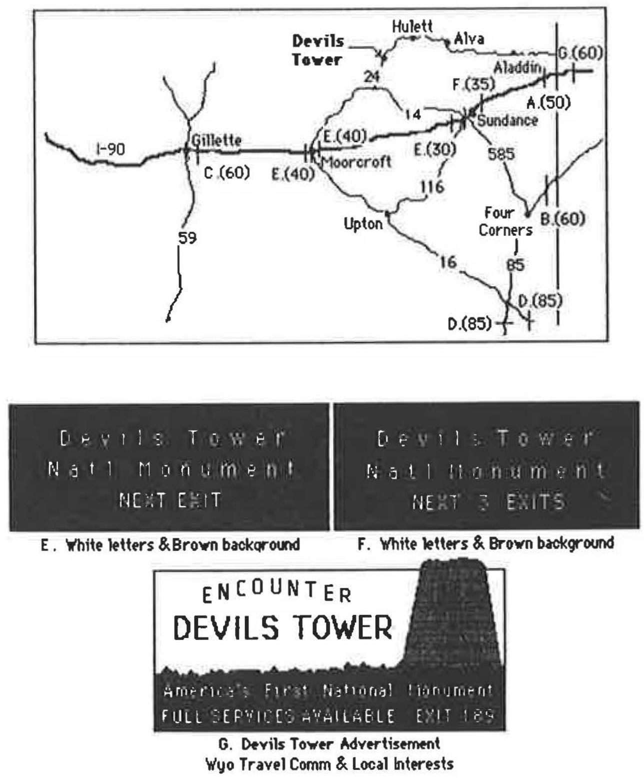

FIGURE 2 Devils Tower signing. Signs E, F, and G were not covered during the study. A slash indicates where the sign was located. The letter indicates which sign was used at each location. The number in parentheses following the letter designates the distance from the sign to Devils Tower. Signs E and F measure 6 ft × 17 ft. No dimensions were available for Sign G, but it is larger than Signs E or F.

vides insight into both questions. Devils Tower was not a preplanned stop for 14.2 percent of the visitors interviewed. South Pass City was not a preplanned stop for 21.7 percent of the visitors interviewed. Before the study, it was thought that retired persons would be more prone to impulse visitation because they would have more leisure time. The data presented in Table 4 indicate that this group (retired persons) was not overrepresented at either site. However, a higher percentage of retired persons chose to visit either site after their vacation began than any other group.

Road signing was indicated by 9.4 percent of the motorists interviewed as an information source. Obviously road signing guides most motorists, but they were specifically asked whether the road signing influenced their decision to stop at the site. Before-and-after data for both sites are presented in Table 5. The data indicate that signing influenced less than 5 percent of the visitors interviewed at Devils Tower and 10 percent at South Pass City. Of the 34 motorists influenced by the road signs to visit Devils Tower, only 1 was a resident of Wyoming. For South Pass City, 43 motorists indicated that signing influenced their visit, and 9 were from Wyoming. The addition of DPS appeared to provide an increase in visitor use at South Pass City.

Statistical analysis (chi square test) was performed on the data for both sites to determine whether the addition of DPS had an effect. The test results are presented in Table 6. When stop type (a stop at Devils Tower planned before or after the start of the vacation) and influence were compared, the two

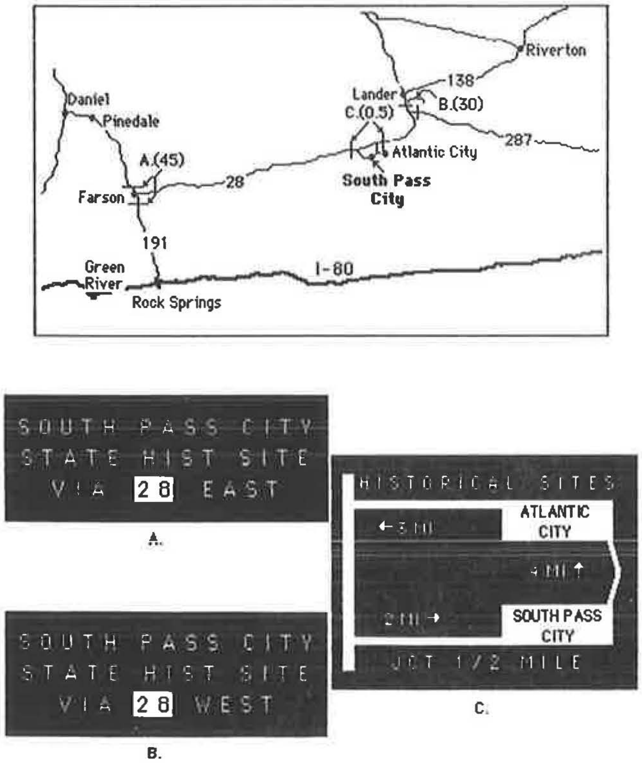

FIGURE 3 South Pass City signing. Signs A and B were covered or left uninstalled for the first five-day period. Sign C, the diagrammatic sign, was not covered. A slash mark indicates a sign location. The letter designation indicates the sign associated with the slash marks. Numbers in parentheses designate the distances in miles from the signs to South Pass City. Signs A and B measure 5 ft × 10 ft. Sign C measures 10 ft × 12 ft.

variables were highly dependent ($p = 0.0000$). Motorists influenced by the signs had not planned to visit Devils Tower before the start of their vacation and vice versa.

Although retired people visiting South Pass City made up only 8.5 percent of the motorists interviewed, 21.6 percent of this group indicated that they were influenced by road signing. The group and influence variables tested dependent for South Pass City, indicating that retired persons were more influenced by the signing than the other groups. Also, because the variables tested dependent after the four new decision point signs had been installed, these signs may have helped divert motorists to South Pass City.

MAJOR STUDY FINDINGS AND CONCLUSIONS

Promoting the many historical and recreational opportunities in Wyoming is a major goal of the state. In this effort to increase tourism, there are many approaches. Once a visitor is in Wyoming, lengthening the time of stay may occur because of increased impulse visitation resulting from additional signing for the opportunities that exist. In order to evaluate the effectiveness of increased signing, additional word message road signs were installed for both South Pass City and Devils Tower in advance of route junctions. These additional signs were located from 40 to 60 mi in advance of the primary junction to each site. The concept of DPS is to provide information concerning this tourist attraction in the hope of diverting motorists to the site and thereby increasing their stay in Wyoming.

In this study of actual visitors to the two sites, over 60 percent of the motorists surveyed obtained information about the sites from brochures, the Travel Commission, a previous visit, or in the case of Devils Tower, the movie "Close Encounters of the Third Kind." Television, radio, newspapers, and magazines each contributed less than 1 percent as an information source for South Pass City. Except for magazines (8.4 percent), these same sources also contributed little as a primary information source for Devils Tower. Conversation with others (word of mouth) was an information source indicated by 31 percent of the surveyed motorists at Devils Tower and 17 percent of those surveyed at South Pass City. Road signs were chosen as an information source by 9 percent of the motorists at both sites.

A major question was whether the advanced recreational signs were observed by motorists. At Devils Tower, the two advanced signs on I-90 were observed by 82.6 percent (247 out of 299) of the surveyed motorists who drove by either sign. Advanced recreational signs on other routes were also observed by 82.7 percent (24 out of 29) of the surveyed motorists. It is important to recall that 83 percent of the surveyed motorists at Devils Tower were traveling on I-90. The advanced recreational signing observance for South Pass City was 67.5 percent (110 out of 163) of the surveyed motorists. Observance of the diagrammatic sign of South Pass City was much higher (97 percent, or 385 out of 396).

Signs influenced only 4.7 percent (34 out of 725) of the motorists surveyed to divert from their trips to Devils Tower. This percentage was about the same for the before (4.4 percent) and after (4.9 percent) portions of the study. Road signs influenced 10 percent (43 out of 434) of the South Pass City motorists to divert and visit that site. There was a 6.2 percent difference in sign influence between the before (7.0 percent) and after (13.2 percent) portions of the study. As a group, retired persons (8 out of 31, or 21.6 percent) were most influenced by the road signs.

Based on this study of South Pass City and Devils Tower, the following conclusions concerning DPS are drawn:

- Advanced recreational signing (DPS) using word messages appears to be ineffective in diverting motorists to visit historical sites.
- DPS (word message signs) at route junctions 40–60 mi from the site results in little if any increase in site visitation.
- If additional guide signing for historical and recreational sites is used, it should be focused on higher-volume routes immediately adjacent to the site.
- Because of the apparent value of the diagrammatic signs at South Pass City, additional study of this concept is recommended for advanced recreational signs.

RECOMMENDATIONS

Because this was only a pilot study for advanced recreational signing, a recommendation for future research includes sur-

TABLE 2 INFORMATION SOURCES INFLUENCING HISTORICAL SITE VISITATION

Information Source	Devils Tower	South Pass City
Radio	1.5%	0.2%
Television	5.0%	0.2%
Newspaper	0.8%	0.5%
Magazines	8.4%	0.0%
Road Map	19.9%	7.1%
Conversations	31.4%	17.1%
Books	23.6%	12.7%
Road Signs	9.4%	9.4%
Don't Remember	0.0%	0.0%
Other	64.9%	68.0%
Visited Site Before	29.7%	53.7%
Travel Commission/ Brochures	11.0%	9.0%
"Close Encounters Of The Third Kind" (the movie)	20.5%	
AAA Travel Agency Information	3.7%	5.3%

TABLE 3 RESIDENCE OF VISITORS INTERVIEWED

	Devils Tower	South Pass City
Wyoming	29 (4.0%)	191 (44.0%)
Border State	105 (14.5%)	74 (17.1%)
Other State	560 (77.2%)	164 (37.8%)
Foreign Country	31 (4.3%)	5 (1.2%)
Total	725 (100.0%)	434 (100.0%)

TABLE 4 PREPLANNED VACATION STOP AT THE HISTORICAL SITE

Devils Tower		
	Planned Stop Before Vacation Started	Planned Stop After Vacation Started
Couple or Alone	187 (25.8%)	40 (5.5%)
Retired Persons	51 (7.0%)	19 (2.6%)
Family w/Children	284 (39.2%)	30 (4.2%)
Other	100 (13.8%)	14 (1.9%)
Total	622 (85.8%)	103 (14.2%)
South Pass City		
	Planned Stop Before Vacation Started	Planned Stop After Vacation Started
Couple or Alone	110 (25.3%)	39 (9.0%)
Retired Persons	22 (5.1%)	15 (3.5%)
Family w/Children	141 (32.5%)	31 (7.1%)
Other	67 (15.4%)	9 (2.1%)
Total	340 (78.3%)	94 (21.7%)

TABLE 5 SIGNING INFLUENCING SITE VISIT

Devils Tower			
Influence	Before DPS	After DPS	Total
None	344 (95.6%)	347 (95.1%)	691 (95.3%)
Influenced	16 (4.4%)	18 (4.9%)	34 (4.7%)
South Pass City			
Influence	Before DPS	After DPS	Total
None	213 (93.0%)	173 (86.8%)	391 (90.1%)
Influenced	16 (7.0%)	27 (13.2%)	43 (9.9%)

TABLE 6 STATISTICAL ANALYSIS OF ROAD SIGN INFLUENCE

Devils Tower

Variables	Chi Square	DOF	p-Value	Reject H_0
Group by Influence	6.72	3	0.0815	No
Group by Influence Before	3.44	3	0.3280	No
Group by Influence After	4.24	3	0.2369	No
Stoptype by Influence	169.09	1	0.0000	Yes

South Pass City

Variables	Chi Square	DOF	p-Value	Reject H_0
Group by Influence	7.91	3	0.0479	Yes
Group by Influence by Before	2.10	3	0.5519	No
Group by Influence by After	10.68	3	0.0136	Yes
Stoptype by Influence	167.55	1	0.0000	Yes

Group: Couple or alone, retired persons, family, other.

Stoptype: Planned before trip or planned after trip.

Influence: Sign influence: yes, no.

veying motorists not visiting either site. It is important to know whether motorists not visiting either site are observing the signs. More research is also needed to understand the value of repetitive signing. If the diagrammatic signs are added, a placement between 1 and 10 mi in advance of the junction to the site is recommended for study. Recreational and historical sites should also be evaluated. Criteria need to be established to reflect which sites are candidates for advanced recreational signing.

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Integrating Microscopic Simulation and Optimization: Application to Freeway Work Zone Traffic Control

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This paper presents a methodology for optimizing performance of a traffic system on the basis of simulated observations of its microscopic behavior. The method integrates simulation and optimization submodels for describing traffic flow on urban freeway lane closures. The stochastic nature of traffic is accounted for in determining the true system response to traffic control variables. The simulation submodel has been validated at a series of work sites in the Chicago area expressway system. The optimization submodel optimizes a single objective function subject to a set of linear constraints. Preliminary model applications included the determination of an optimum merging strategy to be adopted by traffic entering the work zone in lanes to be closed for traffic. The model recommendation yielded the lowest average travel time in the work zone and, interestingly, did not incorporate many early merges; the latter is often viewed as a desired merging strategy. In addition, the optimum merging strategy varied with the traffic flow level entering the work zone and with the character of the objective function to be optimized.

Control solutions for complex traffic systems often require an explicit optimization of one or more system performance measures. Traffic signal system parameters such as cycle length, splits, and offsets are typically determined from an optimization of delays, queue lengths, etc. On urban freeways, the specification of ramp metering rates, priority lanes, and priority entry is formulated to optimize overall corridor performance using the FREQ model (*1*). The system of interest in this study concerns lane closure procedures on urban freeways. Several key decisions must be made for such a system to operate in a safe and efficient manner, such as the location of advance warning devices, the length and position of the construction taper, the distance between tapers for multilane closures, the location and layout of ramp tapers, etc. Although empirical guidelines exist for these parameters, as listed in the *Manual on Uniform Traffic Control Devices for Streets and Highways* (MUTCD) (*2*), they have yet to be evaluated in the context of overall optimal system performance.

A key hindrance toward integrating traffic simulation and optimization models lies in the nature of the traffic process. Traffic flow descriptors are generally stochastic in nature, ranging from headways and speeds to critical gaps and lane selection. This behavior complicates the evaluation of the system performance and consequently the process of reaching an optimal solution. Traffic models that represent the traffic system in terms of its microscopic components (individual drivers, vehicles, etc.), will usually require a large programming and debugging effort, exhibit more stringent storage requirements, and consume more computing time, while providing greater resolution and potentially more accuracy relative to alternative comparison (*3*). Although there is some consensus among traffic analysts as to the value of microscopic traffic simulation models in mimicking traffic behavior, their utility has been curtailed as a result of their inability to formally optimize system performance (short of trial-and-error procedures, which still would not guarantee an optimal solution). Examples of such evaluation models include NETSIM (*4*) for signalized networks, INTRAS/FREESIM (*3*) for freeway corridors, and FREECON (*5*) and ARTWORK (*6*) for freeway and arterial lane closures, respectively.

It is interesting to examine some of the popular design models that do contain some mechanism for optimizing system performance. Signal timing models such as SOAP (*7*) for isolated intersections, Passer II (*8*) or MAXBAND (*9*) for arterials, and TRANSYT (*10*) or SIGOP (*11*) for networks are all deterministic, macroscopic models in which traffic is represented in terms of average flow rates that occur consistently (or at least in a predetermined pattern) throughout the simulation exercise. It has been common practice among traffic engineers to use design models such as TRANSYT to optimize network signal settings and subsequently evaluate those settings in a more realistic traffic environment as in the NETSIM model.

This gap in current modeling capability has provided the impetus for the research effort described herein. The selection of the freeway lane closure traffic system was in part due to the authors' interest and experience in the topic, and also to fulfill the requirements of a research grant. In addition, the system provided a unique environment for testing some of the well-established traffic control procedures in the freeway work zone area.

RESEARCH APPROACH

The approach employed in this study consisted of the development of an integrated microscopic simulation and optimization model for urban freeway lane closures. A review of existing microscopic models indicated that (a) none of the models contained a formal optimization feature and (b) freeway work

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zone models did not account for the presence of ramps in modeling traffic flow. It was also important to limit the system size to maintain a viable optimization scheme that would require multiple simulation runs. Thus, the concept of using a large-scale system such as INTRAS/FREESIM, although noble in purpose, was beyond the computational limits and scope of a first-cut investigative study of this nature.

This paper presents the results of the integrated model development. The optimization submodel borrows an algorithm developed by Azadivar and Talavage (*12*), which is specifically designed to optimize stochastic systems. The simulation submodel is capable of representing an urban freeway environment with or without lane closures or ramps. The model was successfully interfaced with the optimization procedure to yield the preferred traffic control solutions. It must be stated that the optimization submodel is structurally independent of the simulation process, requiring only the specification of and periodic updating to observations of the system objective function. Thus, its interface with other type of models can be accomplished in a straightforward manner.

The remainder of the paper includes a brief description of the two submodels, validation studies of the simulation submodel, a definition and demonstration of the integrated model applications, and conclusions of the major findings along with considerations for potential model applications.

MODEL DESCRIPTION

Simulation Submodel

The simulation model developed in this study is coded in FORTRAN and uses the powerful capabilities of the SLAM II simulation language developed by Pritsker (*13*). The simulated freeway segment is represented in the simulation model by a system of finite length and width. The first point on the segment corresponds to the system entry point; the end of the freeway segment is represented by the system exit point. The model is designed to handle freeway sections of up to four lanes in each direction, including an entrance and exit ramp. The overall logic of the simulation model is shown in the flowchart in Figure 1. In this paper, only model attributes that differ from previous models are discussed.

Lane Assignment at Entry Point

Vehicles are generated (at the system entry point) from one stream of traffic and subsequently assigned a lane according to one of two criteria: (a) the lane that allows the highest entry speed or (b) a prespecified lane distribution (in which lane distribution must be specified by the user) at the system entry point. The results presented in this paper are based on the first criterion of lane assignment.

Desired Speed

A different desired speed definition was introduced that assumes the driver's desired speed to be somewhat dependent on traffic conditions, as opposed to the commonly used free flow speed.

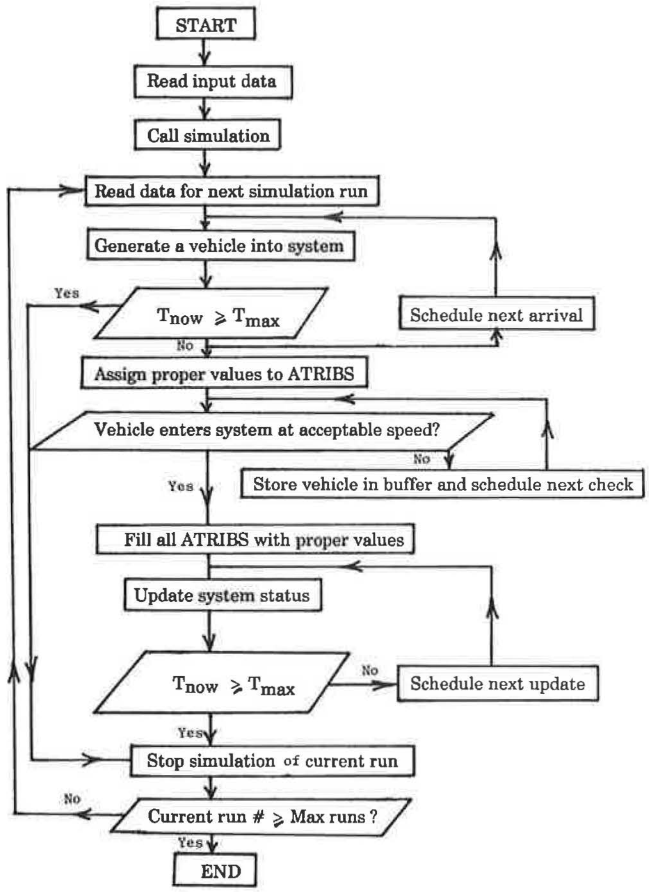

FIGURE 1 Overall simulation model logic.

The use of this new definition in the simulation has many advantages in terms of modeling convenience and quality of simulation results.

Vehicle Updating

The updating procedure was accomplished in a unique manner. Rather than updating all vehicles in the system at periodic intervals, only one vehicle is updated at a time at interval Δt (depending on the individual driver's reaction time τ). The main objective was to reduce the execution time to render the model feasible for optimization. While a vehicle's attributes are being updated, vehicles in the vicinity of that vehicle are accounted for. The model guarantees that all vehicles in the system are updated within 0.5 sec, which is very reasonable compared with 1.0-sec fixed-step sizes used in previous studies (*5, 14*) and to the 0.1 sec recommended in another study (*15*). The use of large updating step sizes without affecting the behavior of the model was achieved by using the proper car-following model. In this updating routine, all statistics are collected at both ends of the system as well as at different stations along the freeway segment.

Car-Following Model

The literature abounds with car-following models designed to predict the behavior of drivers in platoons (*3, 16, 17*). A review of the existing models revealed many drawbacks to their applications for work zone analysis. A new car-following model was developed in a manner suggested in the work by Gipps (*16*) with some modifications to make it amenable for use at lane closures. For example, in the original model Gipps assumes that 50 percent of the driver's reaction time is used as a safety margin to account for possible delays by the following driver in reacting to the lead driver's actions (hereafter termed θ). Thus, the distance $0.5\tau V$ is taken as the safety margin. In this study, the parameter θ is considered to vary depending on the traffic volume. A minimum distance was set in the model to guarantee adequate safety margins at the low speeds that are often experienced at construction zones. Unlike other car-following models, the outcome from this model is the maximum speed that can be achieved by the following driver after one reaction time, τ. Thus, an estimate of the required uniform acceleration or deceleration rate that can be applied by the following driver over the next time period τ is readily determined.

Driver's Critical Gap

The driver's gap is very critical in making lane-changing decisions and has been the subject of many studies (*18–20*). The driver's critical gap is stochastically assigned according to a probability distribution function that incorporates the following features:

- Drivers with higher desired speeds accept shorter gaps. This feature was previously used in a study by Rathi (*21*) of freeway lane closures.
- Drivers in closed lanes accept shorter gaps as they move toward the taper. This concept of nonstationary gap acceptance was originally developed by Abella (*22*).
- Critical gap for drivers in closed lanes decreases in value with an increase in gap searching time. This concept was reported in studies (*23, 24*) of left-turn traffic at intersections. In these studies, the driver's critical gap decreased with each rejected gap (i.e., waiting time for the adequate gap).

Lane Change Procedure

The lane change procedure is one of the essential components of the simulation model. Many variables and features have been included in the model under this element to mimic real-life lane change behavior. Two types of lane changes have been used in the model as defined in the INTRAS (*3*) model—nonessential and essential lane changes. Nonessential merges occur from one lane to another at any time and location over the highway segment. Some constraints, however, have been set to limit these merges to reasonable frequencies. On the other hand, essential merges are those to be made from closed lanes, and, as initiated in the model, the freeway section upstream of the construction taper is divided into N segments, each of length L_i (see Figure 2). The last segment ends at the beginning of the first taper. As mentioned earlier, traffic upstream of the first advance warning sign follows a prespecified lane distribution model. Let P_i denote the ratio of Segment i closed-lane traffic (that has not attempted the merge) to the total closed-lane traffic entering Segment 1. Thus, P_i is always 1, whereas P_{N+1} is 0. On the basis of this definition, these P's are subjected to a set of constraints in which $P_i \geq P_{i+1}$. For the given ratios of P_i and P_{i+1} at the beginning and the end of Segment i, the percentage of closed-lane traffic that would initiate the merge over that segment can be determined. For instance, if $P_2 = 0.80$ and $P_1 = 0.55$, then 20 percent and 25 percent of all closed-lane traffic (entering Segment 1) would initiate the lane change on Segments 1 and 2, respectively. It should be emphasized that these P_i's can be either input or, as discussed later, optimized on the basis of a given system measure of performance. In case of multiple lane closures, the same procedure has been applied with the exception that drivers in the outer closed lane attempt to perform the second merge immediately after they complete their first merge in a closed lane.

Having initiated the merge, the driver starts an attempt to move to an adjacent lane. The model first identifies the adjacent lane into which the driver is attempting to merge. Lane identification is based on the type of lane changing and the characteristics of gaps in adjacent streams. The model also identifies the effective gap in the target lane into which the driver will be merging. This gap is bounded by two vehicles: effective lead and lag vehicles. The effective lead or lag vehicle need not currently be in the target lane; therefore, the effective gap could be less than the apparent gap (see Figure 3).

Drivers involved in essential merges are assumed to be more alert and therefore can accept shorter gaps. Thus, in the merging process, the reaction time might be shorter than the normal reaction time under routine driving conditions. However, the model checks first to see if the gap is acceptable with no temporary reduction in reaction time for both merging and lag drivers. If the test fails, a second trial is made to see whether the driver will accept the gap with some temporary reduction in reaction time. This reduction may explain the shorter gaps observed in the field that would theoretically be unsafe under normal driving conditions. Interestingly, this concept was implemented indirectly in the INTRAS model

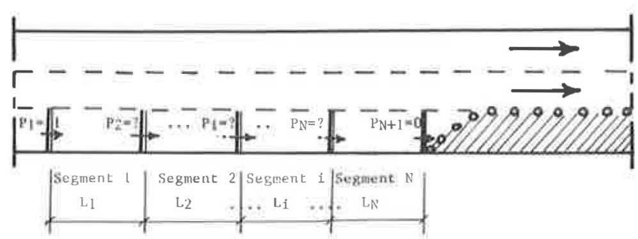

FIGURE 2 Segment definition for development of merge strategy.

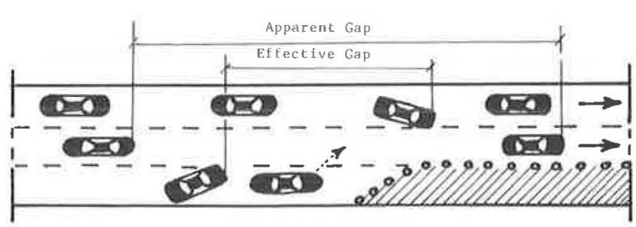

FIGURE 3 Apparent versus effective gap size.

(*3*) by allowing a temporary unsafe position of the merging vehicle during the finite period of the lane change. This allowance in INTRAS was set to enable the representation of forced lane changing, with a vehicle crowding into what might normally be considered an unavailable gap. The assumption of alertness for essential merges was also applied on a limited basis to some nonessential merges in the open lanes if the actual speed falls below a certain percentage of the driver's desired speed. While repeating this procedure, the drivers consider further actions, especially when the check fails to produce an acceptable gap in several trials. In this case the following options are pursued in the given order:

1. Accelerate if vehicle is not in the proper position to use the adjacent gap,
2. Decelerate if vehicle is not in the proper position to use the adjacent gap,
3. Accelerate to use the gap ahead of the adjacent gap, or
4. Decelerate to use the gap behind the adjacent gap.

Optimization Submodel

The optimization algorithm (SAMOPT) used in this study was developed for optimizing the response function of simulation models with stochastic behavior. Because of the stochastic nature of the simulation systems, the result of each evaluation by simulation is only a noisy observation of the true response. The algorithm uses these noisy responses to select values of the decision variables of the system such that the true response is optimized. Principles of stochastic approximation techniques have been used in developing this algorithm, which guarantees convergence to the optimum if a large number of observations are made. Even with limited sample sizes, the algorithm will yield reasonably accurate answers. The algorithm was further enhanced to incorporate decision variables that are subject to a set of linear constraints (*12*, *25*). Additional details about the source code and the validation of the algorithm can be found elsewhere (*25*).

In summary, the advantages of the optimization procedure are as follows:

- It can be interfaced with objective functions and decision variables obtained from microscopic simulation models;
- It can handle up to 10 decision variables (even more with some manipulations in the array dimensions);
- It accepts linear constraints on the values of decision variables; and
- It guarantees an optimum solution within a finite number of simulation runs; with a limited number of runs, the procedure still yields reasonable solutions.

On the other hand, working experience with the algorithm revealed some drawbacks:

- The procedure uses the number of simulation runs between $2 \cdot (2^n + 1)$ and $(N/2)$, where n is the number of the decision variables to be optimized and N is the maximum number of simulation runs available, to locate an initial point before applying the stochastic approximation technique. As many as half of all simulation runs are expended in finding that initial point. The procedure does not have the flexibility to use an initial point input by the user.
- Although economical in its overall use of simulation runs, the procedure performs only two simulation runs at each tested point in the initial runs, which may not be sufficient to distinguish noise from trend.
- Tolerance levels for the decision variables (which represent the convergence criteria) are dimensionless and must be small in magnitude regardless of the dimension of the decision variables.

Interfacing the Simulation and Optimization Submodels

Interfacing the submodels requires that one of the two models serves as a subroutine for the other. For example, if the optimization routine SAMOPT is to be used with N simulation runs, it first determines the values of the decision variables at the current run and calls the simulation submodel to pass the objective function on the basis of these decision variables. It then determines the decision variables to be used in the next simulation run, and so on. On the other hand, if the simulation submodel is to call SAMOPT, then it must retrieve from SAMOPT, on the basis of the current objective function, the values of the decision variables to be used in the next run, and so forth. The latter strategy was applied in this study. The interfacing logic of both submodels is shown in the flowchart in Figure 4.

In addition to the advantage of running both submodels interactively, some enhancements were performed on the optimization algorithm:

- An initial point can now be input by the user. This feature saves significant computer time, especially when the user has good information about a feasible initial point.
- In conjunction with the identification of an initial point, several system performance measures can be stored and used subsequently for optimization purposes.
- The optimization process usually requires a large number of simulation runs (typically about 80 runs for this application). With the enhancements, these runs may be divided into several small jobs rather than done in one job. Although this feature will not save CPU time, it allows performing the optimization at the user's convenience. This division is sometimes essential when there is a limitation on the maximum CPU time per job or in ranking jobs for execution.
- To economize on computer resources, a variable simulation time was employed. Longer simulations are performed in the process of determining the initial points as well as in analyzing the optimal solution.

MODEL VALIDATION

Because the optimization submodel was validated previously (*25*), this task is limited to the simulation submodel. Field data were collected from several freeway sites in the Chicago metropolitan area for validation purposes.

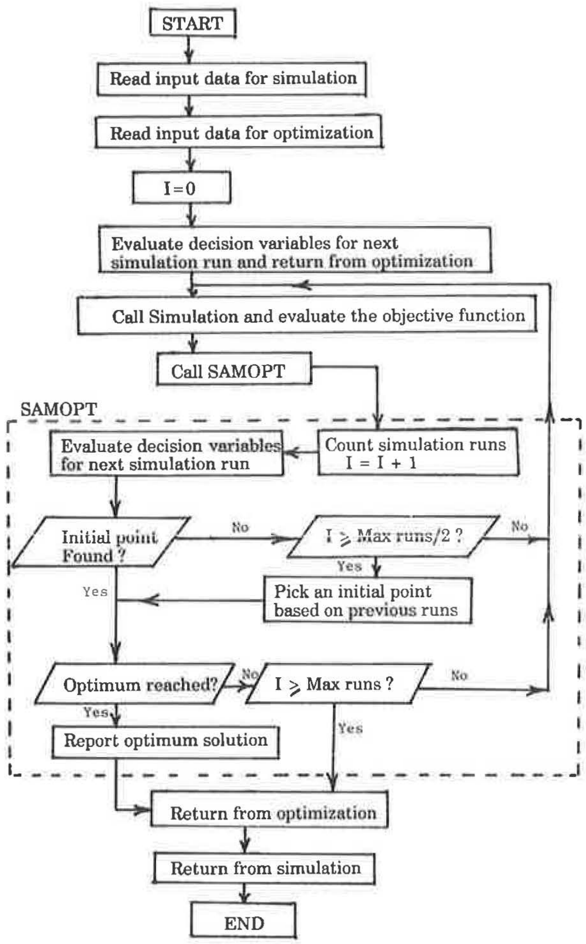

FIGURE 4 Interfacing of simulation model with the optimization routine.

Data Collection

Field Data Collection

Data were collected using two sets of video records. Each set consisted of a video camera with time feature and a portable VCR. The data incorporated most of the lane closure configurations (left versus right closure, single versus multiple closures). Filming was concentrated (when possible) at five key locations: (a) entry point (farthest point upstream of the taper where traffic is free flowing), (b) upstream of the taper (about 500 ft), (c) at taper, (d) downstream of the taper (about 500 ft), and (e) at exit point. Data collected included speeds, headways, and lane distribution of traffic at the key locations.

Data Reduction

Data were reduced in the laboratory using standard video reduction techniques. To ensure the synchronization of the data collected at both stations, individual vehicles were traced in both films; in most cases, it was possible to identify the same traffic at different locations. Headways and speeds were best fitted by lognormal and normal distributions, respectively. Moreover, a freeway lane at the bottleneck section carried up to 1,900 to 2,000 veh/hr. Furthermore, no significant reduction in speed due to construction was found when the demand volume was less than 1,700 to 1,800 veh/hr per lane in the bottleneck section. At higher rates, system breakdown (characterized by large speed variations and frequent stops) occurred in the taper area.

Validation Scenarios

Basic Segment

The model was first validated as a basic freeway segment. The parameters tested under this case were the car-following parameter θ and desired speed V_d. Several simulation runs were made for different combinations of V_d, θ, and flow levels. Lane capacity and speed obtained from each simulation experiment were compared with the corresponding values in the 1985 Highway Capacity Manual (HCM) (*26*) and with field observations. The calibrated car-following model and desired speed parameters (as a function of the traffic volume) are presented in Table 1. As indicated, the safety factor is inversely proportional to the flow rate. This relation may be explained by the fact that drivers are more sensitive to the gap length, expressed in terms of distance rather than time. Thus, if a safety distance is to be considered under all conditions, this distance (expressed in time) is shorter under low-volume conditions (because of higher speeds) than under heavy-volume conditions.

Space mean speeds from the calibrated model in comparison with field data as well as with the 1985 HCM values are shown in Figure 5. There was no significant drop in speed when the traffic volume was less than 1,600 veh/hr per lane; beyond this level, average speeds decreased substantially. Figure 5 also shows that the simulation model behaves consistently compared with the field data and the 1985 HCM. Hence, it was concluded that the model is a valid representation of basic freeway segments and ready for further testing with lane closures.

Lane Closures

A significant number of parameters were introduced into the model to account for the behavior of individual drivers at construction zones. The model parameters were first verified through an extensive sensitivity analysis on the basis of 90 simulation runs. The sensitivity analysis was performed using the more economical strategy 2^k factorial design (*27*). Subsequently, all model parameters were fixed except for the vector P_i, which was either provided as input or determined through optimization procedures, as discussed later in this paper.

For illustration purposes, the model results are compared herein with observations taken at two-lane closure sites in the Chicago area. A sample size of 300 observations was collected from the simulations for each site. Statistical testing consisted of a series of t-tests on the difference in mean speeds and headways (observed versus simulated) and the Kolmogorov-Smirnov test (*15*) for the lane distribution of traffic.

TABLE 1 CALIBRATION OF MODEL PARAMETERS WITH NO LANE CLOSURES

Parameter	Flow level (vphpl)								
	≤600	800	1000	1200	1400	1600	1800	1900	≥2000
θ (%)	10	10	10	10	10	10	10	20	40
V_d (mph)	60	58	57	56	55	53	48	43	35

θ Car-following model parameter.

V_d Driver's desired speed.

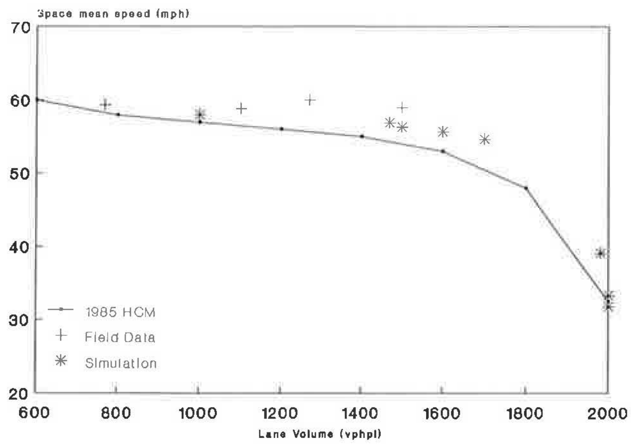

FIGURE 5 Space mean speed versus lane volume (without closures).

Figure 6a shows the layout of the first site on the Edens Expressway where construction took place on the right lane of the three-lane segment. Field data collected at that site and corresponding results from the simulation model are presented in Tables 2 and 3, respectively. Because information about the pattern of merge attempts or actual merges from the closed lane was difficult to collect in the field, the results from simulation runs were based on an experimental merge pattern that best fit the field observations.

The results in Tables 2 and 3 show an excellent statistical agreement between field and simulation data in terms of speed, flow, and proportion of flow in each lane. For instance, the average observed and simulated exit speeds were 58.5 and 58 mph, respectively. Statistically, these observations were not significantly different at the 5 percent significance level.

The second site was located on the I-88 Expressway where construction took place on the right lane of a three-lane freeway segment as shown in Figure 6b. Field data are presented in Table 4 and the corresponding simulation results are summarized in Table 5. The simulation model was again able to reproduce results that were quite comparable with its field counterparts.

The statistical agreement between field and simulation data consistently observed in most of the sites studied (including the two sites covered here) indicates that the microscopic simulation model is a valid representation of the lane closure traffic system.

MODEL APPLICATIONS

The microscopic simulation model developed in the course of this study has two significant applications:

1. Evaluation of existing traffic systems—to be done by providing the model with the proper input parameters as observed in the field and comparing field performance with optimum performance from the model.
2. Design of traffic systems—to be done by interfacing the simulation model with the optimization algorithm SAMOPT, to optimize the traffic system performance.

In this paper, the model is focused on the latter application. In a freeway lane closure system, the distribution of merges along the zone has a profound impact on system performance. A commonly held view among traffic engineers is that early merges are beneficial to traffic operations at work zones, because few vehicles are likely to be stranded at the taper. However, this strategy may result in evacuating the closed lane further upstream than is actually warranted, thus adding to the congestion on the through lanes. Obviously, the major drawback of a late merging strategy is the likelihood of conflicts occurring at the taper due to the high speed difference between vehicles in the through lanes and those attempting to merge.

Without a formal assessment of what constitutes an optimum merging strategy under various flow levels, it is difficult to justify a static placement of advance warning signs that are

TABLE 2 FLOW CHARACTERISTICS AT SITE 1 (EDENS EXPRESSWAY AT LAKE STREET, THREE LANES, RIGHT LANE CLOSED)

Station	Item	Lane[a] 1	Lane[a] 2	Lane[a] 3	Total/ Average	95% Conf. Interval
STATION I						
	Traffic volume (vph)	1224	1464	732	3420[b]	
	Speed (mph)	63.3	60.9	57.2	61.0[c]	60.2-61.7
	Lane distribution (%)	0.36	0.43	0.21		
	Cumulative lane dist.	0.36	0.79	1.00		
STATION II						
	Traffic volume (vph)	1800	1644	192	3636[d]	
	Speed (mph)	60.6	56.4	56.4	58.5	57.7-59.3
	Lane distribution (%)	0.49	0.45	0.05		
	Cumulative lane dist.	0.50	0.95	1.00		

[a] Lanes numbered from median to shoulder.

[b] Total flow rate on the freeway segment at station.

[c] Average speed on the freeway segment at station.

[d] Flow rate is higher than at station I due to ramp traffic.

TABLE 3 SIMULATED CHARACTERISTICS AT SITE 1 (EDENS EXPRESSWAY AT LAKE STREET, THREE LANES, RIGHT LANE CLOSED)

Station	Item	Lane 1	Lane 2	Lane 3	Total/ Average	95% Conf. Interval
STATION I						
	Traffic volume (vph)	1157	1432	853	3442	
	Speed (mph)	61.2	60.6	59.6	61.0	60.5-61.5
	Lane distribution (%)	0.34	0.42	0.25		
	Cumulative lane dist.	0.34	0.75	1.00		
	K-S difference[a]	0.02	0.03[b]	0.00		
STATION II						
	Traffic volume (vph)	1647	1801	215	3663	
	Speed (mph)	59.0	56.2	55.6	58.0	57.2-58.8
	Lane distribution (%)	0.45	0.49	0.06		
	Cumulative lane dist.	0.45	0.94	1.00		
	K-S difference	0.05[b]	0.01	0.00		

[a] Absolute difference between cumulative lane distribution (field vs. simulation).

[b] Not significant at the 5% level (max $K\text{-}S_{5\%,\,300} = 0.08$).

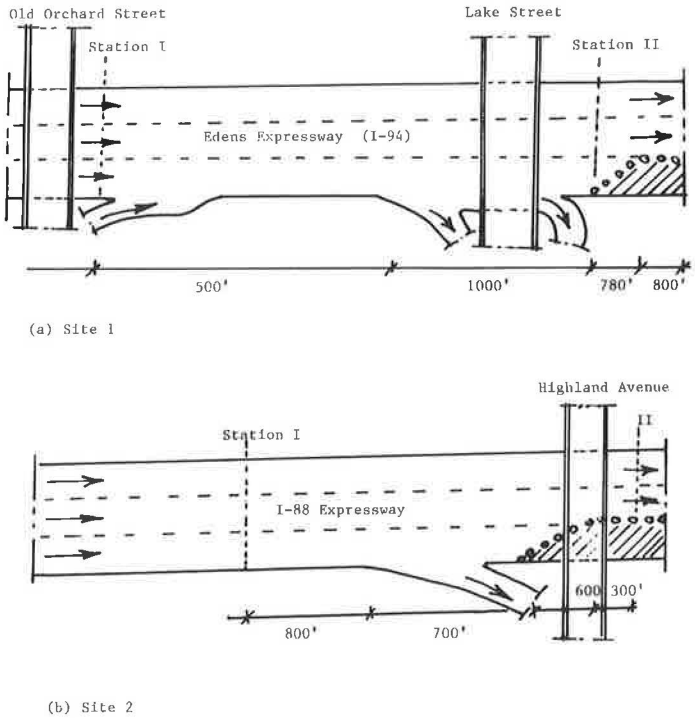

FIGURE 6 Layout of construction sites.

TABLE 4 FLOW CHARACTERISTICS AT SITE 2 (I-88 EXPRESSWAY AT HIGHLAND AVENUE, THREE LANES, RIGHT LANE CLOSED)

Station	Item	Lane 1	Lane 2	Lane 3	Total/ Average	95% Conf. Interval
STATION I						
	Traffic volume (vph)	1500	1309	957	3766	
	Speed (mph)	62.8	61.3	59.6	61.5	60.6-62.3
	Lane distribution (%)	0.40	0.35	0.25		
	Cumulative lane dist.	0.40	0.75	1.00		
STATION II						
	Traffic volume (vph)	1614	1353	-[a]	2967[b]	
	Speed (mph)	62.8	59.4	-	58.5	57.7-59.3
	Lane distribution (%)	0.54	0.46	-		
	Cumulative lane dist.	0.54	1.00	-		

[a] Lane closed in that section.

[b] Total flow rate lower than station I due to the exit ramp.

TABLE 5 SIMULATED CHARACTERISTICS AT SITE 2 (I-88 EXPRESSWAY AT HIGHLAND AVENUE, THREE LANES, RIGHT LANE CLOSED)

Station	Item	Lane 1	Lane 2	Lane 3	Total/ Average	95% Conf. Interval
STATION I						
	Traffic volume (vph)	1317	1615	816	3748	
	Speed (mph)	60.1	59.1	64.6	60.4	59.2-61.7
	Lane distributión (%)	0.35	0.43	0.22		
	Cumulative lane dist.	0.35	0.78	1.00		
	K-S difference	0.05[a]	0.04	0.00		
STATION II						
	Traffic volume (vph)	1507	1487	-[b]	2994	
	Speed (mph)	61.6	57.6	-	59.5	58.0-61.1
	Lane distribution (%)	0.50	0.50	-		
	Cumulative lane dist.	0.50	1.00	-		
	K-S difference	0.04[a]	0.00	-		

[a] Not significant at the 5% level.

[b] Lane closed in that section.

aimed at promoting an optimum strategy. This factor was considered to be the key application of the integrated model. In this paper, the results are reported for one performance measure, travel time in the work zone, with the qualification that other measures may yield entirely different optimum merging strategies. An added benefit of this exercise is the ability to use the model for evaluating field merging patterns against their derived optimal. Corrections can then be implemented in the field (i.e., in sign placement, taper length, etc.) aimed at bringing the observed merging pattern closer to its optimal.

In the simulation model, a merge pattern is established by randomly assigning each driver in the closed lanes a desired merge segment. When a driver enters a desired segment, the model automatically schedules a lane change attempt. Thus, the merge pattern is actually representative of merge attempts rather than merge executions. In any case, the model will output both distributions at the end of the simulation and optimization run. In the optimization mode, the model is essentially queried for the average percentage of drivers that should be assigned to the individual segments.

RESULTS

Before the optimization process was carried out, sensitivity runs were performed to determine whether merging strategies had a significant effect on the proposed system measure of performance, travel time. Two distinct patterns of merge initiations were studied at different flow levels. The early merge pattern assumes that all closed-lane traffic attempts to evacuate the lane within 0.7 mi from the taper. On the other hand, a late merge pattern assumes that all closed-lane traffic begins evacuating the closed lanes within 0.3 mi from the taper.

Results from this analysis are shown in Figure 7. The early merge produced results that were significantly superior to those under the late pattern at all flow levels. Furthermore, there appeared to be a range of flow over which the differences between the two merge strategies are more pronounced. This range varies from 1,500 to 2,000 veh/hr per lane. Below this range, there was no significant difference between the effect of the two merge strategies on travel time. Beyond that range, there was a breakdown in system performance with the late merge strategy, because of which a majority of closed lane traffic could not merge within the simulated time; thus the results were primarily representative of traffic in open lanes. It should be emphasized, however, that the results shown in Figure 7 do not imply that intermediate merge patterns should be expected to yield intermediate travel time values between the two extremes.

On the basis of this information, the optimization was performed within the flow range specified. Two flow rates were selected, 1,700 and 1,850 veh/hr per lane, respectively. The input and output of the optimization procedure at these two flow levels are discussed in some detail in the next section.

The integrated model was applied to determine the minimum travel time for a freeway work zone segment (3 lanes

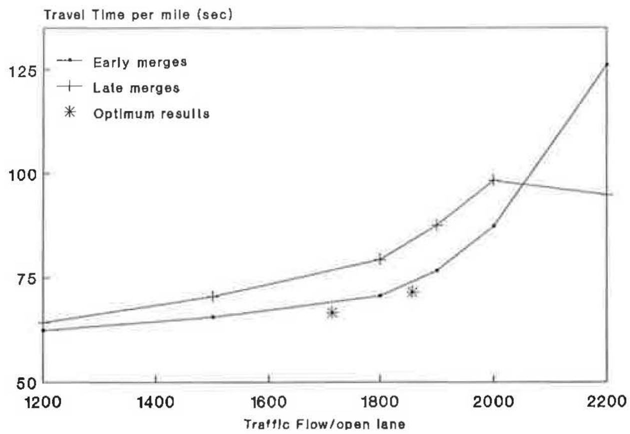

FIGURE 7 Travel time versus traffic volume.

and right lane closed) by determining the optimum merge pattern from the closed lane. In all runs (due to CPU limitation), the system was simulated for 15 min including a warm-up period of 5 min. The freeway segment upstream of the construction taper was divided into four equal segments, each ¼ mi in length. Thus, three decision variables were to be optimized (see Table 6 and Figure 2), namely the proportion of closed-lane traffic entering Segments 2, 3, and 4 to that entering Segment 1. Therefore, three decision variables and two inequality constraints were input to the model. A lower bound of 0, an upper bound of 1.0, and a tolerance of 0.025 were input for each variable. The two distinct constraints were $P_2 \geq P_3$ and $P_3 \geq P_4$. The typical maximum number of simulation and optimization runs required for flow rates of 1,700 and 1,850 veh/hr per lane were 70 and 80, respectively. A lengthy simulation run (of about 40 min) was performed after the optimal solution was reached to collect a sample of 1,500 observations. This large sample was needed to perform statistical analyses on the observations using the batch mean method (*28*). The advantage of the batch mean technique is that it can analyze data and construct confidence intervals for the system measures of performance even with the presence of correlation among the individual observations. With that in mind, performing one lengthy simulation run was more economical and beneficial than running several independent replications, each for a short period. The reason is that in the latter method a warm-up period for each replication is required during which no data are collected. In addition, with short simulation runs, the system may never reach a steady state condition, in which case the measure of performance obtained by averaging over a number of runs will not be representative of the system performance at the steady state condition.

Common output parameters of the optimization procedure are presented in Table 6. The optimum travel time is plotted on Figure 7 for both flow levels analyzed. As indicated in Table 6, exactly one-half of the maximum simulation runs specified for each flow rate were expended in determining the initial point. The optimum point under both flow levels was closer to the initial point, which implies that the model was quite successful in finding the initial point. The results also show that as the flow rate increased, a late merging strategy seemed to be more appropriate in terms of minimizing the travel time. Furthermore, as seen from Table 6 and Figure 7, the optimum (minimum) travel time resulted from neither the early nor late merging strategies. However, the results obtained from the early merge strategy were much closer to the optimum than those from the late merge strategy. Cumulative distributions of the optimum distribution of attempted and completed merges over the segments upstream of the taper for each flow level are shown in Figure 8. The horizontal distance between the merge attempt and merge completion curves is the distance traveled while searching for an acceptable gap. This value consistently increased as traffic approached the taper. At that point, the distance decreased because drivers were forced to merge in that zone. The search distances were rather long but this is not unexpected at the near-capacity flow levels that were tested. Unexpectedly, however, the average search distance decreased as the flow rate increased, specially at locations near the taper or further upstream of the taper. This observation does not necessarily mean that the

TABLE 6 OPTIMIZATION RESULTS FOR TWO FLOW RATES

Parameter	1700 vphpl		1850 vphpl	
	Initial	Optimal	Initial	Optimal
No. of simulation runs used	35	66	40	77
Optimal travel time (sec/mile)	66.5	66.5	72.0	71.5
Decision variable, P_2	0.44	0.46	0.94	1.00
Decision variable, p_3	0.19	0.19	0.31	0.34
Decision variable, p_4	0.06	0.02	0.19	0.25

(Note: P_i is the proportion of closed lane traffic that has not attempted the merge upstream of segment i).

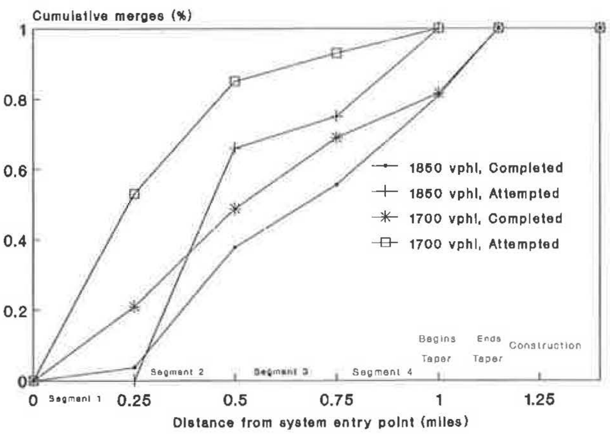

FIGURE 8 Recommended merge strategies.

search time decreased with an increase in flow rate but may simply be the consequence of speed reductions in both traffic streams. The figure also shows that as the flow level increased, completed merges occurred much further downstream (almost no merges occurred in Segment 1 with the 1,850 veh/hr per lane scenario). In addition, the figure shows that, under the optimal solution, up to 18 percent of closed-lane traffic, under both flow levels, merged along the construction taper. Interestingly, field observations at that flow rate revealed about the same percentage.

CONCLUSIONS

Because the work discussed in this paper is a first attempt at optimizing the performance of microscopic traffic systems, it has focused on the major optimization concepts within the integrated model. However, valuable conclusions can be drawn from the work zone applications of the model. These conclusions are limited to the specific lane closure configurations studied in this paper and should not be generalized without further validation effort.

- The integrated microscopic model can be used in optimizing freeway work zone traffic systems at a reasonable computational cost.
- The optimization model SAMOPT was enhanced in this study and consequently its efficiency in using computer CPU time has been considerably improved.
- The merge strategy from closed lanes at work zones had a significant impact on the system performance measured by travel time; this impact was more pronounced at flow rates ranging between 1,500 and 2,000 veh/hr per lane.
- The optimum merge strategy that resulted in minimum average travel time over the work zone was neither an early nor a late merge pattern.
- As the flow level increased, the optimum strategy recommends that the first attempt to merge from the closed lane be made further downstream compared to that for lighter flow rate.
- The model can be applied to designing other work zone elements such as the determination of the optimum length of taper, ramp work zone controls, and location of advance warning devices.

IMPLEMENTATION CONSIDERATIONS

Model Utilization

With the capability of the model to formally optimize a traffic system, the model can be applied to

- Determining an optimal merge strategy at freeway lane closures. This is an important step toward optimizing the traffic control plan in the field and ultimately providing the proper traffic control device to promote such merge pattern. This work, however, was beyond the scope of this study.
- Determining the optimum length of the construction taper as well as lengths of speed change lanes.
- Determining whether speed reduction is advisable at site for certain ranges of traffic volumes.

Model Limitations

Although the model applications are numerous, users must be aware of its underlying assumptions pertaining to the driver's critical gap and reaction time. Therefore, when field observations appear not to match the general scope of such assumptions, the results must be viewed with great caution.

RECOMMENDATIONS FOR FURTHER WORK

Following are some recommendations for future work in this area:

1. Broadening the optimization work to include different lane closure configurations (left versus right, single versus multiple closures, construction near ramps, etc.) to generalize the findings in this paper.
2. Interfacing the optimization algorithm with other existing traffic models to study and optimize other traffic problems (such as NETSIM model for intersections).
3. Testing the optimization of traffic systems by using other objective functions (such as acceleration noise) to see whether they yield the same optimum solution.
4. Investigating the use of multi-objective or utility functions.

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Crash Tests of Work Zone Traffic Control Devices

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Full-scale vehicle crash tests were used to evaluate performance of typical work zone traffic control devices. Modified test procedures and evaluation criteria from National Cooperative Highway Research Program Report 230 were used in 108 tests, providing significant insight into impact performance. Plastic drums used as channelizing devices, cones, tubes, and vertical panels performed well in most tests, presenting no hazards in terms of passenger compartment intrusion, interference with vehicle control, or threat to workers and other traffic from impact debris. Various nonstandard forms of ballast placed on top of or inside channelizing devices detracted from performance, and sometimes posed a severe threat to test vehicle occupants, workers, and other traffic. Similarly, impact debris formed in several tests on Type I and III barricades and portable signs and supports posed a threat, and was often thrown long distances through work zones. Warning lights attached to traffic control devices were also thrown free in a number of tests, and appeared to threaten workers and other traffic.

With increased emphasis on repairing and rehabilitating the existing infrastructure, work zones have become commonplace on the nation's highways. A variety of signs, channelizing devices, and other traffic control devices (TCDs) guide and control traffic in these zones. Primarily, TCDs convey information to the motorist (*1*), and a number of recent studies have developed a wide array of work zone TCDs that effectively accomplish this purpose (*2*, *3*). However, work zone TCDs must also fulfill an important secondary function. Work zone traffic accidents are common occurrences, and TCDs are often involved because of their close proximity to the traveled lanes. Thus, in addition to transferring information, they must perform safely when impacted by errant vehicles.

Performance criteria for permanent highway safety appurtenances such as traffic barriers and sign supports have existed for some time (*4*), and also apply to temporary work zone safety devices such as portable traffic barriers. Considerable research (*5*, *6*) on traffic barriers and related features for highway work zones has provided information on their performance. However, only limited published data describe impact performance of work zone TCDs (*7*), and no performance criteria have been proposed or accepted for widespread use.

This paper describes a 1988 study by the New York State Department of Transportation (NYSDOT) that evaluates impact performance of TCDs commonly encountered in New York work zones. Test procedures were modified to deal specifically with the TCD types tested, and performance criteria were developed to evaluate the results. In all, 108 full-scale tests were conducted on 62 different combinations of TCDs and installation conditions.

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TEST PROCEDURES AND EVALUATION CRITERIA

The tests were performed during the summer of 1988 at the Department's Highway Safety Test Center in Scotia, New York, near Albany. All tests were recorded by two electronic video camera-recorders and one 35-mm movie camera. In addition, still photographs documented the devices tested and the results. Because the size and weight of the devices were small, significant vehicle deceleration on impact was not expected. To simplify procedures and permit completion of a greater number of tests within the time available, accelerometers were not installed in the vehicles. After each test, damage to the vehicle and test device was noted. Particular attention was given to any tendency of the devices to penetrate the passenger compartment or to cause windshield damage. Postimpact location of the test device was noted, as was all debris formed by the impact.

Two categories of test vehicles were used—1,800-lb Honda front-wheel-drive sedans and full-size rear-wheel-drive sedans of various makes and models, each weighing about 4,500 lb. Windshield conditions were recorded in detail before and after impact to assess actual damage from each test. Test speeds varied from 20 to 60 mph, representing the range of speeds typically encountered in work zones.

NCHRP Report 230 (*4*) presents safety evaluation guidelines for crash tests involving highway safety appurtenances. Those evaluation factors include structural adequacy of the device tested, risk of injury to vehicle occupants, and postimpact trajectory of the test vehicle. Traffic control devices evaluated in this study, however, serve a different function. In addition to differences in structural capacity and intended function, the environment in which work zone TCDs are used varies considerably from that of typical permanent safety appurtenances. Considering the evaluation factors in NCHRP Report 230 (*4*), as well as the intended function of work zone TCDs and the environment in which they must function, three specific criteria were developed for evaluating these tests:

1. Passenger Compartment Intrusion—Intrusion into the vehicle by any debris from the test device caused by the impact was considered unacceptable because it greatly increases risk of injury to its occupants.

2. Loss of Vehicle Control—Because work zones often provide restricted operating space for vehicles, and numerous hazards are frequently located close to the designated travel lanes, interference with driver control of the vehicle resulting from a TCD impact is considered unacceptable. Loss of control may occur in four ways: (a) physical interference by the test device with vehicle steering and braking; (b) windshield damage restricting driver visibility or startling the driver so that vehicle control is lost; (c) debris thrown into opposing traffic lanes appearing hazardous to an oncoming driver, causing emergency evasive action, leading to loss of control and a secondary collision; (d) sand or other debris scattered on the pavement leading to loss of control of other vehicles, especially motorcycles.

3. Physical Threat to Workers or Other Vehicles—Because of close proximity of construction workers and other traffic to the TCDs, devices or fragments thrown by an impact may present a hazard. Size, shape, weight, composition, and distribution of debris were recorded for each test and evaluated to determine whether the debris constituted a hazard.

Following completion of the test program, each test was rated according to these three criteria. In addition, cosmetic damage to the vehicle and TCD damage were noted because each represents a cost factor. Rating factors for each criterion are presented in Table 1.

TRAFFIC CONTROL DEVICES AND TEST PARAMETERS

Six types of traffic control device were tested—steel channelizing drums, plastic channelizing drums, temporary traffic signs and supports, Type I and III barricades, and miscellaneous small channelizing devices. In addition, a variety of ballast procedures, warning lights, and other parameters were

TABLE 1 SUMMARY OF EVALUATION FACTORS

PASSENGER COMPARTMENT INTRUSION

1. Windshield Intrusion
   - a. No windshield contact
   - b. Windshield contact, no damage
   - c. Windshield contact, no intrusion
   - d. Device embedded in windshield, no significant intrusion
   - e. Partial intrusion into passenger compartment
   - f. Complete intrusion into passenger compartment
2. Body Panel Intrusion (yes or no)

LOSS OF VEHICLE CONTROL

1. Physical Loss of Control
2. Loss of Windshield Visibility
3. Perceived Threat to Other Vehicles From Debris
4. Debris on Pavement

PHYSICAL THREAT TO WORKERS OR OTHER VEHICLES

Harmful Debris (yes or no)

VEHICLE AND DEVICE CONDITION

1. Vehicle Damage
   - a. None
   - b. Minor scrapes, scratches, or dents
   - c. Significant cosmetic dents
   - d. Major dents to grill and body panels
   - e. Major structural damage
2. Windshield Damage
   - a. None
   - b. Minor chip or crack
   - c. Broken, no interference with visibility
   - d. Broken and shattered, visibility restricted but remained intact
   - e. Shattered, remained intact but partially dislodged
   - f. Large portion removed
   - g. Completely removed
3. Device Damage
   - a. None
   - b. Superficial
   - c. Substantial, but can be straightened
   - d. Substantial, replacement parts needed for repair
   - e. Cannot be repaired

tested. The TCDs and other parameters discussed in the following sections are presented in Table 2.

Steel Channelizing Drums

Steel 55-gal drums, once widely used as channelizing devices in highway work zones, are no longer permitted on New York projects. Five empty drums weighing 50 to 55 lb were tested, all with closed tops.

Plastic Channelizing Drums

Twelve different models from six manufacturers or suppliers were tested, including five specific types. Two-piece drums with detachable bases had a closed top and open bottom, and snap over a low base unit. Ballast may be placed inside on the base. On impact, the two pieces separate, with the ballast and base intended to stay near the impact point. One type of one-piece drum used had a closed top and open bottom, with external tabs provided at the bottom for ballast. Another had a closed top and bottom, but the base was slotted to form radial fingers. The drum is inverted, ballast is inserted through the slotted fingers, and the drum is then placed with the slots down. Drums were also tested with an open top, with the ballast simply placed inside on the base. They may be specifically designed for use in this manner, or two-piece units may be purchased without a base and inverted for use. The final drum consisted of upper and lower pieces separated at about midheight. Ballast may be placed inside, and the top is fitted over the bottom to form a closed unit.

Temporary Sign Support

Six different types of supports were tested. A 12-lb steel tripod support can accommodate sign panels up to 48 in. square, either rectangular or diamond shaped. A fixed wood support included a nominal 4- by 4-in. by 16-ft wood post imbedded 4 ft in the ground, and stiffened by 2- by 4-in. by 8-ft diagonal braces. These braces were attached to stakes driven into the ground and to the post 6 ft above the ground. The support was tested with the longitudinal brace facing both toward and away from the impact vehicle. Height from the ground to the bottom of the 4- by 4-ft by ⅝-in. plywood diamond sign panel was 7 ft. Tall portable wood supports were constructed from 2- by 4-in. and 2- by 6-in. wood elements. Base dimensions were 3- by 4-ft for one support and 27 in. by 5 ft for the other. Two 2- by 4-in. vertical supports were stiffened by one lateral and two longitudinal 2- by 4-in. diagonal braces. Tests were conducted with these braces facing both toward and away from traffic. A 4- by 4-ft rectangular plywood panel 52 in. above ground was included. Both were ballasted using two 50-lb sandbags. A low portable wood support constructed from 2- by 4-in. wood elements was tested with diagonal braces facing both toward and away from the impacting vehicle. Base dimensions were 3 by 4 ft. A 4-ft-wide by 3-ft-high plywood sign panel was mounted 12 in. above the pavement. Ballast was provided by two 50-lb sandbags. A proprietary steel support had four horizontal legs in an X pattern and an adjustable steel vertical support attached to the legs through a spring mechanism. A 4-ft diamond aluminum sign panel was mounted about 4 ft above ground. Ballast was provided by four 50-lb sandbags, one on each leg. A generic design developed by the New York State Thruway Authority was constructed using

TABLE 2 SUMMARY OF TRAFFIC CONTROL DEVICES AND TEST PARAMETERS

Device and Description	Device and Description
STEEL DRUM (55 gal)	BALLAST
PLASTIC DRUM	Sandbag internal
Two-piece, detachable base	Sandbag external
One-piece, base tab	Sandbag on top
One-piece, base fingers	Sandbag suspended
Open top	Water
Two-piece, split middle	Gravel
SIGN SUPPORTS	Concrete block on top
Steel tripod	Miscellaneous material
Low wood, portable	Does not apply, none
Tall wood, portable	WARNING LIGHTS
Tall wood, fixed	Light not attached
Metal, miscellaneous	Light attached with bolt
BARRICADES, TYPE I	Light attached with bolt and washer
8-ft wood, plastic legs	Light attached with bolt and cable
2-ft plywood, metal legs	No light
3-ft metal, metal legs	
5-ft metal, metal legs	
BARRICADES, TYPE III	
Wood	
PVC	
Metal	
MISCELLANEOUS CHANNELIZING DEVICES	
Cones	
Tube	
Vertical panels	

1½ in. and 1¾ in. square perforated steel tube. Two 1¾ in. square, 5-ft-long legs with vertical stubs supported two 1½ in. square, vertical supports that slipped into the base stubs. Two transverse braces connected the vertical supports. A 4- by 5-ft plywood panel was mounted 6 ft above the pavement. Four 50-lb sandbags (one on the end of each leg) provided ballast.

Type I Barricades

Four different models of Type I barricades were tested. The smallest was a 2-ft plywood and metal barricade fabricated from steel angle legs and 2-ft by 6-in. plywood panels and lateral braces, weighing 19.3 lb. A 3-ft metal barricade included round tubular-steel legs and a 3-ft by 6-in. sheet metal panel weighing 18.2 lb. A 5-ft metal barricade consisted of square tubular steel legs and a 5-ft by 8-in. sheet metal panel. Its weight was 31.8 lb. The largest Type I barricade included a 2- by 8-in. by 8-ft wood panel and molded plastic legs, weighing 29.9 lb.

Type III Barricades

Four models were tested. The first was constructed from 2- by 6-in. wood elements, and was 4 ft wide by 5 ft high. It had three panels and weighed 60 lb. Because of its weight, no extra ballast was used. Two variations of polyvinylchloride (PVC) plastic were tested, each 4 ft wide by 5 ft high. These are shown on NYSDOT Standard Sheet 619–4R1 as Alternates A and B. Alternate A had glued joints, while Alternate B was not glued but included an external tie wire to provide stability, plus an internal rope to retain debris on impact. Both were constructed using 3-in.-diameter pipe meeting ASTM D 2665 standards. A metal unit was constructed from 1½ in. square 12-gauge perforated steel tube, and was also 4 ft wide by 5 ft high. The panels were lightweight aluminum, of total weight 57 lb. This device included hinges attaching the vertical members to the base, and was intended to fold down on impact. No ballast was used.

Miscellaneous Channelizing Devices

These devices included cones, a tubular marker, and vertical panels. Three cone types were tested. Two were one-piece cones fabricated from flexible plastic 34.5 and 36 in. high. The third was a rigid plastic two-piece cone, 36.5 in. high. The detachable base can be filled with sand or water for ballast and slipped over the cone body. All three cones weighed about 11 lb each. A 42-in.-high, plastic, two-piece tubular marker weighed 13 lb and included a heavy plastic base for stability. Two vertical panels were tested; one was a 6- by 36-in. plastic panel mounted on a fiberglass vertical support. It was attached to a 16-in.-square steel base plate to provide ballast, and weighed 33 lb. The other was an 8- by 24-in. plastic panel mounted on a nylon support, and attached to a 13- by 18-in. PVC plastic base. Its total weight was 22.5 lb.

Ballast

Eight different methods of ballast, plus unballasted TCDs, were included in these tests. A single sandbag weighing 50 lb was the standard ballast device for these tests. This sandbag consisted of dry gravel inside a reinforced polypropylene sample bag closed with packing twine. For the channelizing drums, a single sandbag was placed inside on the base, externally on the ballast tab at ground level, or on top of the drum. For sign supports and barricades, up to four sandbags were placed on the base supports, depending on the number required to provide stability against overturning from wind loads. For one drum test, a 30-lb sandbag was suspended inside the drum, hung from the top by a cable. Two traffic cones were tested with suspended 8-lb sandbags. One inverted drum was filled halfway with water weighing about 150 lb. One open-top drum was ballasted with 180 lb of loose gravel inside. A concrete block weighing 42 lb was placed on top of plastic drums in two tests. This ballast is similar in size and weight to heavy-duty batteries sometimes placed on top of drums to power warning lights. Pieces of rock or broken concrete pavement provide similar ballast. Construction debris consisting of a broken 42-lb concrete block and 13 lb of wood scraps was placed inside one open-top plastic drum.

Warning Lights

Type A warning lights were attached to a number of devices by various means (see Table 2).

RESULTS

Table 3 presents full-scale tests in this investigation.

Steel Drums

None of the five tests on steel drums provided satisfactory results in terms of all evaluation criteria. None resulted in passenger compartment intrusion, but all five interfered in some measure with vehicle control. Two 1,800-lb cars and one 4,500-lb car rode up onto the collapsed drum, with partial or full loss of steering control in 45- and 60-mph tests. In addition, the small car nearly rolled over in the 45-mph test before coming to rest partially on the drum. In the other two tests, at 30 and 45 mph with 1,800-lb cars, the drum bounded ahead of the car, threatening injury to workers as well as loss of control by other drivers resulting from severe evasive maneuvers. Figure 1 shows an 1,800-lb vehicle riding up on a 55-gal drum in a 60-mph impact.

Plastic Drums with Sandbag Ballast

Drums ballasted with 50-lb sandbags at ground level (see Figure 2) underwent 24 tests, with satisfactory results in 18 tests. Five of the six unsatisfactory tests had drum parts flying into traffic areas with potential for causing severe evasive maneuvers. The sixth unacceptable test resulted from sandbag

TABLE 3 SUMMARY OF FULL-SCALE TEST RESULTS

Device	Total Tested	Satisfactory	Failed Evaluation Factor* Passenger Compartment Intrusion	Loss of Control	Threat of Debris
Steel Drums	5	0	0	5	2
Plastic Drums					
50-lb Sandbags	24	18	0	6	0
Unballasted	15	11**	0	4	0
Nonstandard Ballast	7	2***	2	3	3
Warning Lights	19	5	1	3	14
Temporary Signs and Supports	10	1	0	9	9
Types I and III Barricades	9	1	2	8	7
Small Channelizing Devices	19	16	0	1	2
Total	108	54	5	39	37

*Some devices failed more than one factor, thus total failures may exceed total devices tested.
**Four tests included drums thrown to one side, but not judged to threaten other traffic.
***One test rated satisfactory for primary criteria resulted in extensive vehicle damage.

FIGURE 1 An 1,800-lb car impacting a 55-gal steel drum at 60 mph resulted in loss of vehicle control.

ballast in an open-top drum scattering across the pavement, and causing a skidding hazard. Typical impact performance by plastic drums ballasted with sandbags at ground level consisted of the sandbag and base (if used) remaining near the point of impact, with the drum staying against the front of the car or under it. Even in several impacts with the front corner of the vehicle, drums wrapped around the car's front and stayed there or came to rest under it. Drums with detachable bases, external base tabs, or slotted base fingers all displayed similar behavior. Damage to plastic drums was variable, 11 tests resulting in only superficial damage. Seven drums were completely destroyed, and the other six experienced intermediate damage. Both open-top drums and one two-piece drum split at midheight were totally destroyed, as was one with slotted base fingers. This severe damage related to ballast being trapped inside the drum, thus offering increased resistance to movement by the drum on impact. It was also apparent that some brands of drums were more resilient than others, experiencing less tearing and breakage in similar impacts. Some drums were used in several tests—although some were completely destroyed after only one impact, others were still serviceable after several impacts. Plastic drums with sandbag ballast placed at ground level generally provided excellent performance. However, open-top drums with internal ballast and two-piece drums split at midheight both resulted in debris that could threaten other traffic.

Unballasted Plastic Drums

Performance was similar to that of the drums ballasted with 50-lb sandbags. Test results were completely satisfactory in 7 of the 15 tests, with the drum staying with the car. In four other tests, drums were pushed to the right by brushing impacts. Although drum trajectories were not sufficient to threaten other traffic significantly, they did include thrown debris. Because of their light weight and soft material construction, this debris did not threaten workers. Four out of 15 tests, all involving two-piece drums split at midheight, resulted in the top half being thrown high into the air and a long distance into the work zone, even for 30-mph impacts by only 1,800-lb cars. This behavior was considered a possible threat to other traffic, and these four tests were classified as unsatisfactory. Damage was similar to ballasted drums, with 11 out of 15 drums suffering only superficial damage. Four drums were destroyed—two from corner impacts with front tires rolling over the drum, and the other two from shattering and tearing on impact. Other than reducing the drum damage caused by added resistance of the sandbag ballast, performance of unballasted drums was similar to those with ballast. On the basis of these tests, bagged sand ballast at ground level, up to 50 lb per drum, does not appear to affect drum performance adversely.

FIGURE 2 Typical impacts with plastic drums ballasted with 50-lb sandbags resulted in drums staying with the front of the car (top), being pushed aside in a brushing impact (center), and two-piece drum being thrown high into the air (bottom).

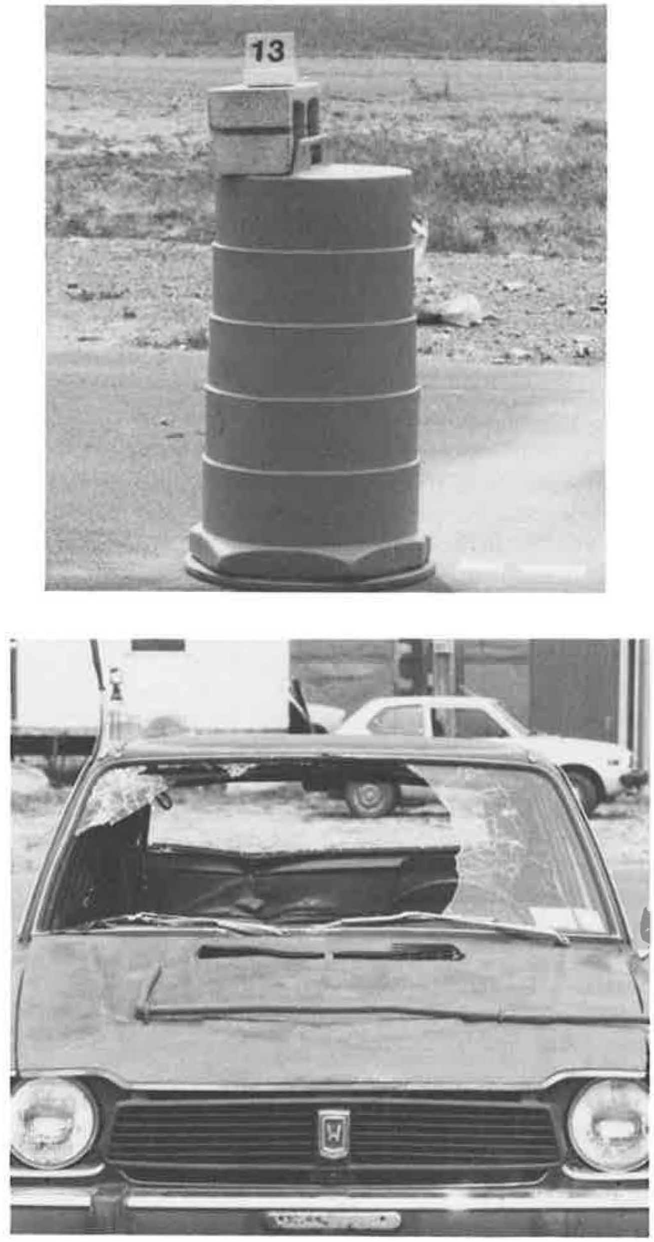

FIGURE 3 A 42-lb concrete-block ballast placed on a drum resulted in unacceptable intrusion into the passenger compartment.

Plastic Drums with Nonstandard Ballast

Ballast, other than bagged sand at pavement level, provided satisfactory results in only two of seven tests. A suspended 30-lb sandbag hanging from the top of the drum provided acceptable results. Another drum containing about 20 gal of water met the three primary criteria, although the drum was destroyed and the front of the car sustained substantial damage in a 60-mph test. The five other tests were considered unsatisfactory. Two tests used 42-lb concrete blocks on top of the drum as ballast. In the 30-mph test, the block entered the passenger compartment through the windshield, and nearly exited the rear window (Figure 3). In a similar test at 45 mph, the block impacted and severely crushed the leading edge of the roof, but did not enter the passenger compartment. Both of these tests represented potentially fatal injuries to vehicle occupants. A sandbag on top of a drum resulted in sand scattered over a wide area of pavement, considered unacceptable debris. An open-top drum, ballasted inside with 180 lb of gravel, was torn apart on impact, and the drum's top portion was thrown and could have threatened other traffic.

Finally, an open-top drum ballasted with construction debris—broken concrete and 2- by 4-in. lumber—resulted in debris thrown throughout the work zone, an unacceptable risk to workers and other traffic. In addition to unacceptable behavior in terms of the primary evaluation criteria, all three open-top drums and the drum with a sandbag on top were destroyed by the impacts (this last drum had been impacted in four previous tests). Added resistance of the heavier ballast and the inability of internal ballast to separate from the drum resulted in severe impact forces. In previous tests with standard ballasts, most drums withstood similar impacts with only minor damage.

Plastic Drums with Warning Lights

Of 19 tests of plastic drums with Type A warning lights attached (Figure 4), only five met the primary evaluation criteria. Lights were attached to the drums by various methods. The primary problem was that the lights, weighing about 6 lb including lantern batteries, separated on impact and flew through the work zone, creating hazards to workers or other vehicles' windshields. In two 60-mph tests, batteries traveled about 250 ft from impact, and over 150 ft in several others. Several attachment methods were examined. In one test with an unbolted warning light set into a retainer pocket molded into the top of the drum, the light detached on impact as expected. In 11 tests, the light was attached using a ½-in. bolt without a washer. Attachment points to the drums included various retainer tabs and pockets, and on open-top drums the light was bolted to the side. In all but 2 of these 11 tests—both at 30 mph—the bolts pulled through the plastic and the lights detached on impact. In eight of the nine tests in which lights broke free, they were considered a hazard to workers, and in the ninth the light embedded in a windshield. In three other tests, lights impacted and damaged windshields, although there was no penetration, and then were thrown into the work zone.

In an attempt to avoid light detachment, seven additional tests were conducted with 1-in.-OD washers installed behind the bolt heads to prevent their pulling through the plastic. In three tests with 4,500-lb cars, two at 60 mph and one at 30 mph, the lights remained attached to the drums, and the drums stayed with the front of the car on impact, thus providing acceptable results.

None of the four tests with 1,800-lb cars at 60 mph were acceptable. In one, the bolt and washer pulled through the plastic and the light impacted the windshield. In the second, the top of the drum broke apart, throwing the light into the work zone. In the third, the light unit remained attached but the battery compartment ruptured, throwing the batteries into the work zone. In the fourth, the light remained attached but the increased weight of the light on the top of the drum, combined with the low frontal profile of the small car, resulted in the drum's flying over the car rather than staying in front, presenting a potential threat to other traffic and workers.

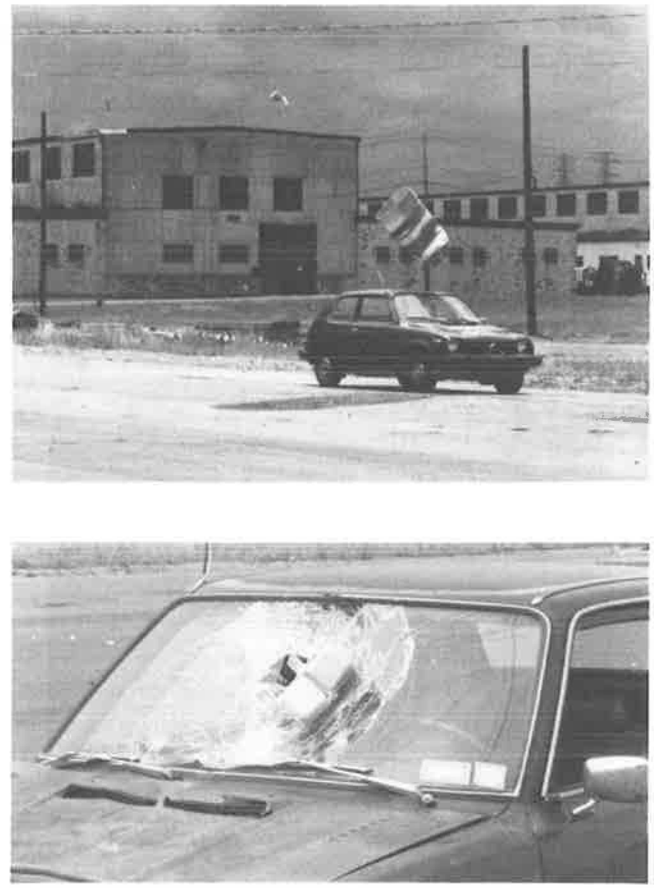

FIGURE 4 Attaching warning light to channelizing drums resulted in lights being thrown on impact and drums flying over the vehicle (top), with varying degrees of windshield damage (bottom).

Temporary Sign Supports

Of 10 supports tested, only one met the three primary evaluation criteria. Nine tests resulted in interference with control of the vehicle from windshield impacts or threatening debris, as well as debris considered a threat to workers or to the windshields of other vehicles (Figure 5). In four tests on low-mounted signs, with the bottom of the panel at bumper height, rigid wood or metal panels were flipped back into the car windshield, three of the four resulting in windshield damage. In addition, the steel tripod and wood supports were all thrown on impact, threatening other workers and traffic.

In four 60-mph tests on high-mounted signs on timber supports (panels were above the car roof) the panels presented no hazard. In every case the test vehicle passed under the panel, which dropped to the pavement near its original location. However, the 2- by 4-in. lumber braces were thrown on impact and presented a hazard to other vehicles and workers. In three out of four tests, debris from the support also impacted and damaged the test vehicle's windshield.

Use of a commercial metal tripod with a 4-ft diamond sign panel mounted 4 ft above the pavement, tested at 55 mph, resulted in the panel's being pulled down into the windshield on impact. The vertical support did not fracture or release on impact, but instead deformed against the front of the car. The panel broke free after striking the hood and windshield and was thrown over the car, presenting a hazard to workers and other traffic. Except for one leg that broke free but remained on the pavement, no harmful debris resulted from the support.

FIGURE 5 A portable low-mounted sign resulted in windshield penetration and debris (top). A tall portable sign resulted in unacceptable debris, although the panel cleared the car (bottom).

If equipped with a flexible rather than rigid panel, this support might perform acceptably.

The metal support constructed by the Thruway Authority was the only one to perform acceptably. The vehicle impacted one leg and passed under the panel. One base support broke free and slid along the pavement, and the panel and remaining support fell at the impact point.

Type I and III Barricades

Four tests on Type I barricades resulted in debris being thrown into the work zone, threatening workers and other traffic (see Figure 6). In three 60-mph tests, debris was thrown from 102 to 172 ft, and in the single 30-mph test, 70 ft. Considering that these barricades weighed 18 to 32 lb and included various steel and wood members, this debris appears to present a significant hazard if it were to strike a worker or the windshield of another vehicle. In each case, debris was thrown high in the air, presenting a substantial risk that such contact would occur.

A 45-mph impact on a wooden Type III barricade resulted in unacceptable debris—pieces of 2 by 6 in. were thrown 150 ft from impact. This was expected, and wooden Type III barricades have not been permitted on New York State projects for the last decade.

Three tests of PVC-plastic Type III barricades resulted in their shattering, with debris thrown up to 207 ft from impact at 60 mph. Resulting debris was light in weight, and did not appear to represent a significant hazard to workers or other traffic. All three tests resulted in broken windshields on the test vehicles. In two of the three tests, a warning light was attached to the top barricade rail that contributed to windshield damage. In the third (at 60 mph) the windshield of the large sedan was shattered by impact with the top barricade rail (Figure 7). All three tests with PVC barricades were thus considered unsatisfactory because of windshield damage. These barricades were all constructed using heavy grade pipe (ASTM D 2655) and a lighter grade might prevent this damage.

Results of a single 60-mph test on a metal Type III barricade were considered acceptable. It deformed around the front of the 1,800-lb vehicle and produced no debris or impact with the windshield. The barricade was extensively damaged, with some cosmetic damage to the front of the impact vehicle, but no threat to workers, other traffic, or occupants of the test vehicle.

FIGURE 6 Collision with a Type I barricade resulted in unacceptable debris.

FIGURE 7 A 60-mph impact with a PVC Type III barricade resulted in a shattered windshield.

Small Channelizing Devices

In 19 tests on cones, tubes, and panels, 16 provided acceptable results. Two of the three unacceptable tests resulted in warning lights attached to the devices being thrown on impact. In addition, one vertical panel provided unacceptable results when its base plate was tipped over before impact. A front tire impacted the leading edge of this steel plate, resulting in a blowout and partial loss of steering control. In addition, the plate was thrown into the work zone by the impact, although it remained near pavement level. In seven out of nine tests, at speeds ranging from 20 to 60 mph, the panels, vertical supports, and base plate connections were damaged to the extent that replacement parts were required to place the device back in service, and one was damaged beyond repair. In this regard, vertical panels were inferior to cones and tubes, with only 2 of 10 devices tested requiring repair after impact. Except for warning lights added to these devices and an improperly deployed vertical panel, these small channelizing devices appear to perform very well in full-scale impacts, presenting no significant hazard to workers or traffic.

SUMMARY AND FINDINGS

Test procedures and evaluation criteria based on modifications to those in NCHRP Report 230 (*4*) provided considerable insight into the performance of typical work zone traffic control devices. Results of 108 full-scale tests show that some devices create hazards when impacted. Performance deficiencies noted included penetration of the passenger compartment through the windshield, loss of or interference with vehicle control, and debris thrown through the work zone that was considered potentially hazardous to workers or passengers of other vehicles.

Although some test results were not considered acceptable, many devices performed well in a number of tests. Plastic channelizing drums, both unballasted and ballasted with 50-lb sandbags, typically performed well, in most cases staying with the car's front after impact. However, open-top drums with ballast inside and two-piece drums split at midheight generally did not perform as well. Small channelizing devices—cones, tubes, and vertical panels—also performed well in most tests. On the other hand, 55-gal steel drums performed poorly, resulting in loss of vehicle control or threatening workers and other traffic when the drums were thrown through the work zone.

Nonstandard ballast, especially heavy ballast on top of drums, caused potentially severe results from penetration of the windshield and debris thrown through the work zone. Warning lights attached to channelizing devices also detracted from performance. In some cases, lights were thrown free on impact and damaged the windshield or were thrown through the work zone, causing a hazard to workers and other traffic. In other cases, lights caused drums to fly over the impacting vehicle rather than remain in front of it, but no lights completely penetrated a windshield.

Most portable sign supports tested did not perform acceptably. Rigid panels mounted at bumper height impacted windshields, threatening intrusion into the passenger compartment. Panels mounted above roof level cleared the car and remained near the impact point. However, debris from temporary timber and steel supports was thrown through the work zone in most tests, causing severe hazard to workers and other vehicles.

Type I and III barricades also provided mixed results. All four Type I barricades tested, even in 30-mph tests, were thrown on impact and appeared to represent a risk to workers and other traffic. PVC-plastic Type III barricades resulted in considerable debris, although this was not considered a significant threat. However, all PVC Type III barricade tests resulted in windshield damage. A steel Type III barricade performed well, with no debris and no windshield damage.

Based on 108 full-scale crash tests on 62 combinations of work zone traffic control devices and installation conditions, the following findings can be stated:

- Full-scale vehicle tests based on modified NCHRP Report 230 procedures and evaluation criteria provided significant insight into impact performance of work zone traffic control devices.
- Many typical work zone traffic control devices performed well, but some devices and deployment conditions resulted in potentially hazardous performance in a number of tests.
- Plastic drums, cones, tubes, and vertical panels performed well in most tests when properly deployed and ballasted.
- Improperly ballasted channelizing devices, especially ballast placed above ground level, may present a significant hazard to motorists and workers.
- Warning lights attached to channelizing devices became flying objects in a number of tests, which resulted in windshield damage in some tests, although none completely penetrated a windshield. They may also threaten workers when the lights are thrown into a work zone.
- Most temporary sign supports tested did not perform well. Rigid sign panels mounted at bumper height were thrown onto windshields. In addition, debris from several supports threatened workers and other traffic.
- Type I and III barricades had mixed results. Some performed well, but others resulted in windshield damage, unacceptable debris, or both.

This research is also described in Research Report 147 (*8*), available from the Engineering Research and Development Bureau, New York State Department of Transportation, Albany, N.Y. 12232. That report provided detailed data on each of the 108 tests conducted, as well as an expanded description of the test procedures and evaluation criteria used.

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Improving Work Zone Delineation on Limited Access Highways

FRANK D. SHEPARD

The purpose of this study was to investigate vehicle guidance through work zones by evaluating the effectiveness of two primary components of traffic control relative to delineation. First, a comparison of the steady-burn lights now used on top of temporary concrete barriers was made with experimental reflectorized panels. Second, the addition of closely spaced, raised pavement markers as a supplement to the existing pavement markings was evaluated. The study was limited to work zones on Interstates and four-lane highways. The results of this investigation have led to the recommendation that (a) steady-burn lights on temporary concrete barricades should be replaced with reflectorized panels fabricated with high-intensity sheeting and placed along the tangent sections only and (b) closely spaced, raised pavement markers should be used as a supplement to existing pavement striping in areas where the roadway alignment changes.

With traffic volumes increasing and many roads operating at or near capacity, the upsurge in highway construction, coupled with the rehabilitation of existing facilities, will result in greater exposure of motorists to work zone activities.

The seriousness of the problem of safety in work zones is reflected in FHWA statistics that show work zone fatalities rising from 489 in 1982 to 678 in 1985. Virginia statistics show that 29 people died and 167 were seriously injured in work zone accidents in 1985. Work zone safety is therefore of high priority, and it is important to find ways of protecting the motoring public and the work force.

One way of increasing work zone safety is to provide clear and positive guidance for motorists approaching and traversing the area. Whenever a work zone is present, motorists are required to travel a section of highway that may deviate from their expected travel path because of narrow lanes, closed lanes, and detours.

The magnitude of the problem is demonstrated by the following list, which encompasses the sources of confusion prevalent within work zones:

- Roadway geometry and alignment are different from the original and expected layout.
- There are conflicting travel cues, including different pavement colors and textures; pavement joints are not parallel to traffic flow or are not between lanes of travel.
- Old pavement markings often have not been erased, and erased markings create different roadway color and texture.
- There is a lack of visibility because of weather, lighting, dirt, and worn pavement markings.
- There is a lack of uniform application of markings within similar work zones.
- Drivers' views of markings are obstructed by a high volume of traffic or by trucks.
- Opposing headlight glare is greater than normal.

All of these sources of confusion impose an added burden on drivers at the same time that they are forced to perform a maneuver that may be unfamiliar and unexpected.

Therefore, it is important that every effort be made to reliably indicate the direction of road alignment and the severity of any change in direction. The *Manual on Uniform Traffic Control Devices* (MUTCD) states: "The intended vehicle path should be clearly defined during day, night, and twilight periods under both wet and dry pavement conditions."

The Virginia Department of Transportation provides an array of traffic control devices in work zones including signs, pavement markings, delineators, steady-burn lights, and barriers, all of which define travel lanes. Two components of this traffic control system that influence motorist guidance are steady-burn lights placed on top of the concrete barriers and pavement markings placed on the roadway. Because of the importance of using optimal delineation techniques in work zones, the effectiveness of these two traffic control systems was investigated.

Steady-Burn Lights

Steady-burn lights are used in Virginia to help delineate the vehicle path through and around obstructions in a construction or maintenance area. They are placed on top of precast concrete traffic barriers, at 80-ft centers on the barrier taper (between chevrons) and tangent sections. Although the steady-burn lights are quite visible, there are several reasons to questions their use:

- Lights are dependent on batteries, and thus require maintenance. When a light burns out, the 160-ft spacing leaves partial and often confusing guidance.
- Many states use steady-burn lights on a limited basis. For example, the New Jersey Department of Transportation (DOT) found that the use of 6- × 12-in. reflectorized panels instead of steady-burn lights caused no decrease in the proportion of vehicles using the lane adjacent to the temporary construction barrier and caused no damage in the mean speed and speed variance. The New Jersey DOT has been using the reflectorized panels on tangent sections of temporary concrete barriers for 5 years and has reported no problems. Lights are still used in the taper area.

Virginia Transportation Research Council, Box 3817 University Station, Charlottesville, Va. 22903.

- Steady-burn lights cost from $0.70 to $1.40 per light per day.
- Recent research by the Virginia Transportation Research Council investigated the use of reflectorized panels as concrete barricade delineators (as a substitute for lights). It was found that the devices were feasible in terms of application and cost.

Because of these concerns, the possibility of replacing the steady-burn lights with reflectorized panels was investigated.

Pavement Markings

Pavement markings serve an important function because they help provide smooth, safe transitions from one lane to another, onto a bypass or detour, or into a reduced width of traveled way. Pavement striping is primarily used to clearly define the intended vehicle path during day, night, and twilight periods under both wet and dry pavement conditions.

One technique that can be used to enhance work zone delineation involves the use of raised pavement markers as a supplement to the pavement striping. These markers are bright and protrude above the road surface, providing improved visibility, especially during hazardous wet pavement conditions at night. In a previous study (*1*), it was the consensus of 11 highway agencies that the use of raised pavement markers in high-hazard locations enhanced the delineation and improved the overall safety of the locations. This study and many others (*2–4*) have been concerned with the advantages of using raised markers for roadway delineation; however, it is felt that there is still room for improvement in techniques for work zone delineation. The state of Virginia recently conducted preliminary studies using different raised marker devices and spacing as a supplement to existing edge line markings. These techniques provided positive guidance in the transition areas.

PURPOSE AND SCOPE

The purpose of this study was to investigate vehicle guidance through work zones by evaluating the effectiveness of two primary components of traffic control relative to delineation. First, a comparison was made between the steady-burn lights now used on top of temporary concrete barriers and experimental reflectorized panels. Only tangent sections of the work area were considered (no transitions). Second, the addition of closely spaced raised pavement markers as a supplement to the existing pavement markings was evaluated. Observations were limited to areas where the roadway alignment deviated from the original, i.e., lane and road transitions and detours. The study was also limited to work zones on Interstates and four-lane highways.

STEADY-BURN LIGHTS

Steady-burn lights and reflectorized panels were placed on top of temporary concrete barriers along the tangent sections only. These devices (see Figure 1) were compared at two sites. Site 1 (see Figure 2) was a four-lane divided highway with two lanes closed; therefore, the two southbound lanes carried two-way traffic separated by temporary concrete barriers on which the lights and panels were placed. The barrier was placed on the left side of traffic, and 37 delineators were spaced at 72-ft intervals.

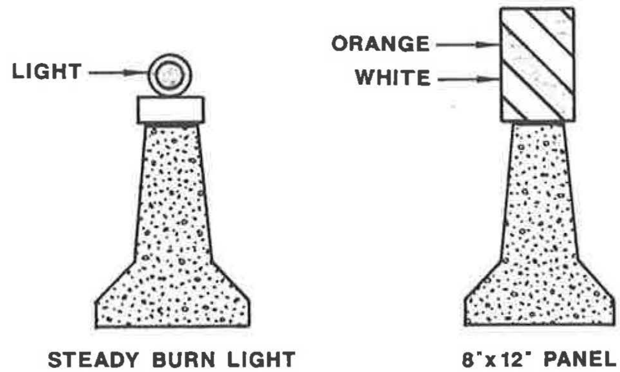

FIGURE 1 Concrete barrier delineators.

FIGURE 2 Site 1, Route 29, Leon, Virginia.

Site 2 (see Figure 3) was an Interstate highway that had temporary concrete barriers placed on the right shoulder. There were 17 delineators spaced 48 ft apart on top of the temporary concrete barricade. Old centerline markings, although partially visible during the day, were not expected to influence driver behavior at night.

Procedure

To measure the effectiveness of the steady-burn lights and reflectorized panels, traffic flow data were collected using a system of traffic counters with rubber tubes:

- *Vehicle Speed.* Vehicle speeds were recorded using two tubes as a speed trap.
- *Vehicle Placement.* The placement of vehicles relative to the lane line next to the concrete barrier was recorded using tubes of different lengths.

All data were collected on weekdays between the hours of darkness and 5:00 a.m. Videotapes were made of the test sections for the purpose of documentation.

FIGURE 3 Site 2, Interstate 85, Petersburg, Virginia.

Results

Vehicle Placement

Vehicle placement was determined at Site 1 by observing the number of vehicles at 0- to 1.5-, 1.5- to 3.0-, 3.0- to 4.5-, and 4.5- to 6.0-ft intervals from the edgeline for each delineation treatment. Figures 4 and 5 show the percentage of vehicles within each interval from 8:00 p.m. to 1:00 a.m. and from 1:00 a.m. to 5:00 a.m., respectively. Data were collected for two weekdays for each time period and each set of delineators. The results indicated no difference in vehicle placement using the steady-burn lights or the reflectorized panels. It is interesting to note that there was a difference in placement between the two time intervals, probably because of heavy truck traffic during the early morning hours.

Vehicle placement from 9:00 p.m. to 1:00 a.m. and 1:00 a.m. to 5:00 p.m. for the steady-burn lights and reflectorized panels at Site 2 is shown in Figures 6 and 7, respectively. Two weekdays of data were collected for each period and delineation treatment. There were differences in vehicle placement for both periods. The 2- to 4-ft interval and the 9:00 p.m. to 1:00 a.m. time period had 5.4 percent more vehicles for the reflectorized panels, whereas the 6- to 8-ft interval had 5.8 percent fewer vehicles. Also, for the 1:00 to 5:00 a.m. time period, 6 percent more vehicles were found for the reflectorized panels with a placement interval of 4 to 6 ft, and 6 percent fewer vehicles were shown for the 6- to 8-ft interval. This indicates that fewer vehicles were straying from the lane adjacent to the concrete barricades using reflectorized panels as compared with the steady-burn lights.

Vehicle Speeds

The average vehicle speeds observed at Sites 1 and 2 from 8:00 p.m. to 1:00 a.m. are as follows:

	Speed (mph) by Treatment	
Site	*Lights*	*Panels*
1	53.4	53.0
2	55.7	56.3

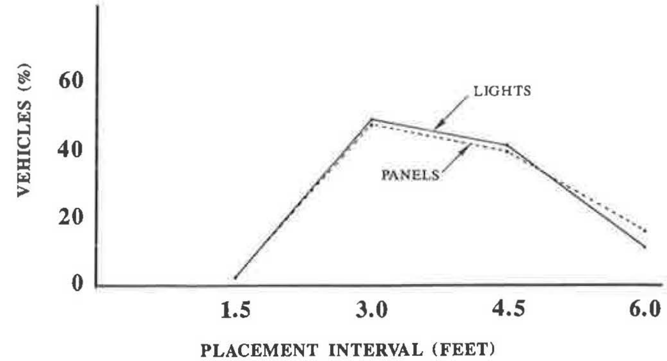

FIGURE 4 Percent vehicle placement from 8:00 p.m. to 1:00 a.m. (Site 1—Leon).

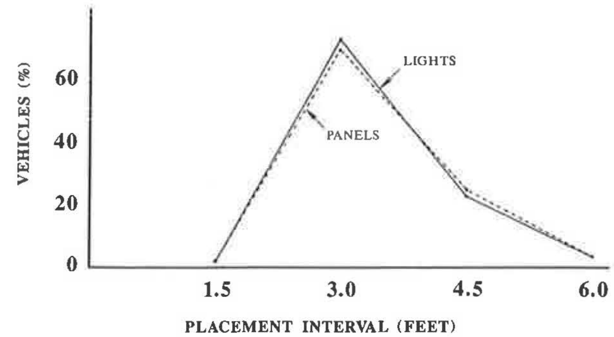

FIGURE 5 Percent vehicle placement from 1:00 a.m. to 5:00 p.m. (Site 1—Leon).

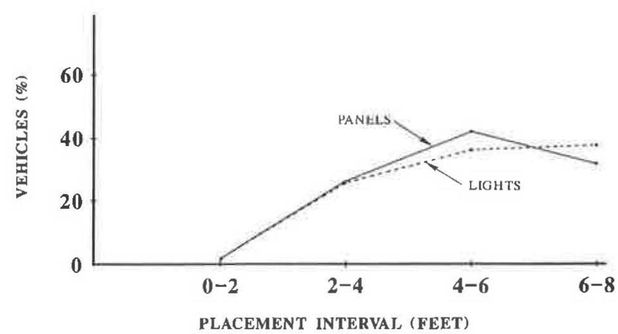

FIGURE 6 Percent vehicle placement from 9:00 p.m. to 1:00 a.m. (Site 2—Petersburg).

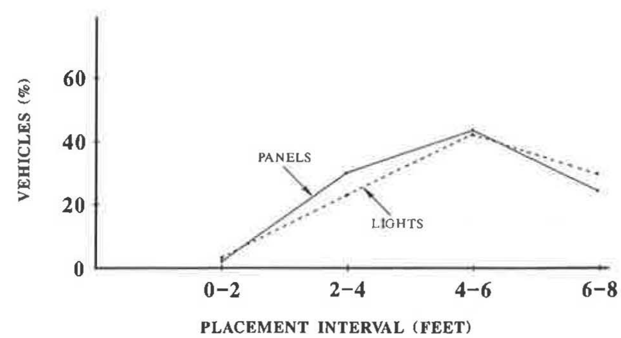

FIGURE 7 Percent vehicle placement from 1:00 a.m. to 5:00 a.m. (Site 2—Petersburg).

Two weekdays of data were collected for the steady-burn lights and reflectorized panels. The results showed no significant difference in speeds between the two delineation treatments.

Videotapes of Test Sites

Videotapes were made at two test sites to compare the lights versus the reflectorized panels. Videotapes were made at Site 1 (Leon, southbound) during daytime, night/dry, and night/wet conditions, and at Site 2 (Petersburg) during daytime and night/dry conditions.

RAISED PAVEMENT MARKERS

The use of raised pavement markers as a supplement to the existing work zone pavement markings was investigated for three sites. The raised markers were placed within the transition areas or where the alignment deviated from the original. The temporary markers were plastic with curve-corner face reflectors and were placed using a butyl pad.

Site 1 was a detour for a four-lane divided highway; the northbound lanes were closed (see Figure 8). The S-shaped detour had preformed tape along the right edgeline and a painted stripe along the left edgeline. The schematic in Figure 9 shows the location and spacing of the raised pavement markers and data collection points.

Site 2 was a four-lane highway with the right lane closed (see Figure 10). Raised pavement markers were added to the existing markings along both the right transition and left centerline. The schematic in Figure 11 shows the location and spacing of the markers.

Site 3 was an Interstate, with left lane closure and raised markers supplementing the existing left edgeline transition (see Figure 12). The markers were placed directly on the new preformed tape. The schematic in Figure 13 shows the marker placement and data collection points.

Procedure

To measure the effectiveness of the pavement striping and striping supplemented with raised pavement markers, traffic flow data were collected using a system of traffic counters with rubber tubes:

- *Vehicle Speed.* Vehicle speeds were recorded using two tubes as a speed trap.
- *Vehicle Placement.* The placement of vehicles relative to the lane line next to the concrete barrier was recorded using different length tubes.

FIGURE 8 Site 1, Rt. 29, Leon, Northbound.

FIGURE 10 Site 2, Route 1, Fredericksburg.

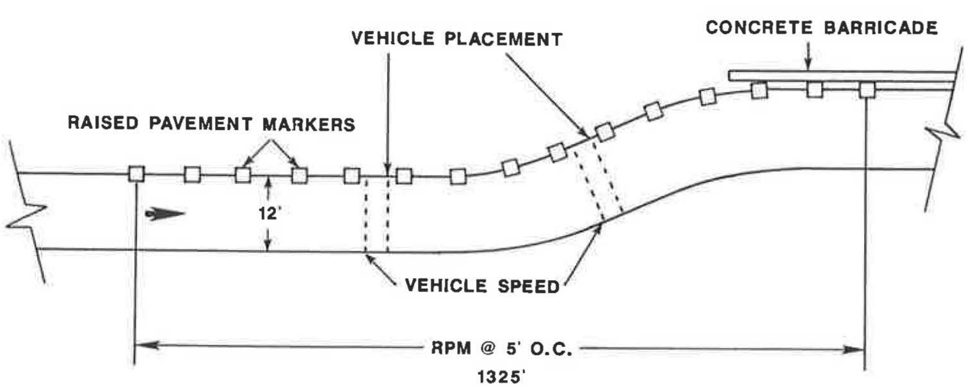

FIGURE 9 Schematic of Site 1, Leon.

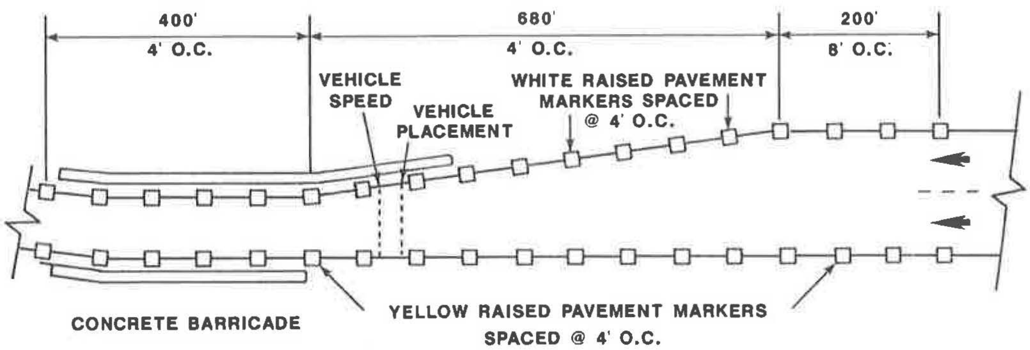

FIGURE 11 Schematic of Site 2, Fredericksburg.

FIGURE 12 Site 3, Interstate 81, Salem.

- *Position of Weave.* The position of weave within the transition area was recorded by dividing the area into zones and determining the magnitude of weaving within each zone.

Because of the importance of delineation during night/wet conditions, it was hoped that each variable could be tested under wet conditions; however, lack of rain limited data collection to dry conditions.

All data were collected on weekdays between darkness and 5:00 a.m. Videotapes were made of the test sections for the purpose of documenting the pavement markings observed.

Results

Vehicle Placement

Vehicle placement was measured for Sites 2 and 3. Figures 14 and 15 show vehicle placement for Site 2 from 9:00 p.m. to 1:00 a.m. and from 1:00 to 5:00 a.m. For both time intervals, there were more vehicles in the 2- to 4-ft interval for the raised pavement markers as compared with no raised markers. Fewer vehicles were in the 6- to 8-ft interval from 9:00 p.m. to 1:00 a.m. and in the 4- to 6-ft interval from 1:00 to 5:00 a.m. for the raised markers. A 12-ft pavement width at the point where the placement was taken meant that vehicles were staying closer to the center of the lane.

Placement for Site 3 is shown in Figures 16 and 17. Little difference in vehicle placement was found for each time period.

Discussion of Results

The raised pavement markers are most effective during night/wet conditions, because the water significantly reduces the retroreflection capabilities of the pavement striping, leaving the raised pavement marker, which protrudes above the water, as a primary source of reflected light. The unavailability of appropriate wet conditions during testing prevented data from being obtained during the time when raised pavement markers are the most effective. Figure 18 shows an example of the raised pavement markers used at Site 1 (Leon, northbound)

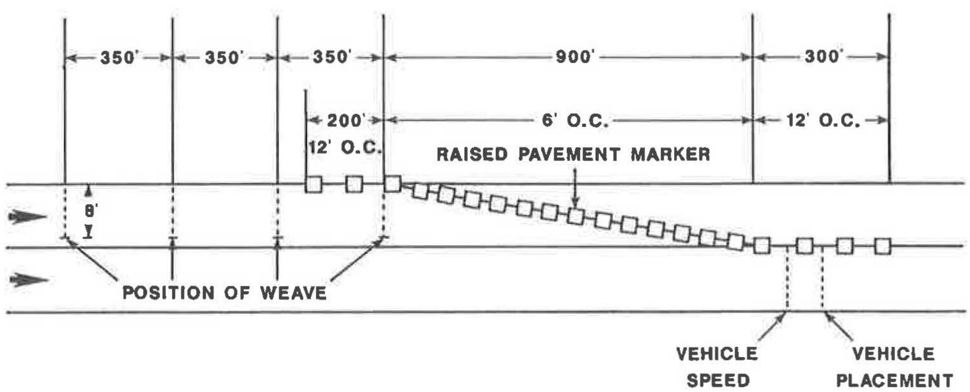

FIGURE 13 Schematic of Site 3, Salem.

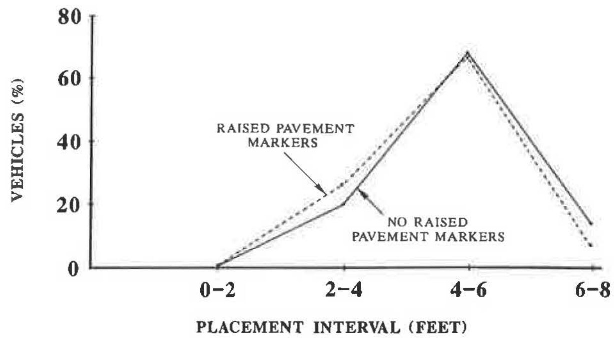

FIGURE 14 Percent vehicle placement from 9:00 p.m. to 1:00 a.m. (Site 2—Fredericksburg).

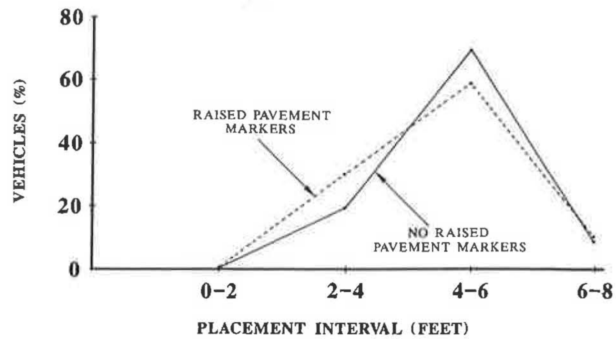

FIGURE 15 Percent vehicle placement from 1:00 to 5:00 a.m. (Site 2—Fredericksburg).

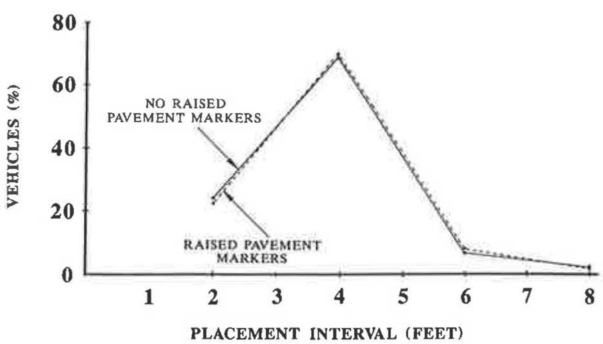

FIGURE 16 Percent vehicle placement from 9:00 p.m. to 1:00 a.m. (Site 3—Salem).

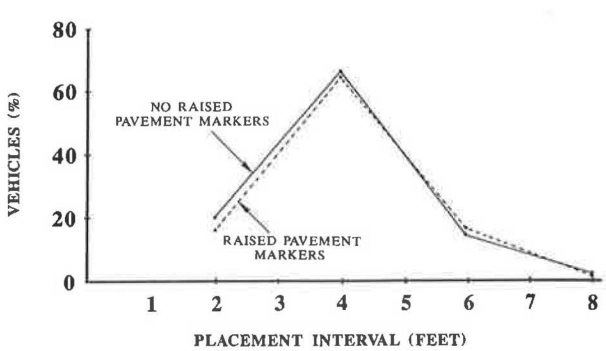

FIGURE 17 Percent vehicle placement from 1:00 to 5:00 a.m. (Site 3—Salem).

during wet conditions. The positive guidance capabilities are obvious; note the low visibility of the painted line. Existing pavement striping at Site 2 was judged to be average, with some parts below average primarily because of poor pavement conditions (cracks, scaling, irregular surface resulting from milling, dirt accumulation, etc.). Therefore, it was felt that the addition of the raised markers at Site 2 would increase delineation by creating a brighter path for motorists to follow. This observation seems to be supported by the placement data, which show that a higher percentage of vehicles traveled in the center of the lane, with less encroachment on the centerline.

Site 3 revealed little difference in vehicle placement with and without the raised pavement markers. This site, however, had new preformed tape for the transition on which the raised markers were placed. This material remained very bright during the test period and provided good guidance. Because of the brightness of the tape, the raised pavement markers did not provide the contrast needed for increased delineation. Under wet pavement conditions, especially heavy rain, the brightness of the pavement striping would be greatly diminished, leaving the raised markers as the primary source of guidance.

Vehicle Speeds

The average vehicle speeds for the three sites are as follows:

	Vehicle Speed (mph) by Treatment	
Site	*No Raised Pavement Markers*	*Raised Pavement Markers*
1a, Leon	41.5	43.5
1b, Leon	43.6	50.0
2, Salem	56.3	55.7
3, Fredericksburg	43.6	45.6

Site 1 had two speed observation points. Speeds were observed for all sites between the hours of 9:00 p.m. and 1:00 a.m. The same weekday was used for comparing each delineation treat-

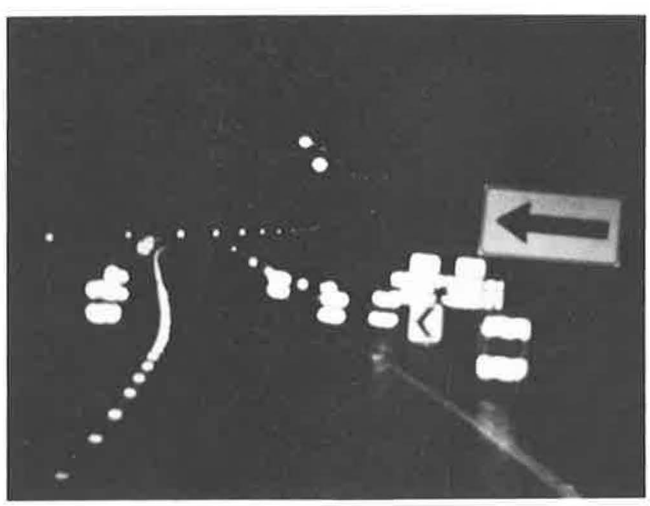

FIGURE 18 Raised pavement markers and night/wet conditions at Leon.

TABLE 1 POSITION OF WEAVE FOR RAISED PAVEMENT MARKERS VERSUS NO RAISED PAVEMENT MARKERS

	Percentage of Vehicles by Position (ft from taper)							
	1,050		700		350		0	
Time	No RPMs	RPMs	No RPMs	RPMs	No RPMs	RPMs	No RPMs	RPMs
9:00 p.m. to 1:00 a.m.	3.6	3.6	2.7	2.7	1.7	1.6	0.5	0.4
1:00 a.m. to 5:00 p.m.	1.5	0.7	1.0	0.4	0.5	0.5	0.2	0.4

NOTE: RPM = raised pavement marker.

ment. Posted advisory speed limits were 25, 55, and 45 mph for Leon, Salem, and Fredericksburg, respectively.

These results show an increase in average speed for Sites 1a, 1b, and 3. Little difference (0.6 mph) was observed at Site 2. The raised pavement markers provided more contrast or brightness than the painted lines on which they were placed at Sites 1 and 3, thus accounting for the speed differential. Also, delineation at the Site 1 detour was felt to be more critical because of the narrow lanes, S-shaped curves, and downhill topography. As noted earlier, the relative brightness of the tape edgeline at Site 2 caused the raised markers to be less effective, resulting in the small difference in speeds at that site.

Position of Weave

The position of weave was observed for Site 3 by recording the number of vehicles in the left lane at the taper and at distances of 350, 700, and 1,050 ft from the beginning of the taper. Table 1 presents the percentage of vehicles in the left lane from 9:00 p.m. to 1:00 a.m. and from 1:00 to 5:00 a.m. Two time intervals were used because of the different characteristics of early and late night traffic.

These data indicate that the addition of the raised pavement markers did not change the position of weave of vehicles approaching the left lane closure.

Videotapes of Test Sections

Videotapes were made of the test sections showing pavement striping versus pavement striping and raised pavement markers. At Site 1, Leon, northbound, videotapes were made during daytime, night/dry, and night/wet conditions. Videotapes were made at Site 2, Fredericksburg, during daytime and night/wet conditions and at Site 3, Salem, during daytime and night/dry conditions.

CONCLUSIONS

Steady-Burn Lights versus Reflectorized Panels

Analysis of vehicle placement data at two sites showed no difference at one site, whereas the other revealed less straying from the lane with the reflectorized panels. Speed data comparisons showed no differences in speeds at the two sites; therefore, it was concluded that the reflectorized panels were at least equal or superior to the steady-burn lights.

Use of Raised Pavement Markers to Supplement Existing Striping

The addition of raised pavement markers influenced vehicle placement at Site 2 by causing fewer centerline encroachments, although little change was noted for Site 3.

Vehicle speeds increased at both observation points at Sites 1 and 3; whereas no change was seen at Site 2. The increase in speed indicates that the drivers were more comfortable and confident of the roadway alignment and the path to follow.

For the night/dry conditions under which the raised markers were tested, positive results favored the use of raised pavement markers for supplementing existing striping.

The temporary raised markers were applied to the surface of the preformed tape at one site and over new paint at another, using butyl pads in both cases with good retention and durability. However, the site where the markers were placed over paint that had been applied to deteriorated pavements, old paint lines, and milled pavement surfaces had definite problems with marker retention. The primary problem was the failure of the paint to adhere to the pavement or old painted surface, thereby causing the marker, along with the underlying striping, to become detached, specially when hit by vehicle tires.

Although it was not within the scope of the project to test methods of adhesion, marker retention and durability will have to be considered if raised markers are to be used.

RECOMMENDATIONS

Steady-Burn Lights versus Reflectorized Panels

It is recommended that consideration be given to replacing the steady-burn lights on temporary concrete barricades with reflectorized panels. The panels should be at least the size of the ones used in this study, and fabricated with high intensity sheeting. They should be positioned at the same intervals as the steady-burn lights; however, they should be placed along the tangent sections only. Steady-burn lights should continue to be placed in the taper areas. Stripes on the panel should slope down toward the pavement. A recent study (*5*) showed that the cost of steady-burn lights was 10 to 20 times the cost of reflectorized panels (8 by 12 in.); therefore, the Department would realize a substantial savings from the use of the panels.

Use of Raised Pavement Markers to Supplement Existing Striping

The use of raised pavement markers as a supplement to existing striping showed signs of helping motorists negotiate work zone areas where there are changes in roadway alignment. These results were for dry conditions; wet conditions should lead to even greater advantages.

The use of closely spaced, raised pavement markers is a definite advantage to motorists because of the positive guidance provided as they approach and drive through work zones that present a variety of often confusing roadway alignment changes.

Because of the importance of providing positive motorist guidance and a safe driving environment within work zones, it is recommended that the Department use raised pavement markers as a supplement to existing pavement striping in areas where the roadway alignment changes (transitions, detours, etc.). There are still many questions relative to location, spacing, retention, durability, and type of raised marker. Until these questions can be answered, it is recommended that the markers be spaced on 4- to 5-ft centers in areas where there are curves or transitions and 8- to 10-ft centers for tangent sections. The method of application to the roadway should allow the marker to be placed or replaced in a minimum amount of time and with a minimum amount of disruption to the traffic flow. Adhesives that can be attached to the marker and can then be hand applied are preferable. The marker should be placed on the surface of the edgeline marking if it is judged to be securely adhered to the pavement surface. For questionable striping, the marker can be placed adjacent to the line, making sure that the pavement is free of dirt and grime.

ACKNOWLEDGMENTS

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Evaluation of Railroad Preemption Capabilities of Traffic Signal Controllers

PETER S. MARSHALL AND WILLIAM D. BERG

The subject of railroad preemption has historically not received much attention in professional literature. All aspects of preemption need to be studied and reported on in greater detail. This research examined and compared the preemption capabilities of a number of currently marketed actuated traffic signal controllers based on the National Electrical Manufacturers Association standard. Shortcomings in their preemption logic were identified, and preemption issues were discussed in terms of their operations. The evaluation was conducted from a pragmatic point of view to determine whether modern controllers allow practical and reasonable preemption design in conformance with accepted traffic engineering practice. Recommendations are offered with respect to minimum desirable operational capabilities, as well as railroad preemption nomenclature and user documentation.

Railroad preemption of traffic signal controllers, a subject not well known to many traffic engineers, has not received much attention in recent literature. Nonetheless, it is an important safety issue that the profession must address. Briefly, railroad preemption is necessary at signalized intersections near or at railroad-highway grade crossings. When a train approaches, normal operation is overridden and a special phase sequence is initiated to

1. Release a queue of vehicles that may be stopped across the tracks, and
2. Prevent signal phases conflicting with the train from displaying a green indication.

Designing an appropriate preemption sequence consists of determining which phases and timings are necessary to remove stopped vehicles from the tracks and specifying which phases, if any, can be allowed to operate during the passing of the train (*1*). Once these have been determined, the design must be implemented in the field, where the engineer can encounter an obstacle, namely, the type of traffic signal controller unit available.

There exists a wide range of controller capabilities for preemption operations, and the controller installed at the location in question may not have the ability to run the desired sequences. The problem can be just as significant, even if a new controller is installed, because of significant differences in types and brands. The objective of this research was to summarize the preemption capabilities of a number of currently used traffic signal controllers, identify shortcomings in their preemption logic, and relate some preemption issues to controller operations (*1*).

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AVAILABLE CONTROLLERS

Modern controllers are microprocessor-based and generally are available in either actuated or pretimed configurations. With the recent advances in microelectronics technology, the cost differential between the two has virtually evaporated. For this reason, as well as the increased flexibility and interchangeability they offer, many agencies are now purchasing only actuated units. Because of this trend, the scope of this research was limited to actuated controllers.

There are two general types of actuated traffic signal controllers available: Type 170 models and units based on the National Electrical Manufacturers Association (NEMA) standard. The capabilities of the Type 170 controllers are software specific and theoretically could operate in almost any manner desired (although available software is limited). Because NEMA controllers, on the other hand, have factory-set configurations and capabilities, this research was further limited to NEMA controller units. (The recommendations herein could be applied in the development of future Type 170 software.) Operations manuals for eight currently available NEMA controllers were obtained from various sources. An attempt was made to include most major manufacturers; however, several did not respond to a request for information and regrettably had to be excluded. The purpose of this research was not to criticize specific manufacturers, so the brand names and models will not be included.

The preemption capabilities of newer NEMA controllers can be summarized in one of two categories:

1. Capabilities and features that are common to all controllers reviewed, and
2. Capabilities and features that are unique to a specific brand or a subset of the reviewed brands.

In the discussion that follows, those features common to all controllers will be listed and described, and minor variations in the application of such features will be noted. In the process, additional related features not found on all controllers will be identified and reviewed. Finally, recommendations will be made as to the minimum desirable features that might be found on future controllers.

NUMBER OF PREEMPTION SEQUENCES

All of the controllers reviewed provide at least three built-in preemption sequences that can be used for either railroad or emergency vehicle preemption. The manufacturers organize these sequences in one of two ways:

1. There is one preemption sequence specifically designated to be used for the railroad preempt, and the remaining two to four sequences are used for emergency vehicle preemptions; or
2. There is no distinction between the preemption sequences, and either or all could be used for railroad or emergency vehicle preemption.

In the second case, each sequence is numbered $(1-n)$ with the lower-numbered sequences having priority over the higher numbers. This means that if a preempt is in progress when a different preempt is requested, the first is overridden by the second only if the first has a lower priority. Because railroad preemption should always have priority over emergency vehicle preemption, the sequence with the highest priority should be used for the railroad preempt (if necessary at the particular intersection), and the lower-priority sequences should be used for emergency vehicles. When the manufacturer specifies one sequence as the railroad preemptor, this priority is assigned automatically.

In comparing the two basic preemption schemes, it is recommended that controllers be configured with a separate, preassigned preempt for railroad operations. This simplifies terminology and instructions, and reduces the possibility of programming errors (i.e., not assigning the railroad preempt to the highest-priority sequence).

- Number of controllers with the recommended capability—4
- Number without the recommended capability—4

PREEMPTION OPERATIONS

All controllers reviewed provide the same basic preemption sequencing, in conformance with currently accepted practice. This includes

1. Entry into preemption,
2. Termination of the phase in operation,
3. Track clearance phase,
4. Hold interval, and
5. Return to normal operations.

Within this basic framework, there are significant variations and contrasting features between controllers. These will be summarized in the following paragraphs.

Entry into Preemption

Because of the limited amount of advance warning time commonly available, railroad preemption sequences are usually initiated by the controller immediately on detection of the train. Several controllers, however, allow a choice between a locking or a nonlocking mode of operation, similar to that of inductive loop detectors. In the locking mode, the controller initiates preemption immediately, and once the sequence has been initiated, it cannot be shortened or aborted. In the nonlocking mode, a programmable delay timer is initiated when the train is detected. If the preemption call is still present when the timer has expired, the preemption sequence is initiated as before. If the call has been removed (as would be the case if the train had stopped, reversed directions, and moved outside the limits of the tract circuit), no preemption sequences are run and operation would remain as usual.

Even though the majority of preemption installations require the use of the locking/no-delay mode of operation, the nonlocking/delay mode may be useful in areas where track switching operations are common and train speeds are slow. It is therefore recommended that the nonlocking mode (with delay timer) be included as a basic feature on all controllers.

- Number of controllers providing a nonlocking mode—3
- Number of controllers that do not—2
- Number unknown—2

Termination of the Phase in Operation

Before the track clearance phase is initiated, the controller must terminate the phase in operation at the moment the preemption call is received. There are several issues that complicate this operation.

Minimum Intervals

Because of the serious potential hazard involved in a vehicle-train collision, it may be desirable to shorten or eliminate the minimum green interval or the pedestrian clearance interval, or both, in the phase being terminated, so the track clearance green can be presented as soon as possible. This section will not discuss guidelines for determining when, if, or by how much these intervals may be shortened, only related controller capabilities. All of the controllers surveyed allow the engineer to shorten these intervals in some manner, although there is variability among manufacturers as to the amount of control provided. Four controllers offer control over both the minimum green and pedestrian clearance intervals. The duration of each can be reduced or eliminated should a preemption call occur early in the phase. The remaining controllers have more limited capabilities of this sort. One controller ignores the concept of minimum green, but still allows the pedestrian clearance interval to be shortened. This is appropriate if there is an active pedestrian call during the green phase, but if not, there exists a potential for abnormally short green times, depending on where in the interval the preempt call is received. Another controller allows just the opposite; a minimum green can be programmed with no regard for pedestrian activity. This one interval would have to be timed to satisfy both constraints. Although this option is probably preferable, it allows less flexibility and would be inefficient if the minimum green time was considerably shorter than the minimum pedestrian clearance (as may be the case with wide streets). A third controller also ignores the potential minimum green requirement and allows the pedestrian clearance interval to be modified only by aborting the time remaining as the preemption call occurs. This arrangement is less desirable than the others because of the possibility of extremely short green intervals. Furthermore, there is no flexibility in the pedestrian clearance interval: it is either all or nothing.

Based on these comparisons, it is recommended that controllers not ignore minimum green or pedestrian clearance intervals in the event of preemption and have the capability to modify each separately. This operation would provide the greatest amount of flexibility, and would not force the engineer to compromise safety. The decision to reduce or eliminate either interval in the event of preemption is one that should not be made without a detailed engineering analysis and an examination of the relative safety factors involved. The default setting should be to retain these intervals in the event of preemption, unless modified by the engineer.

- Number of controllers with the recommended capability—4
- Number without the recommended capability—3
- Not enough information in the manual to tell—1

Vehicle Clearance Intervals

The *Manual on Uniform Traffic Control Devices* (MUTCD) (*2*) requires that regular clearance intervals be used during preemption. However, a majority of the controllers surveyed allow the engineer to reduce the length of the clearance intervals when clearing for the track phase. From a safety point of view, if eliminating 1–3 sec from the clearance intervals allows the track to be cleared before the arrival of the train, the preemption design is most likely not adequate and other options would need to be considered (such as lengthening the track circuit). It is therefore recommended that controllers should not permit the shortening or elimination of vehicle clearance intervals of any phase at any point in the preemption sequence.

- Number of controllers that allow clearance modification—5
- Number of controllers that do not—3

Specifying the Track Phase

All of the controllers allow the user to specify which phase or phases will be green during the track clearance interval. The requirements for phases to run simultaneously are the same as for normal operations; a phase can be run individually or along with any other nonconflicting phase. In addition, the controllers allow individual control of the overlaps during all preemption intervals. This is particularly useful when a supplementary set of signal faces is being used to control the track clearance phase. As a result, controller capabilities are considered adequate in this area.

Number of Track Clearance Intervals

The various controllers differ in the number of track clearance phases they provide. Three of the controllers offer two separate track clearances whereas the other five offer only one. Two separate track clearance phases may be necessary in instances where the track crosses two different intersection approaches. In such cases, the ITE recommends two separate track clearance phases, but the order in which these occur differs depending on the approach direction of the train (*3*). The approach that the train will initially cross is cleared first. This logically reduces the necessary advance warning time. However, there is some question as to whether this can be readily handled by the controllers in their off-the-shelf configurations. It appears that with some special programming, two of the controllers may be able to provide this option, but it is difficult to be sure without testing the actual units. The other six do not appear to have this ability without complicated external devices or special software.

Given these considerations, it is recommended that all controllers provide the option of running two separate track clearance phases for those relatively rare installations where the track crosses two approaches. Furthermore, the two track clearances should be able to run in reverse order depending on the direction of the train. This would allow controller capability to match recommended practice.

- Number of controllers providing two track clearances—3
- Number of controllers providing only one—5
- Number of controllers potentially able to run these in reverse—2
- Number of controllers not able to reverse—6

Preemption Hold Interval

There are several significant differences among the capabilities of the surveyed controllers with respect to the preemption hold interval. These will be discussed individually.

Cycling

There is a question as to which phases should be allowed to move during the hold interval. The MUTCD suggests that the signals be operated to permit vehicle movements that do not cross the tracks. This does not specify whether it is permissible to cycle through all phases that do not conflict with the track. Some of the controllers permit cycling, while others require a hold on a specific phase. There is no apparent reason why in many situations cycling cannot be permitted, and it may offer operational efficiencies. Therefore, it is recommended that all controllers have the ability to cycle during the hold interval.

- Number of controllers that permit cycling—4
- Number that do not—4

Pedestrian Considerations

There is a related issue of what to do with the pedestrian signals during the hold interval. There is no apparent reason why nonconflicting pedestrian phases could not be serviced during the hold interval. In fact, it may be wise to do so to avoid having the pedestrians grow impatient and attempt to cross against the signal. Therefore, it is recommended that controllers allow the pedestrian phases to operate normally during the preemption hold; however, built-in options should permit the modification of the pedestrian movements during this interval. For example, it may be desirable to inhibit one or more pedestrian phases that normally would be allowed so as to reduce potential pedestrian-train conflicts.

The pedestrian phases should be settable during the hold interval to either active or nonactive. If set to active, they would operate normally if allowed by the combination of vehicle phases in operation. Also, the preemption hold interval would not be terminated following the passing of the train, but would continue until the pedestrian clearance intervals had expired. These intervals would be initiated by the train leaving the track circuit, and the preemption input being removed. This could also function as an exit delay, which will be discussed subsequently. If a pedestrian phase is set to nonactive, it would simply display a solid DON'T WALK indication until the preemption expires.

There is some uncertainty as to these capabilities in the reviewed controllers, chiefly due to lack of information in the manuals. If no detailed information is provided, the controller operates the pedestrian phases during preemption just as it would normally, although again there is no way to be sure without testing an actual unit. This design is probably adequate, but does not provide the desired flexibility.

Several of the controllers allow the pedestrian operations to be modified in some manner during the hold interval. Two controllers allow the pedestrian indications to operate as normal or be set to DON'T WALK, whereas two others allow any or all of the indications to be set to dark (off) during the train passage. There is some question as to whether turning the pedestrian indications off would be wise, as it could lead to confusion among waiting pedestrians as to what action is required of them. There are only two options for pedestrians waiting at the curb: either cross or don't cross. These instructions are both handled by the pedestrian signals so there is really no point in turning them off. Furthermore, the MUTCD requires that pedestrian signals be displayed at all times (if they exist) except when the traffic signal is being operated as a flashing device, in which case they are to be dark (*2*). If the signal was being operated in flash mode during preemption (as is certainly possible) the extinguishing of the pedestrian signals would be accomplished through the flashing logic, so there is no apparent reason to change the manual setting of the pedestrian signals to dark.

One other pedestrian-related feature is available on two of the controllers. They provide the option of modifying the "FLASHING DON'T WALK" pedestrian clearance interval at the end of the preemption hold interval (assuming the pedestrian phases are operating normally), presumably with the intention of shortening it to facilitate a more rapid transition to the exit phase. It is recommended that this capability not be utilized because there is no apparent need for a swift transition as there is when entering preemption (i.e., clearing the vehicles off the track).

In summary, it is recommended that all controllers should allow individual control of all pedestrian phases during preemption. The pedestrian phases should be settable to either active or nonactive. In addition, the pedestrian phases should be automatically inhibited during the track clearance phase so as not to conflict with vehicles clearing the track.

- Number of controllers having full pedestrian control—3
- Number that have limited control—1
- Number that allow setting to dark—2
- Number that are assumed to operate normally—4

Minimum Hold Time

All of the controllers reviewed provide the capability of setting a minimum length of hold interval to avoid a very short green interval. This is useful in situations in which a train enters the track circuit and triggers the preemption but then stops, reverses directions and moves out of the track circuit. Common procedure is to terminate the hold interval as soon as the preemption is removed, and return to normal operations as soon as possible to avoid unnecessary delays to motorists. Without the minimum hold time feature, if a train exited the track circuit just after the hold interval began, there would exist the possibility for short green times. Thus, the minimum hold time is considered a desirable feature that should be retained on existing controllers and incorporated into future units.

Exit Delay

Several of the controllers allow the user to set an exit delay to be timed before terminating the hold. This feature could be useful in areas where switching operations frequently occur because of the possibility that a train may exit the island circuit and then quickly return back across the intersection. Rather than incurring the lost time and delay of unnecessarily running the entire preempt sequence again, a programmed exit delay can hold the preemption for several seconds to minimize this possibility. This feature is considered useful and should be included on all controllers due to the flexibility it provides.

- Number of controllers with an exit delay—3
- Number of controllers without—5

Returning to Normal Operations

In returning to normal operations after a preemption sequence, it may be desirable to return to a specific phase or sequence of phases. Generally, it would seem reasonable to return and service first the phases that were delayed by the train. But if one of the delayed phases is causing a queue to back up into an adjacent intersection, it would be desirable to service that phase as soon as possible. The capability of specifying which phase to return to both after the train has passed and the exit delay (if one exists) has expired, was available on each of the surveyed controllers. If no return phase is specified, most controllers will return to the phase immediately following the phase that was interrupted by the train.

Several of the controllers allow additional exit parameters to be specified. One controller enables the user to specify the first green time of the phase returned to. This feature would be specially useful when clearing problem queues, as mentioned previously. Another controller allows the operation of a slightly modified timing plan for a specified number of cycles after the termination of the preempt; any specified phase can be run with its "max green 2" in operation and all recalls active.

In summary, it is considered desirable to be able to control the exit from preemption to some extent, although this issue is not as critical from a safety standpoint as some of the others previously discussed. The programmable exit phase is a positive feature and should be incorporated in new controllers. In addition, it would be useful to have some control over the timing of the exit phases for one or more cycles.

NOMENCLATURE

There exists a major obstacle in comparing and using the preemption capabilities of NEMA actuated controllers; the nomenclature used by the various manufacturers differs significantly. There is currently no NEMA standard defining preemption terms, so each manufacturer has been free to develop its own. Furthermore, common control strategies, symbols, and terms differ from the examples provided in the ITE Recommended Practice. The combination of these inconsistencies causes much confusion and uncertainty.

NEMA is currently in the process of developing an updated standard for actuated traffic signal controllers (TS-2) that will include a functional standard on preemption. Information available at this time indicates that the standard will require six separate preemption sequences, each with specifiable timing parameters, and signal displays for both a preempted condition and a return-to-normal operation (*4*).

This update is expected to be released for critique and comment within the next year. It is hoped that this paper will serve as a catalyst for that review process by drawing attention to items that should be addressed and stimulating interest in the user community.

TABLE 1 COMPARISON OF RAILROAD PREEMPTION FEATURES

FEATURES \ CONTROLLER	1	2	3	4	5	6	7	8
# of Preemption Sequences	5	4	5	5	5	5	6	3
Specific RR Sequence	No	No	Yes	Yes	No	Yes	Yes	No
Entry Delay	Yes	Yes	Yes	No	?	No	Yes	Yes
Locking, Non-Locking	Yes	Yes	?	No	?	No	Yes	?
Modify Minimum Green	Yes	Yes	No	No	Yes	Yes	Yes	?
Modify Ped Clearance	Yes	Yes	Yes	Yes	No	Yes	Yes	?
Modify Veh. Clearance	Yes	Yes	No	Yes	No	Yes	Yes	No
# of track clearances	1	1	1	2	2	2	1	1
Change Clearance at End	Yes	Yes	No	Yes	No	Yes	No	No
Separate Overlap Control	Yes	Yes	?	Yes	Yes	Yes	Yes	?
Change Overlap Clearance	No	No	?	No	No	No	Yes	?
Hold or Cycle	Hold	Hold	Cycle	Hold	Cycle	Cycle	Cycle	Hold
Modify Veh Clearance	Yes	Yes	No	No	No	No	No	No
Set Ped Activity	Yes	Yes	?	?	Yes	Yes	?	?
Exit Delay	No	No	No	Yes	No	Yes	Yes	No
Specify Exit Phase	Yes	Yes	Yes	Yes	Yes	Yes	Yes	No
Set Exit Phase Timings	No	No	No	No	Yes	No	Yes	No
Minimum Hold Time	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes

DOCUMENTATION

The review of the operations manuals from the various manufacturers revealed several inadequacies that deserve comment. First, there is a wide disparity among the different manuals in the amount of information they contain describing the preemption capabilities of the controller. Some manufacturers provide little descriptive information about their unit's capabilities, only direct programming instructions. This causes difficulty when evaluating the actual capabilities of the unit without having one to experiment with. Furthermore, some manufacturers include too much technical information about their controller's design. This information is appropriate, but should be separate from the capability descriptions and programming directions.

The operations manuals are often poorly organized and not well written. Occasional errors and inconsistencies were discovered, particularly in the example preemption sequences provided by many of the manufacturers. Overall, each manual has its good points and bad points, but all could use improvement in one or more areas.

The ideal manual would be one that offered a verbal description of controller capabilities followed by specific programming instructions. Technical and electronic information could be separated from operation instructions and placed in a separate specifications and repair manual.

CONCLUSIONS AND RECOMMENDATIONS

From the review of the railroad preemption capabilities of the sample NEMA traffic signal controllers, it is clear that there are some major differences among the controllers, and that the features present on the controller available at a given preemption installation will, to some extent, dictate what control strategies are possible.

Some of the features included on individual controllers are excellent and should be included on all controllers. Other features are inappropriate and the use of them would actually violate accepted national standards. The implementation of some features simply does not allow enough flexibility to create safe and efficient preemption designs. Flexibility is the key to preemption hardware because each installation will have its own unique requirements. A summary of the features of the surveyed controllers is presented in Table 1.

During the review of individual features, recommendations have been given as to the minimum operational capabilities that might be included in a controller design. In summary, the following general observations are offered:

- Several common controller features allow the user to violate accepted national standards,
- Several common features do not offer enough flexibility to permit efficient preemption design,
- Controller operation manuals could be improved, and
- The upcoming NEMA standard for preemption should be thoroughly reviewed by actual end users to ensure it promotes compliance to traffic engineering standards and flexibility in preemption design.

One aspect of controller hardware that was not addressed in this study is systems considerations. What happens at a preemption location when the intersection controller is part of a signal system? How does the controller drop out of and subsequently return to system operations? What are the preemption capabilities of proprietary closed loop systems? What preemption options are built into advanced traffic signal control systems? There is currently little information available to answer these questions.

Finally, further work needs to be done concerning the capabilities of the track circuit hardware as it relates to traffic signal preemption. Manufacturers could be contacted directly to obtain information regarding the operation of their products. This study would need to consider the many different types of track circuits in current use, ranging from the simple dc circuits to the newer constant warning time systems.

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Guidelines for the Use of Selected Active Traffic Control Devices at Railroad-Highway Grade Crossings

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Guidelines for selecting and installing active traffic control devices are beneficial to the practicing engineer who has responsibility for field installation and operation. This paper reports on a portion of the field installation and evaluation of two active traffic control devices for use at railroad-highway grade crossings. As a result, guidelines were developed for the use of a four-quadrant gate system and a highway traffic signal system for use at selected railroad-highway grade crossings. The characteristics of crossings that would be conducive to the use of a four-quadrant gate system and a highway traffic signal system were defined, with the objective of improving safety for the traveling public at the crossings. A four-quadrant gate system should be viewed as being between a standard gate system and a grade-separated crossing in terms of providing a level of safety to the traveling public. There are railroad-highway grade crossings that would not be economically feasible to grade separate, but a four-quadrant gate system would be cost-effective. Similarly, there are specific types of crossings that would receive a higher level of safety with the use of a highway traffic signal system and the upgrade would be cost-effective. The guidelines presented address the characteristics of the different types of crossings that would be appropriately served by these two active traffic control systems.

Historically, the engineering profession has assembled information that can be used to guide engineers in the deployment of traffic control devices for use on the highway system. These guidelines have aided the practicing engineer in selecting a particular type of device for a particular application. In addition, warrants are often developed that specify the conditions under which a particular type of device should be used. As an example, the *Manual on Uniform Traffic Control Devices* (MUTCD) provides both guidelines for certain types of devices such as overhead red-flashing beacons and warrants for such devices as stop signs (*1*). Although engineering judgment is essential for the selection and placement of any traffic control device, guidelines and warrants tend to aid an engineer in making a decision as to the type of device that should be used for a given situation. It is with the concept of providing guidance to the practicing engineer that guidelines have been developed for selecting traffic control devices at railroad-highway grade crossings.

Recognizing the need to fully address the issues and problems concerning warning devices at railroad-highway grade crossings, FHWA sponsored research to identify and evaluate innovative, active warning devices with potential for improving safety at these crossings. Through the research, innovative, active devices for use at railroad-highway grade crossings were identified and prototypes developed. The most promising active devices were evaluated in a detailed laboratory study (*2*), and the two devices chosen for field evaluation were

1. Four-quadrant gates with skirts and flashing light signals (see Figure 1), and
2. Highway traffic signals with white bar strobes in all red lenses (see Figure 2).

The field studies assessed the effects of these two traffic control devices on driver behavior and safety at typical grade crossings. In addition, other considerations important to the success and acceptance of these devices for general field use at railroad-highway grade crossings include hardware, installation, system operation, maintenance, and system power requirements.

From the field evaluation, these two devices proved to be technically feasible and practical, and were accepted and understood by the driving public. The cost effectiveness was shown to be extremely favorable for improving safety for motorists (*3–5*). Guidelines were developed for use of these two devices under various field conditions, to aid the practicing engineer in proper use of the devices as well as giving direction as to the conditions under which the two devices would be most cost effective.

According to the Federal Railroad Administration, during the period from 1977 through 1986, injuries and fatalities resulting from motor vehicle accidents at railroad-highway grade crossings decreased from 4,452 and 846 to 2,227 and 507, respectively. Much of this safety improvement may be attributed to the availability of federal funds for grade crossing improvement projects (*6*). The majority of this funding was used to upgrade passive crossings to active ones and has resulted in over one in four of the 192,454 public grade crossings being equipped with active warning devices. In 1986, there were 22,066 crossings (11.5 percent) equipped with automatic gates and 32,778 crossings (17.0 percent) equipped with flashing light signals (*7*).

Even with these improvements, over 50 percent of all car-train accidents in 1986 occurred at crossings with active warning devices, which represent only 28.5 percent of the total crossings (*7*). Thus, active crossings are overrepresented in

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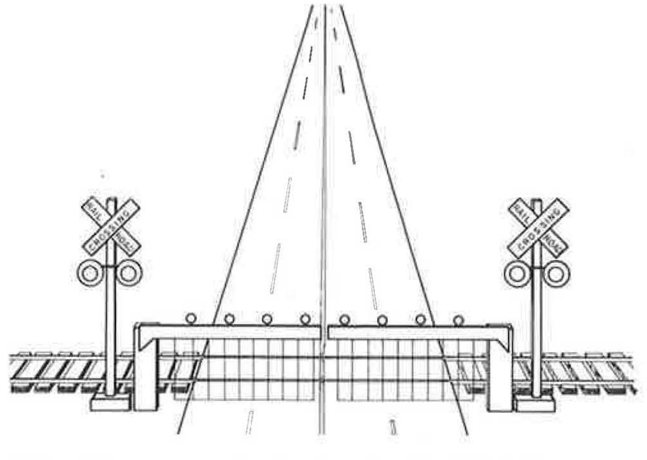

FIGURE 1 Four-quadrant gate system for field testing.

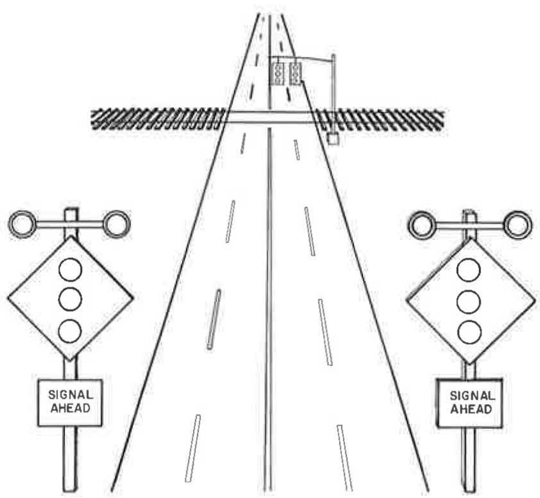

FIGURE 2 Highway traffic signal system for field testing.

terms of number of accidents. Although this apparently high number of accidents may be a result of higher vehicle and train volumes or more complex railroad-highway geometrics at active crossings, it is likely that some of the accidents are caused by motorists either not seeing or not understanding the active warning devices now used (*8*, *9*). Therefore, it seems that active traffic control devices used at crossings can be improved.

The following discussion outlines some of the results from the field testing and the considerations that should be given in choosing the locations for the use of the two devices: the four-quadrant gate system and the highway traffic signal system. These two active traffic control systems are designed to overcome some of the limitations of the existing active traffic control systems used at railroad-highway grade crossings.

FOUR-QUADRANT GATES WITH SKIRTS

The most effective device, in terms of driver response and safety, was the four-quadrant gate with skirts system (*3*, *4*). Based on the field test results (see Figure 3), the four-quadrant gate system outperformed the standard two-quadrant gate system on several key measures and proved to be operationally acceptable under a variety of conditions. This system substantially increased the safety of the crossing compared with the standard two-quadrant gate system based on the evaluation of the measures of effectiveness (MOEs). With the two-quadrant gate system, one or more motor vehicles drove around the closed gates during 84 out of every 100 train arrivals. The four-quadrant gate system reduced the number of gate violations (number of vehicles crossing) from an average of 260 per 100 train arrivals to 0. The four-quadrant gate system also reduced the CL20s (vehicles crossing less than 20 sec before arrival of train) from 60 per 100 train arrivals to 0, and reduced the CL10s (vehicles crossing less than 10 sec before arrival of a train) from 5 per 100 trains to 0.

The four-quadrant gate system did not significantly affect perception-brake reaction time (PBRT) or maximum deceleration levels at the test crossing. During the entire time that the system was in place, no motorists were trapped on the tracks. The system did not appear to increase the risk that a vehicle would be trapped on the tracks, provided the lowering of the far side gate arms was delayed by a few seconds to allow vehicle clearance. The four-quadrant gate system also did not interfere in any way with emergency vehicle operations at the test crossing during the field evaluation. (This would only be a problem for emergency vehicles if the equipment malfunctioned, and, at that point, the vehicle could break the gate arms if the situation warranted.)

In addition to the obvious safety benefits, four-quadrant gates with skirts are relatively easy to install, maintain, and operate, and they are reliable and durable. Worldwide experience with this gate system has been good.

The gates with skirts shown in Figure 3 may be considered a level of traffic control between standard two-quadrant gates and a grade-separated crossing. If standard two-quadrant gates do not provide the level of safety desired and a full grade separation is not economically attractive, then the four-quadrant gates with skirts should be the more cost-effective alternative.

Applications

Obviously, four-quadrant gates are very appropriate for those crossings that tend to have gate arm violations by motorists; the four-quadrant gates with skirts simply stop all violations by blocking the driving range around a gate arm. However, these gates can be used at any crossing where standard two-quadrant gates are warranted. Several types of crossings tend to have a large number of motorists driving around gate arms after they have been lowered. These crossings have certain unique characteristics that tend to result in violations and would be prime candidates for use of four-quadrant gates with skirts.

The characteristics of crossings listed below are good candidates for four-quadrant gates with skirts:

- Crossings on four-lane undivided roadways;
- Crossings with two or more tracks separated by a distance equal to or greater than the storage requirements for one or more motor vehicles;

FIGURE 3 Four-quadrant gate system installed at Cherry Street crossing in Knoxville, Tennessee.

- Crossings with large variations in train speeds and without constant warning time;
- Crossings for which motor vehicle-train collisions pose large potential safety problems such as
  (a) crossings with large numbers of hazardous materials trucks or trains carrying hazardous materials,
  (b) crossings with large numbers of school buses, and
  (c) crossings with high-speed passenger trains;
- Crossings with continuing accident occurrences; and
- Crossings with consistent gate arm violations.

Crossings with the listed characteristics are candidates for the use of four-quadrant gates with skirts, because motorists often desire to drive around gate arms at these crossings, or if an accident does occur, the consequences can be very severe. The four-quadrant gate system tends to indicate to a driver that the crossing is dangerous and that more than normal caution should be exercised. The following discussion reviews the rationale for each type of crossing as a candidate for four-quadrant gates with skirts.

FIGURE 4 Four-lane, undivided roadway crossing.

Crossings on Four-Lane Undivided Roadways

Although several characteristics of crossings tend to result in violations by motorists desiring to drive around the gate arms, crossing geometrics play an important role in permitting or creating a decision to violate gate arms. With crossings on four-lane undivided roadways, there is a sufficient amount of lateral space to permit a motor vehicle to go around a gate arm that only covers two of the four lanes (see Figure 4). If there is sufficient space for maneuvering a motor vehicle around a gate arm with relative ease, some motorists will violate a gate arm, particularly if the driver perceives a long waiting time before the arrival of a train.

Crossings with Two or More Tracks a Substantial Distance Apart

Crossings that have two or more tracks separated by a distance equal to or greater than the storage requirements for one or more motor vehicles result in some gate arm violations. A truck driving around a gate arm for multiple tracks separated by a substantial distance is shown in Figure 5. Field observations indicate that motorists will often pull around one gate arm and use the lateral space between the tracks to reassess whether there are other trains coming on the set of tracks they are now approaching. More violations are expected as the spacing between the tracks increases.

Crossings with Large Variations in Train Speeds and Without Constant Warning Time

There are crossings that have a large variation in train speeds, from slow-moving freight trains of 20 mph or less to high-speed passenger trains of 80 mph or more. When predictors are not used, obviously there is a substantial difference in the length of time that gate arms are down for the approaching

FIGURE 5 Tracks separated by sufficient distance to store motor vehicles.

FIGURE 6 Hazardous materials truck using crossing.

trains. Field observations seem to indicate that, in these types of situations, drivers have difficulty recognizing these varying speeds, i.e., if a driver frequently encounters a gate arm down for a long period of time at a crossing, he has a tendency not to wait for a long activation and will often drive around. Obviously, with fast-moving trains, this creates a severe safety hazard.

Crossings for Which Motor Vehicle-Train Collisions Pose Large Potential Safety Problems

There are crossings where the type of motor vehicles that use it create a potential for severe safety problems should a collision occur between a train and a motor vehicle. Additional safety measures are often necessary to minimize the potential for conflicts at these crossings. Four-quadrant gates with skirts could significantly improve safety at these crossings.

Hazardous Materials Trucks Hazardous materials trucks can pose a serious problem should a collision occur between one of those vehicles and a train as shown in Figure 6. There have been some very serious accidents of this nature in the United States in the last few years. Some of these resulted when gasoline tankers were driven around gate arms. The results were disastrous. Figure 7 shows the results of such a gasoline tanker-train accident. Seven fatalities resulted from this collision, and 19 motor vehicles were destroyed by the resulting fire. In addition, if a hazardous materials truck is stopped at a crossing and a motor vehicle-train collision occurs, the possibility of a secondary collision with the hazardous materials truck presents a serious safety problem. Thus, as the number of hazardous materials trucks using a crossing increases, this safety issue becomes more severe.

School Buses or Public Transportation Buses Crossings with a large number of school buses or public transportation buses pose certain safety problems (see Figure 8). Although it is very unlikely that a school or transit bus driver would ever drive around a gate arm and place school children or adult passengers in a serious safety situation, nevertheless a secondary collision from a hazardous materials truck collision with a train can cause serious safety problems. As the number of bus crossings increases, the magnitude of this safety issue increases.

FIGURE 7 Results of collision of hazardous materials truck and train.

FIGURE 8 School bus and transit bus using crossing.

High-Speed Passenger Trains Crossings with high-speed passenger trains pose certain safety problems due to the possibility of a train derailment as well as the speed of impact of the train with a motor vehicle. Obviously the derailment of a passenger train has the potential for creating a large number of personal injuries and fatalities. Preventing a motor vehicle from moving onto the tracks in front of a high-speed passenger train is highly desirable. In situations where the crossing characteristics result in a desire to drive around a gate arm, four-quadrant gates with skirts will be very effective.

Continuing Accident Occurrences

Continuing accident occurrences at crossings with two-quadrant gates tend to indicate that the standard gate system is not performing as intended. This can be due to a number of reasons, some of which are not necessarily due to motorists who drive around the gate arm. However, when accidents continually occur, using four-quadrant gates with skirts to improve the safety of the crossing if a grade separation is not economically feasible should be considered. The target value of a four-quadrant gate system with skirts is substantially increased over that of a two-quadrant gate system.

Crossings with Consistent Gate Arm Violations

Crossings with consistent gate arm violations, which do not meet one of the preceding situations, also pose a continuing hazardous situation for the traveling public (see Figure 9). There seem to be some crossings that have an abnormally high number of drivers going around gate arms. In these situations four-quadrant gates with skirts will simply eliminate the violations.

Hardware Considerations

With the exception of the gate arms and skirts, all of the hardware and equipment used in the four-quadrant gates with skirts are standard parts, commercially available from several suppliers. Furthermore, the hardware and equipment are the same as those used in standard two-quadrant gates; thus, field crews are familiar with their installation, operation, and maintenance.

A delay relay should be installed in the gate control system in order to stagger the operation of the near- and far-side gate arms. Also, due to the added weight of the arm and skirt assembly, more counterweights will be required on the panarms. This added weight causes no problem in system operation.

To minimize unnecessary or lengthy gate activations, motion sensors or constant warning time train detectors should be installed at crossings where there are switching operations or large variations in train speed. These sensors and detectors will minimize the time during which the gates block the crossing.

The innovative gate arms with skirts, made from kiln-dried redwood, performed successfully and proved that the concept was not only technically feasible but practical and economically feasible.

One point to raise concerning the gate arms and skirts is whether the skirts are cost-effective. The field experience suggests that four-quadrant gates alone may greatly enhance driver performance and safety, and that the additional benefits of skirts may be minimal. The addition of skirts certainly complicates device construction, installation, and maintenance, and increases the cost of a four-quadrant gate installation; however, it enhances visibility considerably, especially at night. Where the geometrics of the approaches are complex and a larger target value is required at the crossing, skirts readily enhance the target value of the gate arms.

FIGURE 9 Multiple large trucks driving around gate arms.

Installation Considerations

Four-quadrant gates with skirts can be installed by regular field personnel within the normal scope of their duties and union contracts. No additional personnel training is required, nor are any special equipment, vehicles, or tools needed beyond those required for the normal installation of a gate system.

The procedures to install four-quadrant gates with skirts are basically the same as those used for standard two-quadrant gates, except for the following special requirements and concerns:

- Due to the increased weight of the skirt and gate arm, additional counterweights may need to be added to the panarms compared with those required for a standard gate arm. This additional counterweight will not affect the operation of the mechanism.
- When the gate arm and skirt are lowered and stopped in the horizontal position, there is a tendency for the unit to bounce or rock up and down a few times. To prevent the bottom of the skirt from striking the pavement during this bouncing, there should be 3 to 4 in. of clearance between the bottom of the skirt and the roadway.

System Operation and Maintenance

It is important that the gate arms be of sufficient length to completely block the roadway. If an opening of just a few feet is left between opposing gate arms, motorcyclists and bicyclists may try to cross in front of a train.

There should be a time delay between the operation of the near- and far-side gates. That is, the near-side gate should start down first, with the far-side gate descent delayed by 5 to 7 sec. The actual delay time is based on vehicle lengths, crossing width, and vehicle operating speeds. The delay is achieved by installing a delay relay in the controller and adjusting the circuit resistance as appropriate.

Three red lights should be used on each gate arm. Thus, a total of six gate lights across the roadway on each side of the crossing would be used. The two outside lights should be operated in the flashing mode, while the four interior lights should be steady-burn lights.

The type of maintenance for four-quadrant gates with skirts is essentially the same as for standard two-quadrant gates.

Power Requirements

The system contains two more gate mechanisms and six more gate lights; thus it uses approximately 50 percent more power. The additional weight of the gate arms and skirts does not increase energy consumption significantly because this weight is accommodated by adding counterweights to the panarms.

Environmental Considerations

The experimental gate arms with skirts were subjected to a variety of environmental conditions. They performed well in high winds and heavy rains, and under snow and ice conditions. They did not swing or sway excessively, nor did they bind up, freeze, or snag. Also, the gates and skirts were essentially self-cleaning from rain.

Emergency Vehicles

Emergency vehicles need to be considered in implementing four-quadrant gates with skirts, particularly at crossings near hospitals and fire stations, or on routes frequented by emergency vehicles. Some ideas and issues regarding emergency vehicle handling are presented below:

- All affected service agencies should be informed in advance of alternate routes and what to do if a malfunction does occur during an emergency run.
- Gate arms that could be raised or rotated out of the way by emergency personnel either manually or electronically could be installed at crossings frequented by emergency vehicles. Also, the far-side gates could be designed to raise automatically if down for more than a specified period of time.
- The four-quadrant gates with skirts could simply not be considered for use at crossings frequented by emergency vehicles and where a suitable alternate route is not available.

It should be remembered that four-quadrant gates would only be a problem for emergency vehicles if the equipment malfunctioned. Obviously, if the gate arms are down because of a train approaching or on the crossing, the vehicle should not proceed. Thus, if malfunctions occur infrequently, four-quadrant gates with skirts should not pose any problems. If a malfunction does occur and a train is not approaching the crossing, an emergency vehicle could simply break the gate arm if the situation warranted.

HIGHWAY TRAFFIC SIGNALS

Driver response to the enhanced highway traffic signals was excellent (*3,5*). The field installation is shown in Figure 10. These signals proved to be both feasible and effective and performed better than standard flashing light signals in reducing the number of motorists that crossed less than 10 and 20 sec in front of an approaching train when predictors were used on both systems. In addition, the violation rate was low. In fact, the highway traffic signals performed similar to standard short-arm gates in discouraging unacceptable track crossings. Compared with flashing light signals with predictors, the highway traffic signal reduced the number of crossings per signal activation from 3.35 to 0.73, and reduced the risky behavior per train arrival from 0.13 to 0.05. ("Risky behavior" refers to the number of vehicles crossing while the flashing light signals are activated and within 10 sec of the train.) Furthermore, the highway traffic signals proved to be less expensive than flashing light signals and much cheaper than short-arm gates. These results suggest that enhanced highway traffic signals do indeed have application to railroad-highway grade crossings. In fact, study results indicate that highway traffic signals would actually improve crossing safety over that afforded by standard flashing light signals and at a reduced overall cost.

Applications

Study results further indicate that, with appropriate revisions to the MUTCD, enhanced highway traffic signals could be used at any crossing where flashing light signals are warranted.

FIGURE 10 Highway traffic signal system installed at Cedar Drive crossing in Knoxville, Tennessee.

Highway traffic signals have a high level of driver credibility and respect because they have been used prudently and have been well-operated and maintained in the vast majority of cases. If highway traffic signals are to be successful at railroad-highway grade crossings, and thus not compromise driver credibility for highway traffic signals in general, then the same high standards of operation and maintenance must be obtained at crossings as at highway intersections. In particular, highway traffic signals should not be considered at crossings where false activations or malfunctions are common. They also should not be used at crossings where the train warning or occupancy times are consistently unreasonably long, i.e., more than 60 sec.

Some crossing situations where highway traffic signals would regularly afford advantages over conventional flashing light signals are identified below:

- Crossings in the vicinity of a signalized intersection or in the middle of a system of signalized intersections, and
- Crossings with complex highway geometrics where drivers are unable to make proper judgments on whether it is safe to proceed across the tracks and where gates would be impractical.

Crossings in Area of Signalized Intersections

Motorists using a crossing that is located in the area of a number of signalized highway intersections are responding with regularity to standard highway traffic signals. To change to a new type of activated traffic control device, generally found nonactivated, requires some adjustments for a motorist from a human factors point of view. Increased perception-reaction times can occur for motorists in these situations through receiving a different stimulus for processing. In providing a repetitive environment for a motorist, there is merit in continuing to provide a standard highway traffic signal system network across a fairly large area to reduce the number of new or different stimuli given to motorists. Figure 11 shows an application of this concept in Denver, Colorado, and Figure 12 shows an application in Knoxville, Tennessee.

Complex Geometrics at Crossings

Traffic encountering complex highway geometrics at crossings is difficult to control with standard railroad active traffic control devices such as flashing light signals or gates. Complex highway geometrics create complex driving maneuvers on the part of motorists. Channelization of motorists becomes critical to ensure appropriate movement of motor vehicles in these areas. In addition, perception-reaction times can be significantly increased for motorists when encountering confusing geometrics or a complexity of active traffic control devices. Complex geometric multileg crossings are difficult to actively control with flashing light signals or gates. However, highway traffic signals, through the use of protected turning movements as well as arrows for directional movement and guidance, can be effective active traffic control devices at these types of crossings. Figure 13 shows an application of this concept in Oklahoma City, Oklahoma, and Figure 14 shows an application in Knoxville, Tennessee. Other complex geometrics can result from limited sight distances, grades on the approaches, alignment, as well as other factors. Highway traffic signals have a unique ability to provide positive guidance to a driver in negotiating complex geometrics of the highway system and thus increase the level of safety.

Hardware Considerations

Except for the Barlo strobe lights in the red signal lenses, all of the hardware used is standard, off-the-shelf highway traffic

FIGURE 11 Highway traffic signals used at a crossing in Denver, Colorado.

FIGURE 12 Highway traffic signals used at a crossing in Knoxville, Tennessee.

FIGURE 13 Highway traffic signals at crossing with complex roadway geometrics in Oklahoma City, Oklahoma.

FIGURE 14 Highway traffic signals at crossing with complex roadway geometrics in Knoxville, Tennessee.

signal equipment available from numerous suppliers in all parts of the country. This includes the signal poles and foundations, mast arms, signal heads, mounting hardware, wiring, controller, and advance sign or flashing beacon units. The ready availability of this hardware and the competitive price market certainly are advantages.

The Barlo lights are currently available only from one source, and production levels are low. Should the enhanced highway traffic signals be adopted for use, it is expected that the current supplier could meet demands at prices comparable to existing active device prices. Other manufacturers would also be expected to enter the market depending on patent restrictions.

Any type of signal controller can be used as long as it is capable of providing a three-part (red, yellow, and green), variable length cycle, along with a flashing red mode. Also, it is desirable to fully unify the signal controller with the train detection controller, placing them in the same cabinet and providing a unified power system.

Installation Considerations

Railroads have the experienced labor needed to install highway traffic signals. The alternative of using highway traffic signal contractors would also be available and, if labor union problems could be resolved, the total cost of installation should be significantly less.

No additional right-of-way or space (above or below ground) is needed for a highway traffic signal compared to a flashing light signal. However, if advance flashing beacons are used, some additional space along the roadway right-of-way may be needed for these devices. The installation of the beacons will generally be handled by the highway agency which would require some additional coordination.

Power Considerations

The enhanced highway traffic signal is powered directly by 120-volt commercial power. This power permits the use of higher wattage lamps (compared to flashing light signals). The higher wattage lamps are bright over a wide angle; thus alignment is not critical as with flashing light signals.

For the field studies, a propane generator was used to provide backup power for the highway traffic signals in the event of a commercial power failure. (Backup power for the train detection system was provided by conventional 12-volt batteries.) The propane generator was capable of powering the traffic signal for 24 hr or more. The generator performed without incident during the months of testing.

Power backup may not be necessary for a highway traffic signal installation since, unlike flashing light signals and gates, a traffic signal has a built-in fail-safe mode. When power is lost, due to a commercial power failure or malfunction, the signal indications go blank. A blank signal, in turn, warns motorists that there is a problem and that conflicts with opposing traffic are likely. Experience with conventional highway traffic signals indicates that drivers will be extremely cautious under these circumstances. Backup generators are not known to be used in the illustrations shown above.

It may be appropriate to define a fail-safe mode as a flashing red for standard highway traffic signals used at a railroad-highway crossing. This mode would not be difficult to achieve with a standard battery system used with standard active control devices. The highway traffic signal should be operated regularly on 120-volt AC power supply. However, should there be a power failure, a simple relay could be used to switch from the 120-volt AC power supply to the battery source to operate only a flashing red light by DC current. Without increasing the existing capability in standard battery installations at crossings, a flashing red mode could be maintained for a sufficient time to cover all but the most extensive power outages caused by storms. The increased safety benefits from the use of highway traffic signals should far outweigh any safety problems caused by power failures from a major storm.

Warning Time and Train Detection

The enhanced highway traffic signals can be easily and economically installed at crossings equipped with flashing light signals. However, for such retrofit installations (and for all new installations), consideration must be given to providing reasonable, uniform train warning times. Warning times (the time that the signal is yellow and then red before the train arrives at the crossing) will depend on the variability in approach train speeds and the type of train detection equipment. Reasonable and uniform warning times are essential to the successful operation of the enhanced highway traffic signals.

Experience suggests that most motorists will stop and wait for a red traffic signal for up to 60 sec, even if there is no opposing traffic in sight. This is true at signalized highway intersections and was also observed at the crossing test site. If the wait time exceeds 60 sec (particularly if there is no opposing traffic), the highway traffic signal may lose credibility for the motorist and violations are likely to occur.

At crossings with variable train speeds, it is desirable to employ constant warning time train detectors to provide warning times in the range of 20 to 30 sec. Constant warning time detectors should not be needed at crossings with uniform train speeds, because the speeds should result in uniform warning times. Highway traffic signals will normally outperform flashing light signals in terms of reducing the number of motor vehicles going over the crossing after the signals are activated, even when both systems have constant warning times.

Traffic Signal Operation and Timing

The highway traffic signals should rest in green until the approach of a train is recognized by the train detectors. When the train is approximately 20 sec from the crossing, the signal should turn yellow and then red. The signal should remain red, with the white bar strobes flashing, until the train is past the crossing.

The length of the yellow vehicle change interval should be 3–6 sec, depending on approach traffic speeds. Recommendations for setting yellow times for highway intersections are presented in the MUTCD and *Traffic Engineering Handbook*, and these guidelines are applicable to grade crossing highway traffic signal installations (*1*, *10*).

A minimum warning time of 20 sec is more than enough to provide adequate train-car separation. In fact, a lesser warning time might minimize motorist delay, uncertainty, and violations, while still providing adequate train-car clearances. This time may be increased where conditions of vehicle length, acceleration characteristics, grades, number of tracks, or other factors dictate.

It must be recognized that hardware malfunctions (namely, false signal activations) are unavoidable. Furthermore, it would severely damage the credibility of a highway traffic signal installation at a grade crossing if the signal remained red during a lengthy malfunction period. Thus, it is desirable to have the signal change indications in the event of a malfunction. With standard signal equipment and controllers, the most practical way to accommodate false activations is to have the signal change to a flashing red indication after a sufficiently long period (long enough to know that the activation is not due to a slow train). A time of 3 min may be acceptable for most installations. This time should be based on specific conditions at the crossing such as train speeds and train lengths.

The highway traffic signal system installed in the field (shown in Figure 10) did not have crossbuck signs, advance warning signs, or advance pavement markings as a part of the traffic controls. The system worked extremely well and, thus, motorists treated the crossing as they would a signalized intersection. The intent was to have a motorist respond to the traffic control device rather than to whether or not a train is presumed to be approaching a crossing. It is recommended that all railroad warning signs (including the crossbucks and advance warning signs) should be eliminated. In their place, intersection stop bars and signal ahead signs with flashing beacons should be installed on the crossing approaches. Stop bars are essential, since the normal intersection cues are not present at a railroad grade crossing. In fact, STOP HERE ON RED signs may be used to supplement the stop bars.

Maintenance Considerations

Highway traffic signal installations require similar maintenance as a standard flashing light signal system. However, flashing light signals, as opposed to highway traffic signals, do require sighting. Maintenance of highway traffic signals could be handled by railroad signal maintainers with little additional training. Typical maintenance needs include the following:

1. The signal lamps must be changed and the lenses cleaned periodically,
2. Routine service checks on wiring and the controller are recommended, and
3. Periodically pavement markings must be replaced and the signs should be cleaned.

SUMMARY

The implementation considerations presented in this paper have been developed through field experience gained from research, consultations with the traffic engineering community, as well as many years of crossing safety experience by project staff. As these systems are implemented and are placed under additional field conditions, it is recognized that modifications may be needed. However, these guidelines will promote successful installation and operation of the two systems.

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DISCUSSION

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In the section entitled "Four-Quadrant Gates with Skirts," the statement is made in the second paragraph, "The four-quadrant gate system also did not interfere in any way with emergency vehicle operations at the test crossing during the field evaluation." This statement addresses the universal concern that four-quadrant gates will block the passage of emergency vehicles when falsely actuated—a potentially life-threatening situation. Some reference should be inserted at this point to the qualifying comments in the paper in the subsection entitled "Emergency Vehicles."

In the subsection "Applications," "Crossings with large variations in train speeds and without constant warning time" are listed as candidates for the installation of four-quadrant gates. In the stated situation, any problem would undoubtedly be due to the absence of constant warning time (CWT) track circuitry and the installation of this circuitry would be the primary solution. Installation of four-quadrant gates in this situation would aggravate the delay to motor vehicles and would increase the likelihood of gate violation. Gate installation should not be mandated before CWT track circuitry is installed and tested. These comments apply equally to the subsection entitled "Crossings with Large Variations in Train Speeds and Without Constant Warning Time" and the subsection entitled "Crossings with Consistent Gate Arm Violations."

In the introductory paragraph to the section entitled "Highway Traffic Signals," the first sentence reads, "Driver response to the enhanced highway traffic signals was excellent." The discussant contends that the observed motorist response is the result of the novelty of this installation at a railroad-highway grade crossing and the conditioned response of the motorist to the traffic signal at the intersection of highways and not to any inherent superiority of the traffic signal display over the standard railroad flashing signal in controlling vehicular traffic at railroad-highway grade crossings.

The results obtained in this research are analogous to the results of the early research conducted with yellow and red Stop signs. However, after the red Stop sign was standardized and had been in use for some years, further research revealed that the driver response to the red Stop sign was practically identical to his response to the yellow Stop sign in the "before" condition of the early research, an example of the favorable but temporary effect of novelty.

The highway traffic signal is a continuously active device cycling on the average of once every minute and alternately assigning the right-of-way to intersecting flows of motor vehicles. It has functioned in this way and for this purpose since its inception and the motorist is conditioned to its meaning and his response.

The railroad grade crossing signal is an intermittent device cycling on the average of a few times a day that reaffirms the assignment of the right-of-way to the railroad and warns of the approach of the train. It, too, has functioned in this way and for this purpose since its inception and the motorist is conditioned to its meaning and his response.

In the opinion of the discussant, the use of the highway traffic signal at railroad-highway grade crossings would require the motorist to ascribe different meanings to the same device. The process of determining the proper response to the traffic signal before him must increase his perception-reaction time and ultimately, will be detrimental to his safety.

The highway traffic signals shown at the railroad-highway grade crossings in Figures 12 and 14 and discussed in the text are the proper display at these locations. The railroad tracks cross through the middle of the intersecting highways that operate full time under traffic signal control. Upon its approach, the train preempts the traffic signal that remains all red during the passage of the train. The successful use of railroad preemption at a traffic signal controlled highway intersection does not imply that highway traffic signals are the preferred device for the control of railroad-highway grade crossings.

AUTHORS' CLOSURE

We would like to thank Mr. Williams for commenting on this paper and we will attempt to respond to the questions raised.

Mr. Williams's comments on improving the track circuitry to ensure constant warning time as opposed to installing four-quadrant gates has some merit. However, when the improvements to the track circuitry cannot be achieved, installing four-quadrant gates would be better than having a large number of gate violations. Mr. Williams is incorrect in saying that the "installation of a four-quadrant gate system under these circumstances would increase the likelihood of gate violation." With the four-quadrant gate system, there will be no gate violations.

Mr. Williams may be correct in his comment that the response to the highway traffic signal could be due to its novelty at a railroad-highway grade crossing. However, we do not believe that this is the case. There has not been enough research conducted to conclude that the response is due only to a novelty effect. We believe that the response is due to the fact that a motorist has to respond to highway traffic signals frequently and, therefore, is conditioned to do so regardless of the location of the highway traffic signal.

The highway traffic signal is not necessarily a continuously active device cycling on the average of once every minute. This occurs only for a fixed-time signal. At intersections where only the minor roadway is traffic actuated, the amount of green time can be extremely long on the major thruway. Thus, motorists do encounter all types of cycling of the highway traffic signal including that found at a railroad-highway grade crossing.

From our research as well as other research, it is questionable whether a motorist is conditioned to the meaning of a flashing-light signal at a railroad-highway grade crossing. From our more than 25 years working in the highway safety field, we believe that many motorists do not fully understand what is required of them at a flashing-light signal. In fact, a flashing red light generally means stop and proceed with caution. This is also true for a railroad-highway grade crossing even though that may not be the correct driver response for a given situation.

At the beginning of our research project, we tended to agree with Mr. Williams in opposing the use of highway traffic signals at railroad-highway grade crossings. But after seeing the data and completing the analyses, we believe that there is merit in applying highway traffic signals to railroad-highway grade crossings. In more than 25 years in highway safety research, we have become convinced that the objective should be to minimize the number of traffic signals and signs to the extent possible. As the number of stimuli that a driver must respond to increases, the probability of error on the part of the driver also increases.

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Evaluation of Constant Warning Times Using Train Predictors at a Grade Crossing with Flashing Light Signals

STEPHEN H. RICHARDS, K. W. HEATHINGTON, AND DANIEL B. FAMBRO

This paper documents the results of field studies conducted to evaluate the effects of train predictors and constant warning time (CWT) on crossing safety and driver response measures. The studies were conducted at a single-track urban crossing controlled by flashing light signals. The test crossing is frequented by variable-speed trains. Before train predictors were installed, highly variable and long warning times were observed. The studies involved comparing data gathered before and after installation of train predictors at the test crossing. The data included warning times, vehicle clearance times (relative to a train's arrival), vehicles crossing, and vehicle speed and deceleration profiles. These data were collected using video camera-recorder systems that were activated automatically whenever a train approached the test crossing. Data were collected for a 2-month period before the train predictors were installed, and for a 2-month period after installation. A total of 139 train movements were observed—89 train movements during the before study and 50 movements during the after study. On the basis of the results of the field studies, the predictor hardware proved to be operationally reliable. Installation of the predictors resulted in more CWTs, a lower mean warning time, and fewer excessively long warning times at the study crossing. Installation of predictors (and the CWT they provide) also improved the overall safety of the study crossing and enhanced driver respect for the flashing light signals. Vehicle clearance times were significantly increased, and risky driver behavior was reduced. Speeds, driver reaction times, and deceleration levels were not influenced adversely.

Since 1973, over $2.3 billion in federal and state funds have been spent to improve railroad-highway grade crossing safety (*1*). Most of these funds have been used to install or upgrade active warning devices, i.e., flashing light signals with or without automatic gates. By 1986, 17 percent of the nation's 205,339 public crossings were equipped with flashing light signals and over 9 percent had flashing light signals with automatic gates (*1*). As illustrated by the reduction in grade crossing accident casualties, the increased use of these active devices has undoubtedly enhanced grade crossing safety. In 1985, 537 motorists and pedestrians were killed in train accidents, compared to a high of 1,780 fatalities in 1966 (*1*).

Notwithstanding the obvious safety benefits of flashing light signals with or without automatic gates, there is increasing concern about the length of the warning time period for these active devices. (Warning time refers to the time between device activation and arrival of a train at the crossing.) Specifically, research (and good common sense) suggests that variable and excessively long warning times may have negative impacts on crossing safety and traffic operations. For example, Hopkins (*2*) reports that frequent users of a crossing become aware that signals flash too long in advance of a train's arrival and proceed through the crossing when the warning device is activated. In a study of crossing accidents (*3*), the predominate contributing factor was excessive warning time. Long warning times resulted in drivers' disregarding the hazard and proceeding across the track in front of an approaching train. A driver behavior study (*4*) also found problems with excessive warning times. It was concluded that by eliminating unnecessarily long warning times and false activations, the rate of disobedience towards crossing signals would be reduced, thus reducing train-involved accidents at active crossings.

Guidelines for warning times at active crossings are presented in the *Manual on Uniform Traffic Control Devices* (MUTCD) (*5*). The MUTCD states only that reasonably constant notice must be provided; it does not specify any maximum warning time for active crossings. At crossings with flashing light signals, the MUTCD does specify a minimum warning time of 20 sec. As a result of these somewhat vague guidelines, warning times vary greatly at active crossings. Sanders (*6*) observed warning times ranging from 5 to over 300 sec at a group of crossings. Heathington et al. (*7*) reported a range in warning times at three study crossings of 14–161 sec.

Warning times are usually controlled by the type of train detection system at a crossing. With standard train detection circuitry, the warning time depends on train speed and the fixed location of the track circuitry relative to the crossing. Therefore, if a crossing with standard detection circuitry has variable-speed trains or switching operations, warning times can be highly variable and excessively long. Likewise, motion sensors, a second type of train detector, cannot provide constant warning time (CWT) if variable-speed trains are present; however, a motion sensor can eliminate the excessive warning times resulting from many switching operations.

The third type of train detector, called a train predictor, can provide a fixed CWT, even at crossings with variable-speed trains or switching operations. Train predictors have been installed at over 6,300 active crossings in the United States, and it is estimated that an additional 13,100 crossings could benefit from this more sophisticated (but more expensive) type of detector (*8*).

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The concept of providing reasonable and consistent warning times at active crossings is well accepted by most groups involved with grade crossing safety. However, the implementation of this concept has been somewhat slow and haphazard, possibly due to the lack of substantive data on the effectiveness and benefits of train predictors and CWT. That is, there is a need to show that consistent and reasonable warning times do significantly enhance safety and traffic operations at active crossings, and that the added expense of train predictors at crossings with variable-speed trains is justified.

In order to more fully evaluate the impacts of train predictors and CWT on crossing safety and driver behavior, a series of field studies was conducted at a single-track, urban crossing in Knoxville, Tenn. (*7*). This crossing, which is controlled by flashing light signals, is frequented by variable-speed trains and switching operations. The studies involved comparing safety and performance data gathered before and after installation of train predictors (and CWT) at the crossing. The following sections describe the study approach and research findings.

FIELD EVALUATION PLAN

Study Approach

A before-and-after study approach was used to evaluate the impacts of train predictors and CWT on driver behavior and safety. That is, performance data were collected at an existing active crossing with conventional detectors, and then again at the same crossing after predictors had been installed. This approach allowed a direct comparison between conventional detectors (which can result in variable and sometimes very long warning times) and train predictors (which provide a reasonable CWT).

The before set of crossing studies (before predictor installation) was conducted in May and June of 1985; predictors were installed in November 1985; the after set of studies was conducted in February and March 1986. The purpose of the 2-month delay following predictor installation was to ensure that drivers had some time to become familiar with the change in warning time conditions at the crossing.

Study Site

The site of the studies was an active crossing (Inventory No. 730643K) in Knoxville, Tenn., located in the northern part of the city on Cedar Drive. The existing active warning devices at the crossing were standard railroad flashing light signals with 8⅜-in. roundels and a bell. The crossing was ranked as the 31st most dangerous crossing in the state in 1985. As shown in Figure 1, Cedar Drive in the vicinity of the crossing is two lanes wide and straight on both approaches to the crossing. The vertical alignment on the westbound approach limits the motorists' line of sight to the crossing. In addition, the thick vegetation in the vicinity of the crossing restricts the drivers' view of approaching trains. The average daily traffic at this site is approximately 14,000 veh/day, and the average through train volume is approximately 10 trains per day. The speed limit on Cedar Drive is 40 mph, and train speeds at the crossing range from 5 to 40 mph. As shown by its hazard ranking and the three car-train accidents that occurred at this site in the past 5 years, this is a hazardous location.

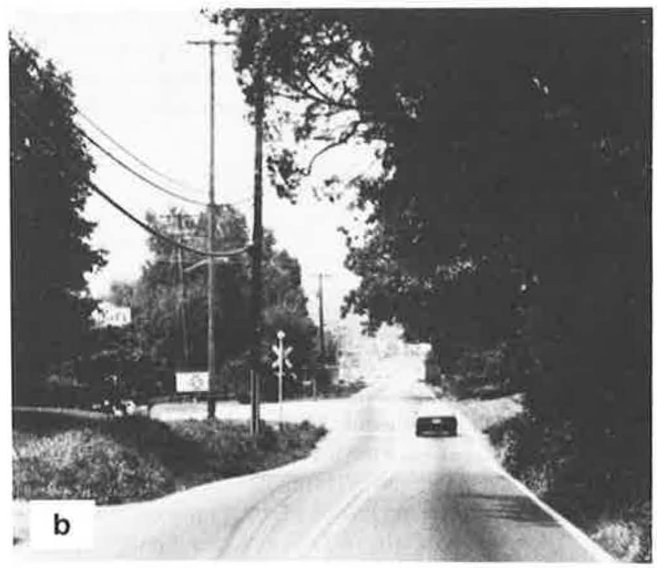


FIGURE 1 Cedar Drive crossing: a, looking east; b, looking west.

Data Collection and Reduction

The key to determining motorist response to the predictors and CWT was to obtain accurate and pertinent data on driver behavior at the crossing and in the decision zone, i.e., the area in which drivers must decide to either stop or proceed through the crossing. Data were automatically recorded on portable video recorders whenever a train was approaching the crossing and reduced by an image processing and pattern recognition process.

Three complete video camera-recorder systems were used for data collection. The video recorders were portable, battery powered, and used standard ½-in. T120 VHS cassettes. The video cameras used with the recorders were black-and-white, closed-circuit television cameras that provided high-quality videotapes under both day and night lighting conditions. The cameras operated on 12-volt DC current and used the recorder batteries as a power source; therefore, they were only energized when the recorders were activated.

Each camera was mounted on a 20-ft pole and located as far from the centerline of the roadway as possible (approximately 60 ft). The first camera-recorder unit was located approximately 300 ft from the crossing, the second approximately 500 ft from the crossing, and the third approximately 700 ft from the crossing. The cameras were aimed towards the crossing, and had overlapping fields of view.

It was important to activate the video camera-recorder systems just before activation of the flashing light signals so that driver response to the signals could be fully evaluated. For this reason, a special pole-mounted train detector system separate from the regular track circuitry was developed and used. The detector system projected an infrared light beam across the track. When a train broke the beam, the detector transmitted an audio (FM radio) signal that activated the camera-recorders. A detector was placed on each approach to the crossing, such that the camera-recorder activation signal was transmitted at least 10 sec before a train's activating the flashing light signals at the crossing.

Measures of Effectiveness

Evaluation of the train predictors and CWT depended on the selection of suitable measures of effectiveness (MOEs). To avoid influencing drivers' behavior, MOEs were selected that could be obtained with a minimum of interference and detection by drivers. One obvious MOE used was warning time (the time elapsed between device activation and train arrival). In addition, several driver performance measures were used to evaluate the safety impacts of warning time and train predictors. These safety-related MOEs included number of vehicles crossing, clearance time, perception-brake reaction time (PBRT), and speed profile and maximum deceleration level, as described in the following sections.

1. *Number of Vehicles Crossing.* This measure was defined as the total number of vehicles crossing the tracks between activation of the warning device and the train's arrival at the crossing. The total number of vehicles crossing was manually counted from the videotapes, and the numbers of vehicles crossing within 10 and 20 sec of the train's arrival at the crossing were specially noted. Vehicles that crossed within 10 sec of an oncoming train (called "CL10s") were considered an indication of risky behavior, because this represents a level of driver performance in which there is little, if any, room for error. This representation was based on 2.5 sec of PBRT, a 20-ft long vehicle starting from a stop 20 ft away from the crossing, accelerating at a normal rate of 4.8 ft/sec^2, and clearing a point 20 ft on the far side of the crossing 2.5 sec before the train's arrival. Vehicles that crossed within 20 sec of an oncoming train (called "CL20s") were considered indicative of aggressive behavior, representing a level of driver performance in which there is some, but not much, room for driver, vehicle, and warning system error. The MUTCD appears to address this point by requiring a minimum warning time of 20 sec (*5*).

2. *Clearance Time.* Clearance time was defined as the difference in time between the time of the last vehicle's crossing and that of the train's arrival.

3. *Perception-Brake Reaction Time.* PBRT was defined as the difference in time between activation of the warning device and activation of the vehicle's brake lights. Only those vehicles whose brake lights were activated were included in the data set. As the observations were not necessarily expected to be normally distributed, nonparametric techniques in the Statistical Analysis Systems program were used to ascertain whether or not observed differences were statistically significant (*9*).

4. *Speed Profile and Maximum Deceleration Rate.* Speed profile data gathered before and after installation of the predictors were evaluated and compared. In addition, a maximum deceleration level was computed from each individual speed profile. These values were then tabulated and plotted as a cumulative frequency distribution. The number of drivers accepting an undesirable level of deceleration (>8 ft/sec^2) was also used for evaluation purposes. In each of the previously described comparisons, the Kolmogorov-Smirnov goodness-of-fit test was used to determine whether any observed differences in distributions were statistically significant (*10*).

The general hypotheses tested in the field studies were that use of the predictors, when compared with conventional train detectors, would result in (1) more consistent warning times and fewer excessive warning times; (2) fewer vehicles crossing in front of the train; (3) fewer undesirable and uncomfortable decelerations; and (4) quicker driver PBRTs. Thus, the overall null hypothesis was that there were no differences in driver performance measures resulting from the installation of train predictors and CWT at the test crossing.

STUDY RESULTS

The results are reported as two studies—a before study (flashing light signals without predictors) and an after study (flashing light signals with predictors). Combining both of these studies, 139 train movements were observed. There were 89 train movements observed in the before study and 50 train movements observed in the after study. For each train movement, the environmental and lighting conditions, train's direction of travel and warning time, number of vehicles crossing, and approaching vehicle's clearance time, speed profile, and PBRT were recorded and subsequently analyzed.

Warning Time

Warning time was defined as the time duration between activation of the flashing light signals and a train's arrival at the crossing. It is the same as the maximum amount of time a motorist would have to wait between activation of the warning devices and the train's arrival at the crossing. It was expected that the installation of the predictors at the Cedar Drive crossing would result in shorter and more consistent warning times.

To verify these premises, the total data set from both studies was subdivided into day and night to ensure that similar train and traffic volume conditions were compared. These two subsets, together with the total data set, were then analyzed. As presented in Table 1, the mean warning time in the before study was significantly longer than in the after study. The mean warning time in the before study was 75.2 sec compared with 41.7 sec in the after study. The Kruskal-Wallis test for two or more independent, continuously distributed populations (*10*) indicated that these differences were statistically significant at the 99 percent confidence level. This result means that, as expected, installation of the predictors decreased the average warning time at the crossing. This finding is shown clearly in the illustration of the frequency and cumulative frequency distributions of the warning times from the two data sets shown in Figure 2. In addition to the before-and-after study results, the Mann-Whitney U test indicated that there was no statistically significant difference at the 95 percent level between the day and night data sets from the two studies.

It should also be noted from Table 1 that, even after predictors were installed, a few very long warning times were observed at the crossing. This was due to the fact that there was a siding track just a few hundred feet north of the Cedar Drive crossing, and predictors were not installed on the siding. As a result, slowly moving southbound trains coming off the siding produced the longer warning times, i.e., these trains activated the signals while still on the siding.

Vehicles Crossing

Average numbers of vehicles crossing in the interval between activation of the flashing light signals and a train's arrival at the crossing are presented in Table 2. As there was a statistically significant difference in the warning times observed during the before and after studies, it was hypothesized that there would be a significant difference in the numbers of such vehicles. The Kruskal-Wallis test verified this premise at the 99 percent confidence level for the day, night, and total data sets, i.e., a significant reduction in the number of vehicles crossing was realized as a result of the predictors being installed. The predictors reduced the average number of vehicles crossing per train arrival from 10.86 to 3.35 when compared to flashing light signals without predictors. Thus, the predictors and reasonable CWT they provide reduced the number of vehicles that crossed in front of an oncoming train by more than a factor of three.

The effects of warning times on the number of vehicles crossing while the flashing light signals were activated are presented in Table 3. Even though the total observations are not distributed evenly throughout the warning time categories, there is clearly an identifiable trend, i.e., the longer the warning time, the greater the number of vehicles that crossed while the warning devices were activated. This relationship is shown in Figure 3.

TABLE 1 WARNING TIMES AT THE CEDAR DRIVE CROSSING

Summary Statistics	Flashing Light Signals without Predictors			Flashing Light Signals with Predictors		
	Day	Night	Total	Day	Night	Total
Sample Size	53	36	89	22	28	50
Mean (seconds)	73.7	77.6	75.2	40.5	42.7	41.7
Standard Deviation	20.6	13.4	17.9	15.5	19.9	18.0
Range (seconds)	47-141	56-119	47-141	27-89	28-121	27-121

Warning Times[a] (seconds)	Flashing Light Signals without Predictors			Flashing Light Signals with Predictors		
	Observed Train Arrivals	Percent of Total Arrivals	Cumulative Percentage	Observed Train Arrivals	Percent of Total Arrivals	Cumulative Percentage
>20	0	0.0	0.0	0	0.0	0.0
20-30	0	0.0	0.0	6	12.0	12.0
30-40	0	0.0	0.0	28	56.0	68.0
40-50	4	4.4	4.4	6	12.0	80.0
50-60	13	14.5	18.9	5	10.0	90.0
60-90	57	64.5	83.4	4	8.0	98.0
>90	15	16.6	100.0	1	2.0	100.0
Total	89			50		

[a]Time between activation of flashing lights and the train's arrival at the crossing.

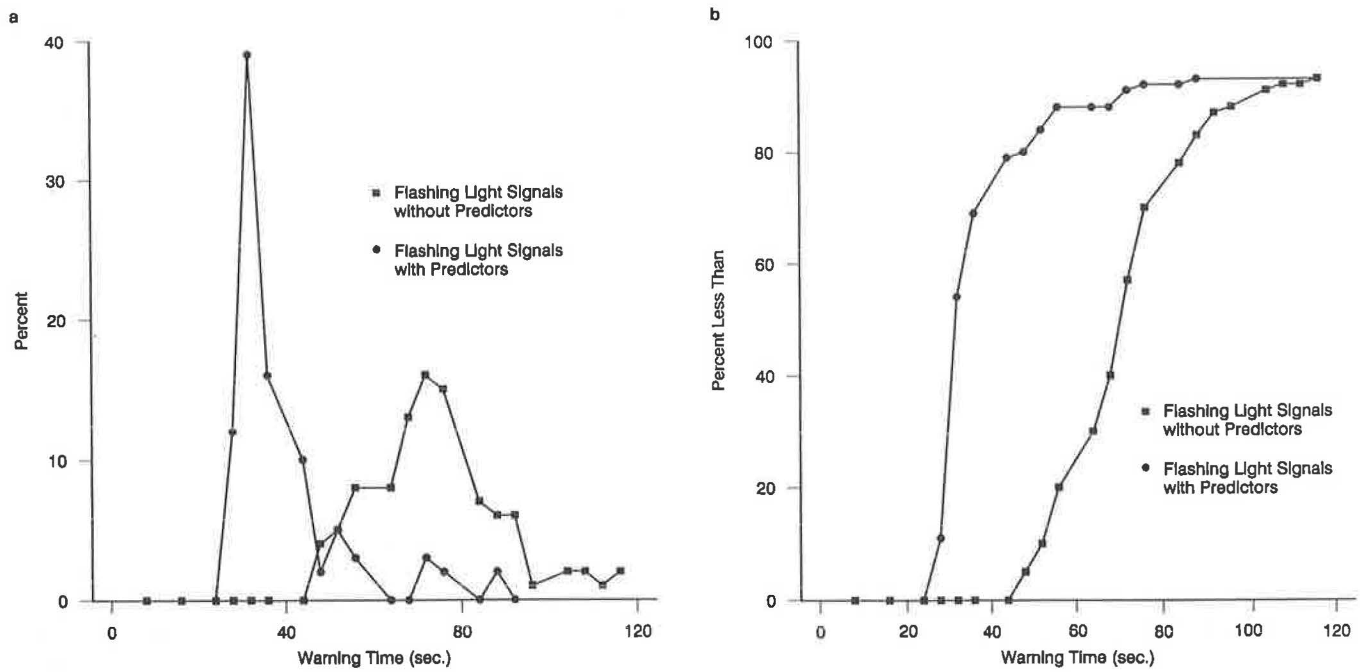


FIGURE 2 (a) Frequency and (b) cumulative frequency distribution of observed warning times at the Cedar Drive crossing.

TABLE 2 VEHICLES CROSSING AT THE CEDAR DRIVE CROSSING

Summary Statistics	Flashing Light Signals without Predictors: Day	Night	Total	Flashing Light Signals with Predictors: Day	Night	Total
Sample Size[a]	53	30	83	21	24	45
Mean (vehicles)	13.28	6.40	10.86	3.86	2.92	3.35
Standard Deviation	7.74	6.28	7.91	3.34	2.50	2.92
Percent >0 Crossing	100.0	97.6	98.8	90.5	83.3	86.7
Percent >1 Crossing	98.1	86.7	94.0	71.4	62.5	66.7
Range (vehicles)	1-40	0-24	0-40	0-12	0-9	0-12

Crossings[b] (vehicles)	Flashing Light Signals without Predictors: Observed Train Arrivals	Percent of Total Arrivals	Cumulative Percentage	Flashing Light Signals with Predictors: Observed Train Arrivals	Percent of Total Arrivals	Cumulative Percentage
0	1	1.2	1.2	6	13.3	13.3
1	4	4.8	6.0	9	20.0	33.3
2	5	6.0	12.0	6	13.3	46.6
3	8	9.7	21.7	6	13.3	59.9
4	2	2.4	24.1	7	15.7	75.6
>4	63	75.9	100.0	11	24.4	100.0
Total	83			45		

[a] Includes only those observations in which vehicles were present before the train's arrival.

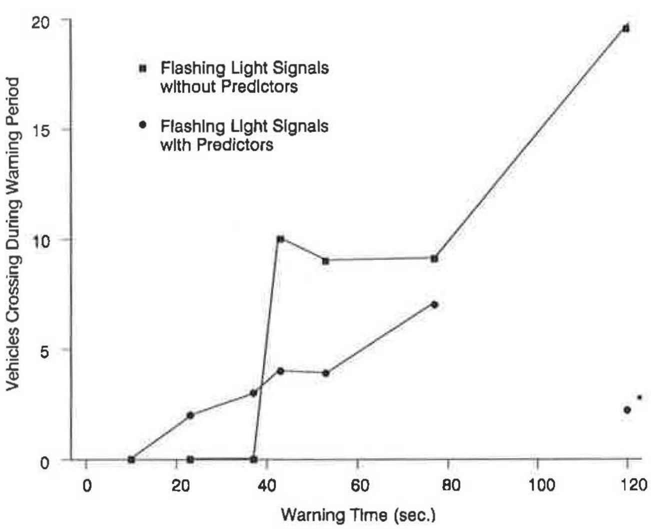
[b] Vehicles crossing after activation of the flashing light signals or the traffic signal changing to yellow and the train's arrival at the crossing.

TABLE 3 EFFECTS OF WARNING TIMES ON NUMBER OF VEHICLES CROSSING AT THE CEDAR DRIVE CROSSING

Study	Warning Time (Sec.)[a]		Observed Train Arrivals[b]	Average No. Crossing (per Arrival)
Flashing Light Signals without Predictors	<20		-	-
	20-30		-	-
	30-40		-	-
	40-50		4	10.00
	50-60		11	9.17
	60-90		53	9.24
	>90		15	19.00
		Total	83	
Flashing Light Signals with Predictors	<20		0	-
	20-30		5	1.60
	30-40		24	2.75
	40-50		6	4.33
	50-60		5	4.40
	60-90		4	6.75
	>90		1	2.00
		Total	45	

[a]Time between activation of flashing lights and train's arrival at the crossing.

[b]Includes only those observations in which vehicles were present.



*There were a few long warning times after predictors caused by slow-moving trains coming off the siding track near the crossing. This data point represents these few observations.

FIGURE 3 Average number of vehicles crossing as a function of warning time at the Cedar Drive crossing.

The general increase in vehicles crossing at higher working times was expected; however, what was not expected was the difference in vehicles crossing with and without predictors. For example, without predictors, warning times in the 40–50-sec range resulted in an average of 10.0 vehicles crossing per train arrival, whereas with predictors, the same warning times resulted in an average of 4.33 vehicles crossing per train arrival (Table 3). This difference is attributed to the shorter and more consistent warning times with predictors.

Crossings Within 20 sec of Train's Arrival

Vehicles (CL20s) crossing within 20 sec of a train's arrival at the crossing have previously been defined as indicative of aggressive behavior, i.e., there is some, but not much, room for driver and vehicular error. Although such behavior is not necessarily illegal, it is characteristic of those drivers who choose to cross within the 20-sec minimum warning time presently required by the MUTCD (*5*). As presented in Table 4, the average number of vehicles crossing within 20 sec of the train's arrival at the Cedar Drive crossing was noticeably less in the after study (in which the predictors were installed), being reduced from an average of 1.82 to 0.78. The Kruskal-Wallis test (*10*) indicated that the reductions were statistically significant for both the daytime and total data sets at the 99 percent confidence level. Thus, as expected, installation of the predictors significantly reduced the number of CL20s at the crossing. There was little difference in the average CL20 rates between any of the nighttime data sets.

A frequency distribution of the observed CL20s at the Cedar Drive crossing is also presented in Table 4. In the before study (flashing light signals without predictors), there were 30 observations with no CL20s, 11 observations with one CL20, and 42 observations with two or more violations. The number of observations in each category was smaller and the percentages were different in the after study (with predictors present). A Pearson's chi-square statistic calculated from a 2-by-3 contingency table (two studies by three CL20 rate categories) substantiated the fact that the differences (fewer multiple CL20s) were significant at the 95 percent confidence level.

The effects of warning times on the CL20 rates at the Cedar Drive crossing are presented in Table 5. From the table, the CL20 rates observed during the before study (without predictors) appear to be higher than those corresponding rates observed after predictor installation. However, the differences cannot be statistically confirmed because of the small numbers of warning times above 40 sec during the after study. That is, there are simply too few corresponding observations to compare between the two studies.

Crossings Within 10 sec of Train's Arrival

Vehicles (CL10s) crossing within 10 sec of a train's arrival at the crossing have previously been defined as an indication of risky behavior; there is little room for either driver or vehicular error. Although not necessarily illegal at a flashing light signal, such behavior intuitively increases the likelihood of an accident's occurring. It was anticipated that installation of the predictors might reduce this type of behavior by providing shorter and more consistent warning times and increased credibility of the warning devices.

As presented in Table 6, 29 CL10s (15 single CL10s and 7 double CL10s) were observed at the Cedar Drive crossing in the before study, i.e., 29 motorists crossed the tracks within 10 sec of the train's arrival. Twenty-five CL10s (13 single CL10s and 6 double CL10s) occurred during the day and four CL10s (2 single CL10s and 1 double CL10) occurred at night. In seven different cases, at least two motorists crossed the tracks within 10 sec of the train's arrival. On the average, there were 0.39 CL10s per train arrival in the before study.

In the after study, 6 CL10s (2 single CL10s and 2 double CL10s) were observed. Four of these CL10s (2 single CL10s and 1 double CL10) occurred in the daytime, and two CL10s (1 double CL10) occurred at night. On the average, there were 0.13 CL10s per train arrival in the after study. A Pearson's chi-square statistic calculated from a 2 by 3 contingency table (two studies by three CL10 categories) indicated that the observed CL10s in the before study (without predictors)

TABLE 4 CL20s AT THE CEDAR DRIVE CROSSING

Summary Statistics	Flashing Light Signals without Predictors			Flashing Light Signals with Predictors		
	Day	Night	Total	Day	Night	Total
Sample Size[a]	53	30	83	21	24	45
Mean (vehicles)	2.34	0.83	1.82	0.95	0.63	0.78
Standard Deviation	1.74	1.60	1.84	0.86	1.10	1.00
Percent >0 Violations	79.2	34.5	63.9	66.7	41.7	53.3
Percent >1 Violations	67.9	17.3	50.6	33.8	8.3	15.5
Range (vehicles)	0-6	0-6	0-6	0-3	0-5	0-5

CL20s[b] (vehicles)	Flashing Light Signals without Predictors: Observed Train Arrivals	Percent of Total Arrivals	Cumulative Percentage	Flashing Light Signals with Predictors: Observed Train Arrivals	Percent of Total Arrivals	Cumulative Percentage
0	30	36.1	36.1	21	46.7	46.7
1	11	13.3	49.4	17	37.8	84.5
2	13	15.7	65.1	5	11.1	95.6
3	12	14.5	79.6	1	2.2	97.8
>3	17	20.4	100.0	1	2.2	100.0
Total	83			45		

[a]Includes only those observations in which vehicles were present before the train's arrival.

[b]Vehicles crossing within 20 seconds of the train's arrival at the crossing.

TABLE 5 EFFECTS OF WARNING TIMES ON CL20 RATES AT THE CEDAR DRIVE CROSSING

Study	Warning Time (Sec.)[a]		Observed Train Arrivals[b]	Average CL20s (per Arrival)
Flashing Light Signals without Predictors	<20		0	-
	20-30		-	-
	30-40		-	-
	40-50		4	3.75
	50-60		11	2.45
	60-90		53	1.63
	>90		15	1.53
		Total	83	
Flashing Light Signals with Predictors	<20		0	-
	20-30		5	0.80
	30-40		24	0.83
	40-50		6	1.00
	50-60		5	0.60
	60-90		4	0.50
	>90		1	0.00
		Total	45	

[a]Time between activation of flashing lights and train's arrival at the crossing.

[b]Includes only those observations in which vehicles were present.

TABLE 6 CL10s AT THE CEDAR DRIVE CROSSING

Summary Statistics	Flashing Light Signals without Predictors			Flashing Light Signals with Predictors		
	Day	Night	Total	Day	Night	Total
Sample Size[a]	53	30	83	21	24	45
Mean (vehicles)	0.53	0.13	0.39	0.19	0.08	0.13
Standard Deviation	0.77	0.43	0.69	0.51	0.41	0.46
Percent with Conflicts	35.9	10.0	26.5	14.3	4.2	8.9
Range (vehicles)	0-3	0-2	0-3	0-2	0-2	0-2
0 CL10s[b]/Arrival	34	27	61	18	23	41
1 CL10s[b]/Arrival	13	2	15	2	0	2
2 CL10s[b]/Arrival	6	1	7	1	1	2

[a]Includes only those observations in which vehicles were present before the train's arrival.

[b]Vehicles crossing within 10 seconds of the train's arrival.

and the after study (with predictors) were significantly different at the 95 percent confidence level. This results means that installation of the predictors appears to have been successful in reducing the amount of risky behavior that took place at the crossing.

Clearance Time

Because predictors significantly shortened the average warning time and reduced vehicles crossing, it was hypothesized that they might give enough credibility to the warning system to increase average clearance times at the crossing. If in fact this was to occur, the overall temporal separation between the cars and trains would be a definite safety benefit.

Clearance times were only recorded for those train arrivals in which a vehicle arrived at the crossing between the activation of the flashing light signals and the train's arrival at the crossing; that is, when there was an opportunity for a vehicle to cross in front of the train. Thus, the number of clearance times observed had to be equal to or less than the number of train arrivals. As presented in Table 7, there were 83 clearance times observed in the before study (without predictors) and 39 clearance times observed in the after study (with predictors). As with the warning time data set, the total data from each study was subdivided into day and night observations to ensure that similar train and traffic volume conditions were compared. These two subsets, together with the total data set, were then analyzed.

The mean clearance times from the total data sets were approximately the same for both studies, ranging from 20.1 to 21.4 sec. The Kruskal-Wallis test for two or more independent, continuously distributed populations confirmed that these differences were not statistically significant at the 95 percent confidence level (*10*). Therefore, installation of the predictors had no measurable effect on the mean clearance times observed at the crossing.

Interestingly, the Mann-Whitney test indicated a statistically significant difference at the 99 percent confidence level for clearance times between the day and night data sets from the two studies. This means that the clearance times observed for day and night operations in both the before and after studies were different. The frequency and cumulative frequency distributions of clearance times from both data sets are shown in Figure 4.

Although the predictors did not affect the mean clearance time at the crossing, they did reduce the occurrence of very short clearance times. This trend is presented in the bottom of Table 7. From the table, 27.7 percent of the clearance times in the first before study would be classified as risky (less than 10 sec), whereas only 10.3 percent of the clearance times observed in the after study would be classified as risky. This is another strong indication of the positive impacts of predictors and CWT on crossing safety.

Speed Profiles

Speed data were analyzed to determine whether the predictors had an effect on approach speeds. In order to compare characteristics of similar vehicles, approach speed profiles for the first vehicle to stop at the crossing in the before study as well

TABLE 7 CLEARANCE TIMES AT THE CEDAR DRIVE CROSSING

	Flashing Light Signals without Predictors			Flashing Light Signals with Predictors		
Summary Statistics	Day	Night	Total	Day	Night	Total
Sample Size[a]	53	30	83	19	20	39
Mean (seconds)	15.7	28.2	20.1	16.2	26.3	21.4
Standard Deviation	13.2	15.0	15.0	5.8	18.9	14.9
Percent >20 seconds	79.3	31.0	62.6	73.7	50.0	61.5
Percent >10 seconds	37.7	10.3	27.7	15.8	5.0	10.3

	Flashing Light Signals without Predictors			Flashing Light Signals with Predictors		
Clearance Times[b] (seconds)	Observed Train Arrivals	Percent of Total Arrivals	Cumulative Percentage	Observed Train Arrivals	Percent of Total Arrivals	Cumulative Percentage
>10	23	27.7	27.7	4	10.3	10.3
10-20	29	34.9	62.6	20	51.3	61.5
20-30	15	18.1	80.7	10	25.6	87.2
>30	16	19.3	100.0	5	12.8	100.0
Total	83			39		

[a]Includes only those observations in which vehicles were present before the train's arrival.

[b]Time between the last vehicle to cross and the train's arrival at the crossing.

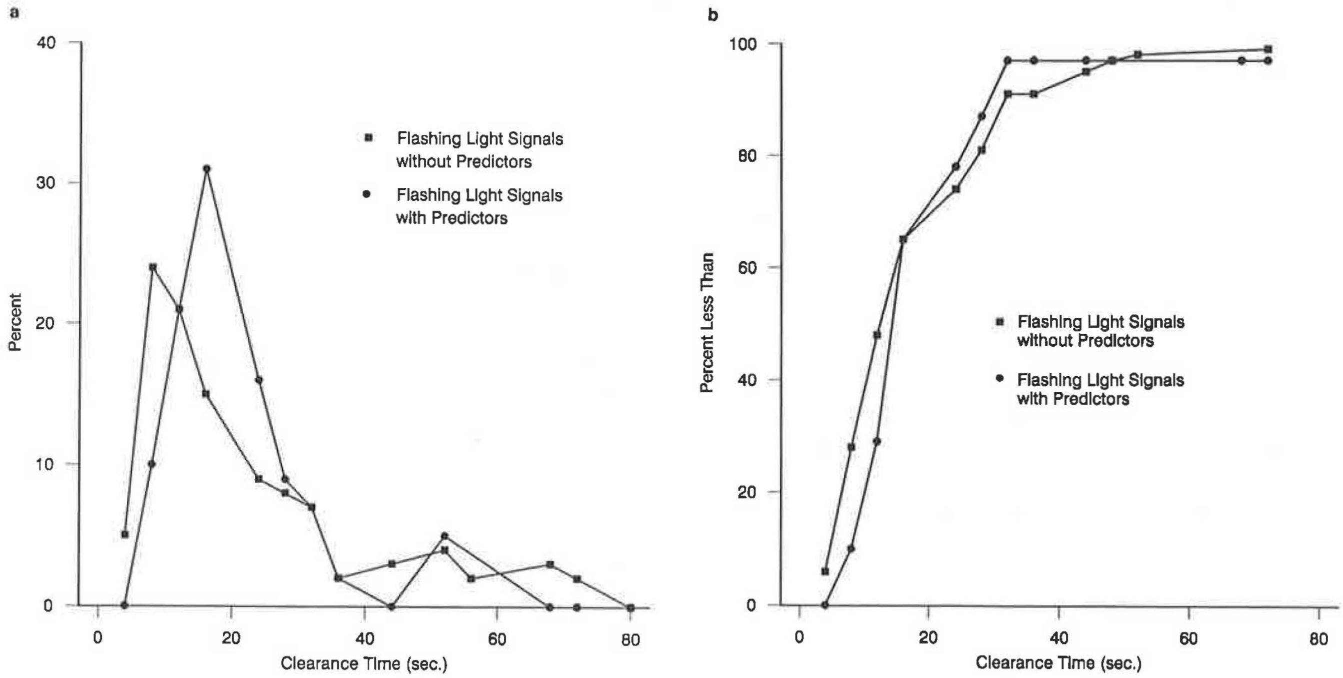


FIGURE 4 (a) Frequency and (b) cumulative frequency distribution of observed clearance times at the Cedar Drive crossing.

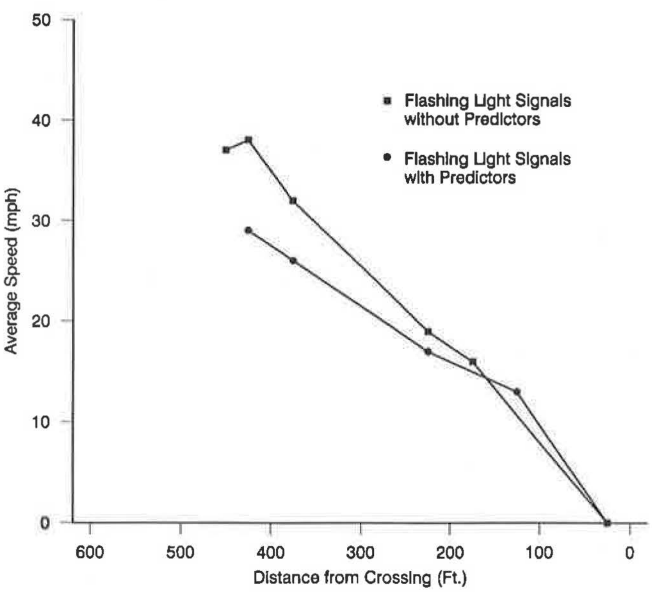


FIGURE 5 Average approach speed profiles for vehicles in advance of the Cedar Drive crossing.

as the after study were plotted as shown in Figure 5. Each data point represents average speeds over 50-ft sections of roadway in advance of the stop bar at the crossing and is plotted at the midpoint of the section. Data in the range of 50–200 ft from the stop bar were obtained from Camera 1 and in the range of 250–450 ft from the stop bar from Camera 2. Insufficient data were available from Camera 3 to plot approach speeds further than 500 ft from the crossing.

Several observations can be made concerning the average approach speed profiles in the before and after data sets. First, the average speeds in the before study were about 5 mph faster than they were in the after study. This speed difference is statistically significant at the 95th percentile and suggests that predictors and the CWT they provide may influence drivers' approach speeds. That is, motorists' increased confidence in the traffic control system may result in their early acceptance of the fact that they will have to stop, and therefore they slow down sooner (in excess of 450 ft from the crossing). It is also important to note that in both studies, the stopping vehicles did so in a safe, gradual, and consistent manner. In addition, the resultant speed profiles appeared to pose no safety problems for approaching motorists.

Perception-Brake Reaction Time and Deceleration

It was expected that the additional credibility resulting from the predictors and CWT may cause motorists to brake sooner and, as a result, slow down more gradually. However, it was also expected that if these differences did exist, they would be small and very difficult to measure. To compound this problem, braking for a flashing light signal is an unexpected event but does not represent a pressure situation to a driver unless a train is also visible. Drivers know that there is at least some length of time before a train's arrival at the crossing, thus driver response to activation of a flashing light signal should be relatively long and probably highly variable.

Average PBRTs in response to the activation of the flashing light signals were 26.6 sec in the before study and 17.1 sec in the after study. For both studies, the standard deviation was almost as large or larger than the mean. The Kruskal-Wallis test indicated that the differences were not statistically significant at the 95 percent confidence level. In other words, the variability in the brake time data precluded being able to find any significant differences that might exist. These long reaction times confirm the premise that braking in response to a flashing light signal at a railroad-highway grade crossing did not represent a pressure situation (short reaction times) and, because of this, was highly variable (large standard deviations). An additional complication with measuring brake reaction times was the difficulty in determining whether the vehicle of interest was braking in response to the activation of the warning device, a slower moving vehicle ahead of it, the roughness of the crossing itself, or something else.

Cost of Train Predictors

The research was not intended to evaluate the cost-effectiveness of train predictors. However, because predictors (and the CWT they provide) were found to be extremely beneficial, a brief discussion of predictor costs is appropriate. First of all, the total cost of the predictors at the single-track Cedar Drive crossing, including hardware and installation cost, was $13,960.97. This cost estimate was provided by the Tennessee Department of Transportation.

From a more general perspective, a basic predictor unit with the redundancy feature costs between $11,500 and $14,000, depending on the supplier and purchase quantity; this cost estimate is based upon input from two railroads (*7*). The cost of a train predictor unit without redundant or backup capability is about 30 percent less. This cost does not include installation costs, battery costs, wiring and relay costs, etc. It should be noted that a single predictor unit normally can handle both approaches of a single track crossing. Multiple-track crossings or crossings with insulated joints nearby will require multiple predictors or sets of unidirectional predictors.

One of the railroads also provided general cost comparisons for installing train predictors versus motion sensors in conjunction with flashing light signals with and without gates. Based on the railroad's estimates, it would cost approximately $42,840 to install flashing light signals with train predictors, whereas it would cost approximately $34,240 to install the same flashing light signals with motion sensors. Thus, the use of predictors versus motion sensors would result in an increased total installation cost of approximately $8,600. For the case of gated crossings, the railroad estimates that it would cost about $61,930 to install standard two-quadrant gates and flashing lights with train predictors, whereas it would cost $50,930 to install gates and signals with motion sensors. In this case, the use of predictors would result in an increased total installation cost of approximately $11,000. These cost estimates are for a typical single-track crossing in Tennessee, and they assume a maximum train speed of 60 mph.

CONCLUSIONS AND RECOMMENDATIONS

The effects of train predictors and CWT on crossing safety and driver response measures were evaluated at a typical grade crossing with flashing light signals. A before and after study approach was used and the results of the studies are as follows:

1. During the 2-month evaluation period, the train predictors performed without a failure or incident.
2. At the test crossing, the installation of train predictors reduced the average length of train warning time from 75.2 to 41.7 sec.
3. Train predictors and CWT they provide reduced the average number of vehicles crossing the tracks while the flashing light signals were activated from 1,086 crossings per 100 train arrivals to 335.
4. The predictors reduced the number of CL20s from 182 to 78 per 100 train arrivals.
5. The predictors reduced the number of CL10s from 39 to 13 per 100 train arrivals.
6. Predictors did not have any adverse effects on speed profiles, brake reaction times, or deceleration at the test crossing.
7. There have been no train-car accidents at the test crossing since the predictors were installed.
8. Based on railroad industry cost estimates, a basic train predictor unit costs between $11,500 and $14,000. It would cost approximately $8,600 to $11,000 more to install predictors at an active crossing, compared to motion sensors.

Based on the study results, the length of the warning time period at active grade crossings is critical to crossing safety and traffic operations. Therefore, it is recommended that train predictors be installed at active crossings that have highly variable and long train warning times. At these crossings, predictors and the CWT they provide will significantly improve crossing safety and enhance motorist respect for the active traffic control systems. Motorist delays at the crossings should also be reduced. As noted previously, there may be as many as 13,100 crossings nationwide with conventional train detectors or motion sensors that would benefit from predictors.

The studies, in documenting the benefits that can be attained by providing predictors and CWT, also emphasized the critical need for additional research. Specifically, research is needed to determine the optional warning time at crossings equipped with predictors, or for that matter, at any crossing with active traffic control. Warrants and guidelines for the use of predictors also need to be developed.

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Assessment of Warning Time Needs at Railroad-Highway Grade Crossings with Active Traffic Control

STEPHEN H. RICHARDS AND K. W. HEATHINGTON

Research was conducted to assess the effects of warning time on driver behavior and safety at railroad-highway grade crossings with active traffic control, i.e., flashing light signals with and without automatic gates. The research included (a) an evaluation of driver response data gathered at three grade crossings in the Knoxville, Tennessee, area; and (b) a human factors laboratory study of drivers' warning time expectations and tolerance levels. In the field studies, the actions of over 3,500 motorists were evaluated during 445 train events. Based on the study results, warning times in excess of 30–40 sec caused many more drivers to engage in risky crossing behavior. The studies also revealed that the large majority of drivers who cross the tracks during the warning period do so within 5 sec from the time they arrive at the crossing. The human factors studies expanded the findings of the field evaluation. Specifically, the studies revealed that most drivers expect a train to arrive within 20 sec from the moment when the traffic control devices are activated. Drivers begin to lose confidence in the traffic control system if the warning time exceeds approximately 40 sec at crossings with flashing light signals and 60 sec at gated crossings. Based on the research, guidelines for minimum, maximum, and desirable warning times are presented. These guidelines are designed to minimize vehicles crossing during the warning period and promote driver credibility for the active control devices.

In the past two decades, over $2 billion has been allocated for improvements at the 192,454 public grade crossing locations in the country. The majority of these improvements involved converting passive crossings to active ones. As of 1986, roughly 30 percent of all crossings had active warning devices—22,066 grade crossings were equipped with automatic gates and flashing light signals and 32,778 were equipped with flashing light signals.

The upgrading of crossings to active control no doubt has contributed to improved crossing safety. Between 1977 and 1986, fatalities at grade crossings dropped from 846 to 501, and injuries decreased from 4,455 to 2,192. Still, over 50 percent of all car-train accidents in 1986 occurred at grade crossings with active devices even though only 30 percent of the total crossings have active control. It is generally recognized that much of this safety problem at active crossings is related to poor driver response to the traffic control. In fact, a study by the National Transportation Safety Board concluded that most accidents at actively controlled grade crossings resulted from drivers intentionally violating the warning device (*1*).

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SYSTEM CREDIBILITY AND WARNING TIME

The poor performance of flashing light signals with and without gates at grade crossings is due in large part to the lack of system credibility for drivers. That is, drivers may not consider these devices to be accurate or reliable, leading to eventual violation of their warning. One factor affecting system credibility is the high number of false activations at some active crossings. Certainly, every effort should be made to minimize these false activations through improvements in track circuitry, train detection equipment, and maintenance practices.

Another factor that may encourage undesirable driver behavior at crossings with active traffic control devices is the amount of time provided between device activation by a train and passage of the train through the crossing, i.e., warning time. Specifically, excessive or highly variable warning times may encourage frustrated drivers to willfully disregard the active devices. Conversely, extremely short warning times leave little margin of safety and poorly accommodate larger vehicles such as combination trucks and buses, especially if those vehicles must first come to a stop as required by many state laws. The current minimum warning time of 20 sec set forth in the *Manual on Uniform Traffic Control Devices* (MUTCD) (*2*) may not be appropriate in all cases.

The warning time issue is hardware related—certain types of train detection devices cannot provide reasonable warning times if train speeds are highly variable. However, train predictors generally can provide a relatively constant warning time at active crossings regardless of train speed. Predictors have been installed at over 6,300 sites in the United States. Studies (*3*) have shown that new train prediction hardware is operationally reliable and that violations, motorist delays, and accidents can be reduced at crossings using predictors, presumably due to the reasonable and consistent warning times they provide (*4*). Another study (*5*) estimated that up to 13,100 additional crossings can benefit from predictor installation.

With the advent of predictors, a new grade crossing traffic control issue has arisen. Now that constant, reasonable warning times can be provided, exactly what these times should be for various conditions must be determined. Also, maximum warning times have not been recommended as of yet, even though experience and intuition suggest that they would improve the credibility of traffic control at grade crossings.

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The contents of this paper reflect the views of the authors, who are responsible for the facts and accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of FHWA.

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RESEARCH OVERVIEW

Research was undertaken to investigate the identified warning time issues.

The objectives of the research were to

1. Identify typical driver behavior and the range of driver behavior (stopping and crossing actions) at active crossings under various conditions and situations;
2. Evaluate and determine the influence and effects of warning time length on driver crossing behavior at gated crossings and crossings with only flashing light signals;
3. Assess driver expectancies and tolerance levels with respect to warning times at active crossings; and
4. Based on the results of the first three objectives, present guidelines for minimum and maximum warning time (optimal range) for active grade crossings.

The research included two major tasks:

1. A field evaluation of driver behavior at active grade crossings; and
2. A human factors laboratory study of drivers' warning time tolerances and expectations.

FIELD STUDY DESCRIPTION

Data for the warning time evaluation were taken from videotapes of driver behavior at three crossings in the Knoxville, Tennessee, area. These videotapes had been collected as part of a recently completed FHWA study (*3*). Two of the study crossings had standard flashing light signals, whereas the third crossing had standard gates with flashing light signals.

The evaluation focused on quantifying the effects of warning time length on key driver response measures. The key driver response measures were (a) vehicles crossing during the warning period (violations); (b) clearance times between a crossing vehicle and the arrival of the train; (c) dwell times, i.e., the amounts of time that motorists waited at the crossing; and (d) exposure times, i.e., the amounts of time that crossing vehicles were on the tracks. Each of these measures intuitively could be affected by the length of the warning period, and each one is related to crossing safety or efficient operations.

Study Sites

The three study sites were all Norfolk Southern crossings. Each of the crossings had relatively high train and traffic volumes, thus affording the opportunity to collect a reasonable amount of driver response data. Also, all three crossings had a history of at least some accidents.

Cherry Street

The first crossing (Inventory No. 730584K) is located in the eastern part of Knoxville on Cherry Street. This double-track crossing has standard automatic gates, standard railroad flashing light signals, and a bell. Cherry Street is a 4-lane, undivided urban street. The roadway approaches to the crossing are straight and level. The average daily traffic at the site is approximately 14,000 veh/day, and the average through train volume is approximately 10 trains per day. The speed limit on Cherry Street is 30 mph. Train speeds at the crossing range from 20 to 40 mph, and motion sensors are installed at the crossing.

Cedar Drive

The second crossing (Inventory No. 730643K) is located in the northern part of the city on Cedar Drive. This crossing has standard railroad flashing light signals. Cedar Drive in the vicinity of the crossing is 2 lanes wide and straight on both approaches to the crossing. The vertical alignment of the roadway and thick vegetation in the vicinity of the crossing restrict drivers' view of approaching trains. The average daily traffic at this site is approximately 14,000 veh/day, and the average through train volume is approximately 16 trains per day. The speed limit on Cedar Drive is 40 mph, and train speeds at the crossing range from 5 to 40 mph.

Initially, the Cedar Drive crossing had standard train detectors, and because train speeds vary substantially at the crossing, warning times tended to be variable and often very long. Data were collected under these conditions. Train predictors were then installed at the crossing, resulting in more consistent and generally shorter warning times. Additional data were collected under these new conditions.

Ebenezer Road

The third crossing (Inventory No. 731461C) is located in the western part of Knox County on Ebenezer Road. This single-track crossing has standard railroad flashing light signals. Ebenezer Road is a 2-lane suburban road, and the roadway's horizontal and vertical alignments limit the visibility of the crossing from both directions. The average daily traffic on Ebenezer Road is approximately 10,000 veh/day, and the average through train volume is approximately 10 trains per day. The speed limit on Ebenezer Road was 40 mph at the time of the studies. Train speeds at the crossing range from 5 to 55 mph; the large majority of trains travel between 45 and 55 mph.

Data Collection

Driver response data were recorded automatically on portable video camera-recorders whenever a train was approaching. Three complete video camera-recorder systems were used at each crossing. The cameras were mounted on 20-ft poles approximately 60 ft from the centerline of the roadway. The first camera-recorder unit at each site was located approximately 300 ft from the crossing, the second approximately 500 ft from the crossing, and the third approximately 700 ft from the crossing. The cameras were aimed towards the crossing and had overlapping fields of view.

To activate the video camera-recorder systems just before activation of the traffic control devices, a train detector system, separate from the regular track circuitry, was used. This special pole-mounted detection system projected an infrared light beam across the tracks. When a train broke the beam, the detector transmitted an audio (FM radio) signal that activated the camera-recorders. A detector was placed on each approach at each crossing such that the camera activation signal was transmitted at least 10 sec before a train activated the traffic control device at the crossing.

Data Reduction and Analysis

Information on weather condition, light condition, train direction, warning time, and type of traffic control were recorded for each train event. At the gated crossing (Cherry Street), the gate delay and descent time also was noted. Vehicle and driver response data were recorded for every vehicle which arrived at the crossing during the entire warning period. These data included vehicle arrival position (first, second, etc.), vehicle type, whether the vehicle was pulling a trailer, direction and lane of travel, whether the vehicle stopped or crossed without stopping, and, as appropriate, the times of stopping, starting up, crossing over the tracks, and clearing the crossing area.

Sample Size

Data were collected at each site for approximately 2 months to observe a sufficient number of both train events and vehicles arriving during the warning period. The sample included several hundred train events, several thousand arriving vehicles, and a wide range of warning times.

Train Events

Table 1 presents the numbers of train events observed at each of the study crossings and all crossings combined. There were 445 train events at the three crossings combined, and vehicles were present during 407 of these events. Also, 258 (66.5 percent) of the train events were in the daytime, and 149 (33.5 percent) were at night.

At Cherry Street, 129 train events were observed; 119 of these events had vehicles present. At Ebenezer Road, 179 train events were observed; vehicles were present during 159 of these events. Data for Cedar Drive are broken down into two groups, i.e., before predictors were installed at the crossing and after predictors were installed. Before installation of predictors, 74 train events were observed; vehicles were present during 70 of these events. After predictor installation, 63 train events were observed; 59 of these events had vehicles present.

Vehicles

A total of 3,555 vehicles were observed—1,030 vehicles at Cherry Street, 1,121 vehicles at Ebenezer Road, 999 vehicles at Cedar Drive before predictor installation, and 405 vehicles at Cedar Drive after predictor installation. The total sample and the samples for each individual crossing were made up predominately of passenger cars. The total sample only included 67 vehicles that were not passenger cars, pickups, or vans. The small number of other vehicle types made it difficult to evaluate the effects of warning times on large vehicles with any degree of confidence.

Warning Times

Table 2 presents the warning time conditions observed at each of the study crossings. A range of warning times was observed at each crossing, thus facilitating the evaluation. There was even a wide range of warning times observed at the Cedar Drive crossing after installation of train predictors. This occurred because train predictors were installed on the mainline track, but there was a siding without predictors just a few hundred feet from the crossing.

TABLE 1 SUMMARY OF TRAIN EVENT SAMPLE SIZES

Train Events	Cherry Street	Cedar Drive (no predictors)	Cedar Drive (predictors)	Ebenezer Road	All Crossings
Total Train Events	129	74	63	179	445
Events with Vehicles	119	70	59	159	407
Events without Vehicles	10	4	4	20	38
Total Daytime Events	87	45	29	139	296 (66.5)[1]
Total Nighttime Events	42	29	34	44	149 (33.5)[1]

[1]Percent of Total Train Events

TABLE 2 SUMMARY OF WARNING TIMES

Warning Times	Cherry Street	Ebenezer	Cedar Drive (no predictors)	Cedar Drive (predictors)
Sample Size	129	179	74	63
Mean Warning Time, sec.	57.9	40.2	75.6	39.8
Standard Deviation sec.	15.6	11.1	19.4	12.8
Range, sec.	28-120	24-110	49-139	26-83
Warning Times				
20-30	2 (1.6%)	14 (7.8%)	0 (0.0%)	5 (7.9%)
30-40	9 (7.0%)	92 (51.4%)	0 (0.0%)	39 (61.9%)
40-50	30 (23.3%)	57 (31.8%)	3 (4.1%)	11 (17.5%)
50-60	39 (30.2%)	5 (2.8%)	12 (16.2%)	2 (3.2%)
60-90	44 (34.1%)	9 (5.0%)	45 (60.8%)	6 (9.5%)
>90	5 (3.9%)	2 (1.1%)	14 (18.9%)	0 (0.0%)
Totals	129 (100.0%)	179 (100.0%)	74 (100.0%)	63 (100.0%)

The mean warning time at the Cherry Street crossing was 57.9 sec, with a range of 28–120 sec. The Ebenezer Road crossing had a mean warning time of 40.2 sec, with a range of 24–110 sec. Before predictors were installed, the Cedar Drive crossing had a mean warning time of 75.6 sec, with a range in warning times of 49–139 sec. After installation of the predictors, the mean warning time dropped to 39.8 sec, with a range of 26–83 sec. Based on a Kruskal-Wallis test, the installation of predictors did significantly lower the mean warning time at the Cedar Drive crossing.

FIELD STUDY RESULTS

General Results

In analyzing the driver behavior data, it was apparent that drivers at the crossing when the traffic control activated could not respond to the devices. Thus, a preliminary analysis was performed to assess driver response during the initial onset of the warning period and to identify those vehicles that should be excluded from the sample. In the case of the Cherry Street crossing, it was also hypothesized that drivers might respond differently during the gate delay and descent period compared to how they would respond after the gates were fully lowered. This issue was also addressed at the start of the evaluation so that the data from the Cherry Street crossing could be handled appropriately.

Onset of Warning Period

Drivers' stopping behavior immediately following device activation was evaluated at each of the study crossings. This evaluation not only served to identify appropriate vehicles to include in the study sample, but also provided insight into drivers' typical perception and brake-reaction times at active crossings.

Figure 1 shows the relationship between driver stopping behavior and arrival time for gates with flashing light signals (Cherry Street crossing) and for flashing light signals (Cedar Drive and Ebenezer Road crossings combined). Arrival time in the figure refers to a vehicle's arrival time relative to the start of the warning period. Arrival time did have a significant effect on the percent of drivers who cross without stopping at all of the crossings. In addition, there was a significant difference in stopping behavior for the two types of traffic control. (In Figure 1, data from two crossings that had flashing light signals are combined.)

With regard to flashing light signals, all drivers who were within 1 sec of the crossing at the time of device activation crossed without stopping. Obviously, these drivers simply had no chance to respond to the signals. For arrival times of 1–4 sec, the percentage of drivers crossing without stopping declined steadily. At around the 4-sec point, the percentage of drivers who crossed without stopping leveled off to approximately 15–20 percent. After this 4-sec point, the large majority of drivers could have stopped at the study crossings, but a consistent few did not. The 4-sec point therefore was selected for sample screening, i.e., vehicles arriving at the crossing less than 4 sec into the warning period were not considered in the sample.

Figure 1 shows that driver stopping behavior during the onset of the warning period is much different at gated crossings compared with crossings with flashing light signals. Most drivers at the Cherry Street crossing did not react to the initial device activation. Instead, most drivers continued to cross without stopping well into the gate delay and descent period. From the figure, 60 percent or more of the drivers crossed without stopping during the first 9 sec of the warning period. Because the average gate delay and descent time at the Cherry Street crossing was approximately 14 sec, it follows that most drivers responded to the onset of a gate with flashing light signal activation by driving to beat the gates. They only stopped when they could no longer clear before the gates were lowered.

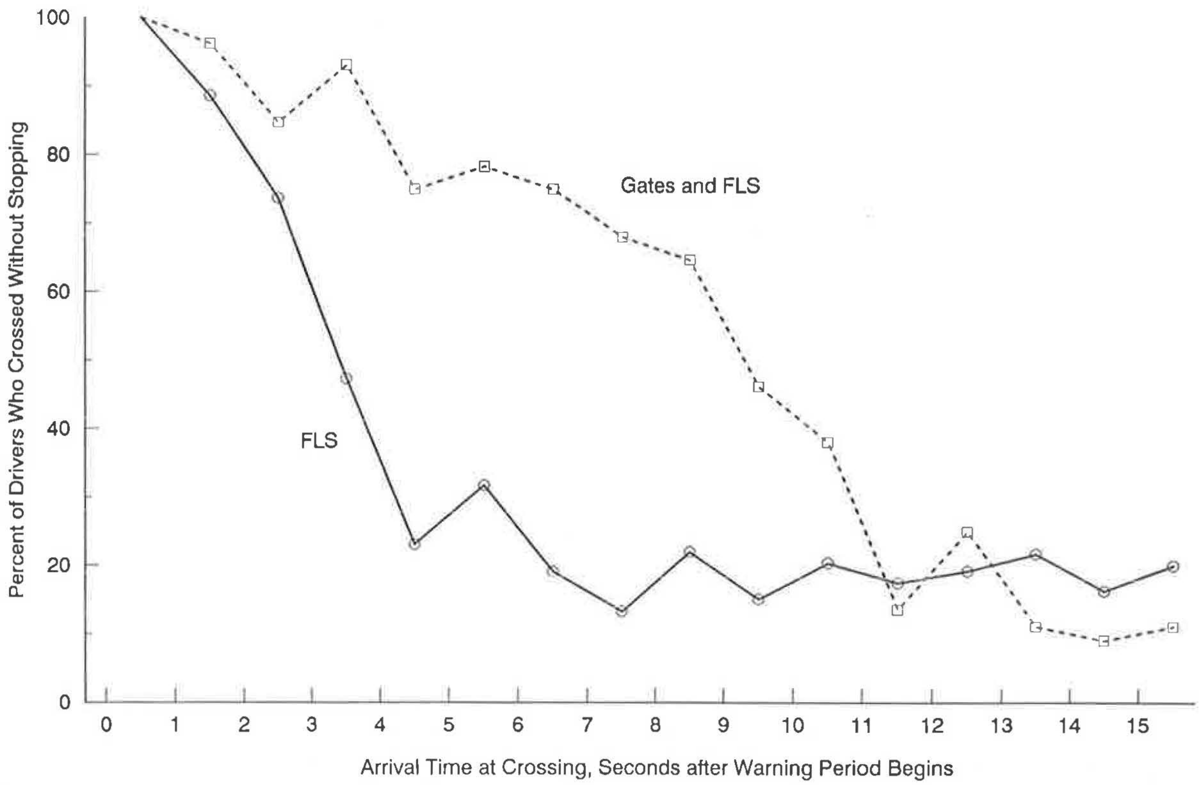


FIGURE 1 Relationship between arrival time at crossing and stopping behavior.

Gate Delay and Descent Period

For the gated crossing, it was theorized that driver behavior during the gate delay and descent period would be different from driver behavior after the gates were fully lowered. This expectation is certainly confirmed by the data for the Cherry Street crossing shown in Figure 1. Therefore, it was appropriate to break down the driver response data for the Cherry Street crossing into the two time periods: (1) before gates were fully lowered; and (2) after gates were fully lowered.

In evaluating driver response data for the gate delay and descent period, it was also noted that the length of this time period apparently had a significant effect on driver stopping behavior. Figure 2 shows the influence of gate delay and descent time on the percentage of drivers who crossed without stopping at the Cherry Street crossing. The percentage of drivers not stopping rose sharply as the gate delay and descent time increased from around 10 to 14 sec. At about 15 sec, the percentage of drivers who crossed without first stopping leveled off at approximately 50 percent. The data in Figure 2 suggest that longer gate delay and descent times may encourage drivers to try to beat the gates and discourage them from stopping. If this is the case, reasonably short times should promote overall crossing safety; however, short gate delay and descent times may also increase the frequency of gate rubs by long, slowly moving vehicles.

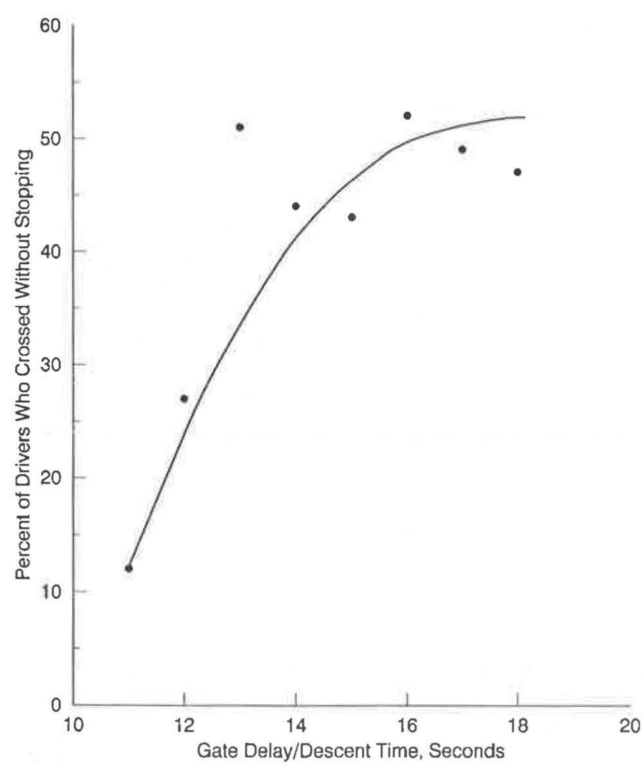


FIGURE 2 Relationship between gate delay and descent time and stopping behavior.

TABLE 3 SUMMARY OF DRIVER BEHAVIOR BY CROSSING

Crossing	Total Vehicles	Percent of Drivers: Crossed without Stopping	Stopped and Crossed	Stopped and Waited
Cherry Street (gate delay/descent)	162	56.2	11.7	32.1
Cherry Street (gates lowered)	768	11.7	28.0	60.3
Ebenezer Road	1,036	14.5	31.1	54.4
Cedar Drive (without predictors)	937	19.0	61.4	19.6
Cedar Drive (with predictors)	363	10.5	45.5	44.1

Summary of Driver Stopping and Crossing Behavior

Table 3 presents a summary of driver behavior observed at the three study crossings. The data in the table exclude those drivers who were less than 4 sec from the crossings when the traffic control was activated.

At the Cherry Street crossing, 60.3 percent of the motorists who arrived while the gates were being lowered stopped and waited; only 32.1 percent of drivers who arrived during the gate delay and descent period stopped and waited. These percentages were lower than expected, particularly the percentage for the gate delay and descent period. Note also from Table 3 that 56.2 percent of the motorists who arrived during the gate delay and descent period crossed without stopping. This relatively high percentage further illustrates that many drivers (in this case over one-half) tried to beat the gates and did not respond appropriately to the advance warning of the flashing light signals before and during gate activation.

In the case of flashing light signals, 14.5, 19.0, and 10.5 percent of the drivers crossed without stopping at the Ebenezer Road, Cedar Drive (without predictors), and Cedar Drive (with predictors) crossings, respectively. Most of these drivers slowed down considerably and looked for the train, but still their actions violated state law and safe driving behavior. The percentages of drivers who did not stop were approximately the same at both crossings with flashing light signals, and with and without predictors at the Cedar Drive crossing. Also, approximately the same percentage (11.7 percent) of drivers did not stop at the Cherry Street crossing after the gates were down. Thus, it would seem that roughly 10 percent of motorists at all the study crossings demonstrated very undesirable behavior.

Table 3 also shows that 54.4, 19.6, and 44.1 percent of drivers stopped and waited at the Ebenezer Road, Cedar Drive (without predictors), and Cedar Drive (with predictors) crossings, respectively. The percentage of drivers who did not cross was approximately the same at Ebenezer Road (which had only flashing light signals) as it was at the Cherry Street crossing (which had gates). On the positive side, this sameness shows that flashing light signals, when operated efficiently, can encourage most drivers to stop and wait. On the negative side, it indicates that driver response to the Cherry Street gates certainly needs to be improved.

The effectiveness of the train predictors installed at the Cedar Drive crossing are also highlighted by the data in Table 3. Only 19.6 percent of the drivers stopped and waited before predictors were installed and warning times were highly variable and sometimes very long. After predictors were installed, this percentage rose to 44.1 percent. This difference was statistically significant at the 0.01 conflict level.

Light Condition

Chi-square tests for independence indicated that light condition effects were significant for the Cherry Street crossing during the gate delay and descent period and for Ebenezer Road. At the Cherry Street crossing, 29.9 percent of the motorists arriving during the gate delay and descent stopped and waited in the daytime; this percentage rose to 50 percent at night. It is theorized that drivers at night were less inclined to attempt to beat the gates because of the reduced visibility and depth perception. Once drivers stopped and the gates were lowered, then the physical and legal restriction of the gates discouraged crossings.

At the Ebenezer Road crossing, 53.4 percent of the drivers stopped and waited in the day and 69.6 percent stopped and waited at night. However, further analysis revealed that the nighttime sample had a disproportionate number of shorter warning times relative to the daytime sample. The nighttime differences were attributed to warning time differences rather than light condition effects.

Weather Conditions

The impact of inclement weather (i.e., rain or snow) on driver stopping and crossing behavior was evaluated. Based on the evaluation, inclement weather had no significant effects on the percentage of drivers who stopped and crossed during the warning period.

Warning Time Effects on Stopping and Crossing Behavior

The effects of warning time on stopping and crossing behavior are shown in Figure 3. The data from the Ebenezer Road and Cedar Drive (with predictors) crossings are combined in the figure, because these crossings had essentially warning time conditions and both had flashing light signals. Separate curves are shown for the Cherry Street crossing, which had gates, and for the Cedar Drive (without predictors) crossing, which had highly variable and long warning times. The Cherry Street crossing data do not include vehicles arriving during the gate delay and descent period.

Warning time had a very significant effect on crossing behavior at the Cherry Street, Ebenezer Road, and Cedar Drive (with predictors) crossings. Generally, a very high percentage of drivers stopped and waited at these crossings if the warning time was relatively short, i.e., 20–30 sec. However, as the warning times increased beyond 30 sec, the percentage of drivers who stopped and waited declined steadily.

At the Cherry Street crossing, approximately 90 percent of arriving motorists stopped and remained stopped for warning times of 20–25 sec. This percentage declined to about 70 percent for warning times of 25–30 sec and to approximately 60 percent for warning times of 30–35 sec. Then, the percentage of drivers who stopped and remained stopped remained fairly constant (at around 60 percent) for warning times up to about 80 sec. After 80 sec, there was again a sharp drop in driver obedience to the gates, to below 30 percent. These data indicate that relatively short warning times (20–35 sec) are desirable to minimize gate violations. They also suggest that, if warning times are greater than about 35 sec, approximately 40 percent or more of the drivers will violate the gates.

At the Ebenezer Road and Cedar Drive (with predictors) crossings, 98 percent of the drivers stopped and remained stopped for warning times of 20–25 sec. At warning times of 25–30 sec, 73 percent of the drivers stopped and remained stopped, and at warning times of 30–35 sec, approximately 90 percent stopped and remained stopped. For warning times beyond 35 sec, the percentage of motorists stopping and remaining stopped declined steadily to less than 20 percent for warning times over 80 sec. As was the case for the gated Cherry Street crossing, these data suggest that relatively short warning times (25–35 sec) are very desirable at crossings with flashing light signals. Driver crossing behavior deteriorates very rapidly for warning times greater than 35 sec.

The severe driver behavior deficiencies at the Cedar Drive (without predictors) crossing are shown in Figure 3. Because of the generally long warning times at the crossing (the mean warning time was over 75 sec), the percentage of drivers who stopped and waited never rose above 30 percent and even dropped to approximately 10–15 percent for warning times greater than 80 sec. Even at moderately long warning times of 45–60 sec, the Cedar Drive (without predictors) crossing performed significantly worse than the Ebenezer Road and Cedar Drive (with predictors) crossings, at the same warning times. This result strongly suggests that long and variable warning times at an individual crossing can have negative impacts that affect overall driver behavior at the crossing. This finding supports the need for consistently short warning times, i.e., some long warning times at a crossing may negate the positive influences of reasonably short times at the same crossing.

Train Wait Time

When drivers arrive at an active crossing too soon before the train arrives, they are unlikely to wait, regardless of the status of the active devices. This issue was addressed in the field studies by evaluating driver crossing behavior as a function

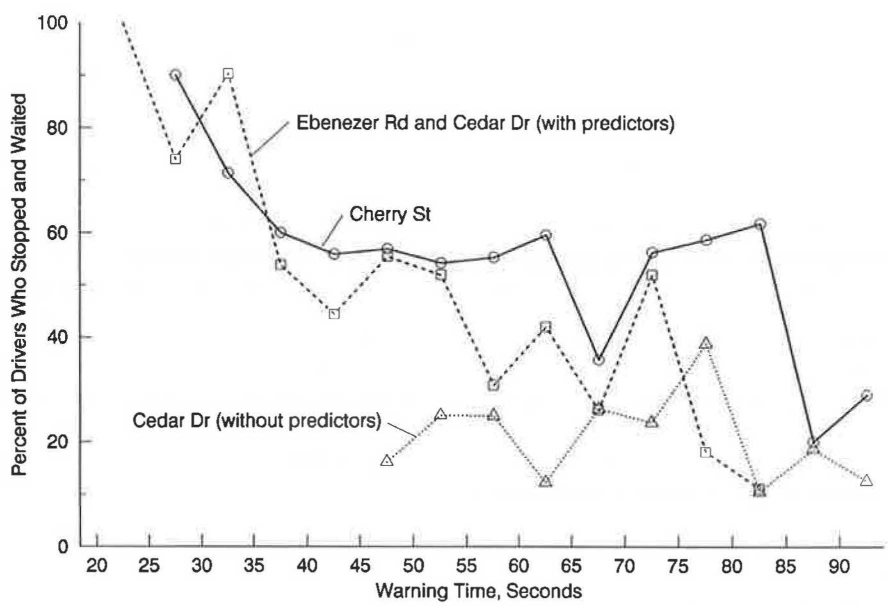


FIGURE 3 Effects of warning time on drivers' stopping and waiting behavior.

of the driver's arrival time at the crossing relative to the train's arrival (train wait time). The results are shown in Figure 4.

At the Cherry Street crossing, 98.2 percent of the drivers arriving at the crossing 10 sec or less before train arrival stopped and remained stopped, and 80.8 percent arriving 10–20 sec before the train arrival stopped and remained stopped. After 20 sec, the percentage dropped sharply. These data suggest that the maximum warning time at gated crossings should be as close to 20 sec as practical, or at least as short as large vehicle clearance requirements will allow.

At all the crossings with flashing light signals nearly all the drivers arriving at the crossing less than 10 sec before train arrival stopped and remained stopped. These percentages were 98.0 percent at the Ebenezer Road crossing, 95.2 percent at the Cedar Drive (without predictors) crossing, and 95.1 percent at the Cedar Drive (with predictors) crossing. For train wait times of 10–20 sec, the percentage of motorists who stopped and waited fell off slightly to 62.5, 51.1, and 64.1 percent, respectively. However, after 20 sec the percentages dropped sharply to below 30 percent in every case. These data suggest that the maximum warning time at crossings with flashing light signals should be near 20 sec, or as short as large vehicle clearance requirements will allow, to minimize unwanted vehicles crossing during the warning period.

Dwell Time

Another important issue related to crossing behavior is dwell time, i.e., the time that crossing drivers spend deciding to cross. Figure 5 shows the dwell time characteristics observed at the three study crossings. Dwell times at the crossings with flashing light signals ranged from around 1 to over 30 sec; however, the large majority of dwell times at these crossings were relatively short. In fact, around 90 percent were less than 5 sec, and 70 percent were less than 4 sec. Thus, the vast majority of crossing drivers at those crossings with flashing light signals stopped and then crossed after taking a quick look for the train.

At the Cherry Street crossing, dwell times tended to be longer, but still were relatively short. Dwell times ranged from 1 to over 30 sec. However, approximately 50 percent of the times were less than 5 sec, and 80 percent were less than 10 sec. At a gated crossing, drivers seem to take more time evaluating the risks, assessing the path they must take, and checking the actions of other drivers. Still, the dwell times do not suggest that drivers are crossing after they get frustrated and tired of waiting; rather they are crossing just as soon as they feel comfortable doing so. As expected, warning time had no significant effects on dwell time.

Warning Time Effects on Clearance Times

Figure 6 shows the mean clearance times observed at each of the crossings by warning time. Clearance time is the difference in train arrival time and vehicle crossing time for those vehicles that crossed. The mean clearance time tended to increase with increasing warning times at each of the crossings. This finding was expected given that most crossing drivers do so within a few seconds of arriving at the crossing. Thus, as the warning time increases, more drivers would be arriving at the crossing a longer time before train arrival. Because the earlier arriving drivers cross fairly quickly (if they are going to cross), this would cause a steady increase in the mean clearance times as the warning time increases.

In Figure 6, the mean clearance times did not differ significantly from crossing to crossing, even at the Cherry Street crossing. It is also important to note that, at the shorter warning times observed (25–35 sec), the mean clearance times were sufficiently large and near 20 sec. This finding supports the current 20-sec minimum warning time as an appropriate minimum level. Even when the warning time was relatively short, clearance times tended to remain near 20 sec, indicating

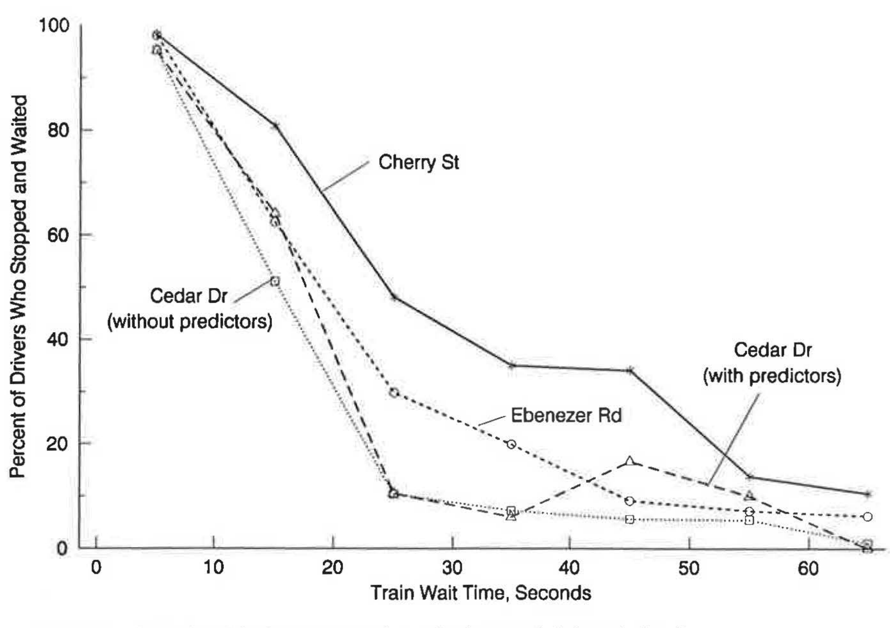


FIGURE 4 Relationship between train wait time and driver behavior.

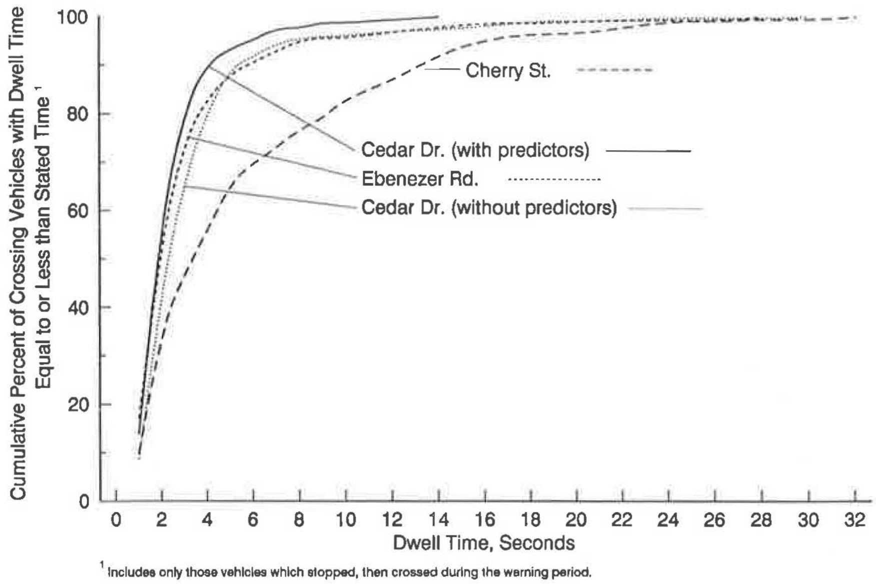


FIGURE 5 Dwell time characteristics.

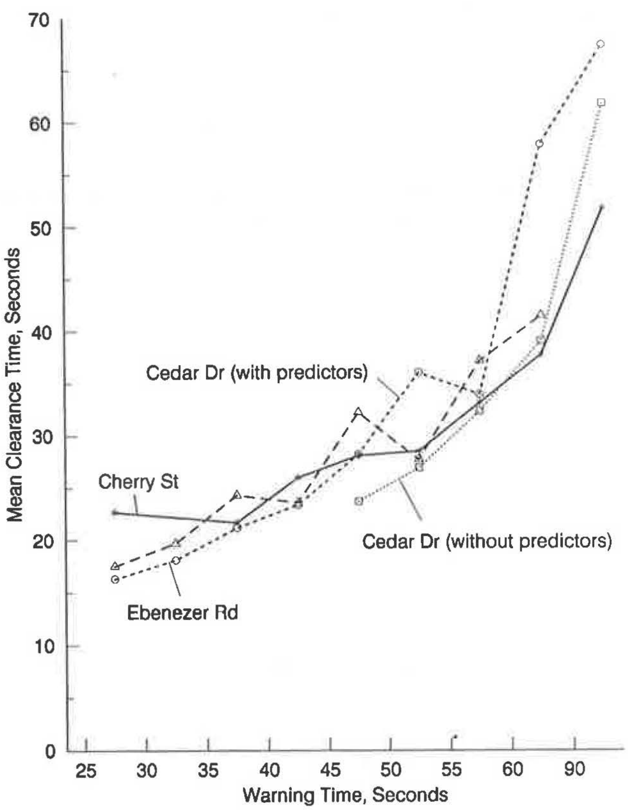


FIGURE 6 Relationship between mean clearance time and warning time.

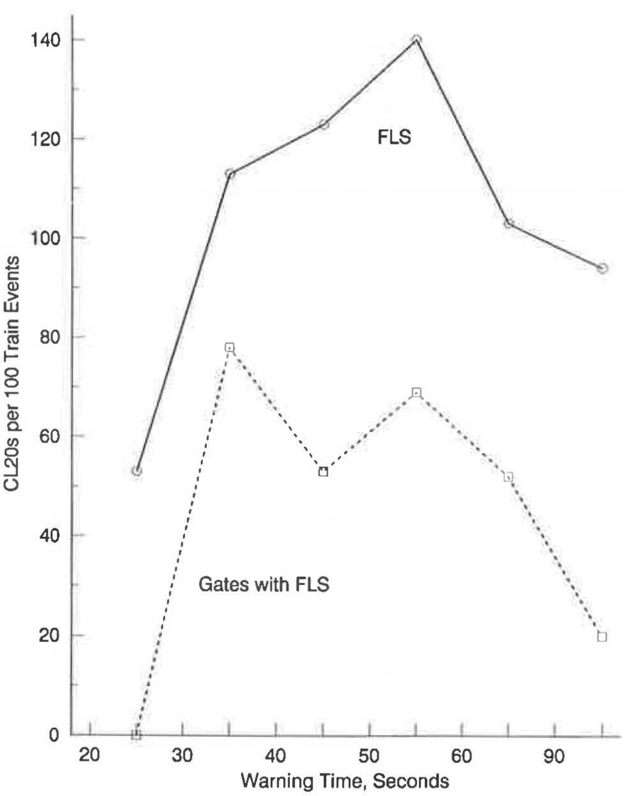


FIGURE 7 Relationship between number of CL20s and warning time.

that the majority of drivers would not regularly accept clearance times of much less than 20 sec.

CL20s

A clearance time of less than 20 sec was defined in the previous research (*3*) as an indicator of risky driver behavior. It would be desirable to minimize the number of CL20s at any active crossing in the interest of safety. Figure 7 shows the effects of warning time on CL20 rates, or the number of CL20s per 100 train events. CL20s were lowest for warning times of 20–30 sec, and they increased significantly for longer warning times.

At the Cherry Street crossing, no CL20s were observed for warning times of 20–30 sec. However, the sample size was very small, i.e., only two trains. For warning times of 30–40 sec, the average CL20 rate jumped to 78.8 per 100 trains. Most of these vehicles actually crossed during the gate delay and descent period and not while the gates were down. This fact emphasizes the earlier finding that drivers are not responding as intended to the advance warning provided by the flashing light signals, but are attempting to beat the gates whenever possible. For warning times greater than 40 sec, the CL20 rate remained high, and was highest for warning times of 50–60 sec.

In Figure 7, data for the crossings with flashing light signals are combined, in recognition of the similarities in clearance time characteristics among the crossings, as shown in Figure 6. There were an average of 52.6 CL20s per 100 trains for warning times of 20–30 sec. At higher warning times, the CL20 rate rose significantly. The highest rate (142.1 CL20s per 100 trains) was observed for warning times between 50–60 sec.

CL10s

The research by Heathington et al. (*3*) also defined a CL10, i.e., a clearance time of less than 10 sec, as a measure of a near-miss or potential car-train conflict. Certainly, it would be desirable to minimize (or better yet, totally eliminate) the number of CL10s at active crossings.

Figure 8 shows the effects of warning times on CL10 rates per 100 train events. At the Cherry Street crossing, there were no CL10s for warning times of 20–30 sec. However, the sample size was very small, i.e., two trains. For warning times of 30–40 sec, there was an average of 11.1 CL10s per 100 train events. This rate was computed based on only one CL10 observed in nine train events and thus may be somewhat misleading. The next highest CL10 rate (10.3 CL10s per 100 train events) was observed for warning times of 50–60 sec.

At the crossings with flashing light signals, there were no CL10s for warning times of 20–30 sec. This rate of zero is based on a sample of 19 train events. For warning times of 30–40 sec, there were an average of 6.9 CL10s per 100 trains, and this rate increased to 16.9 CL10s per 100 trains for warning times of 40–50 sec. Clearly, from these data, shorter warning times discouraged CL10s at the two crossings with flashing light signals. These positive effects were statistically significant.

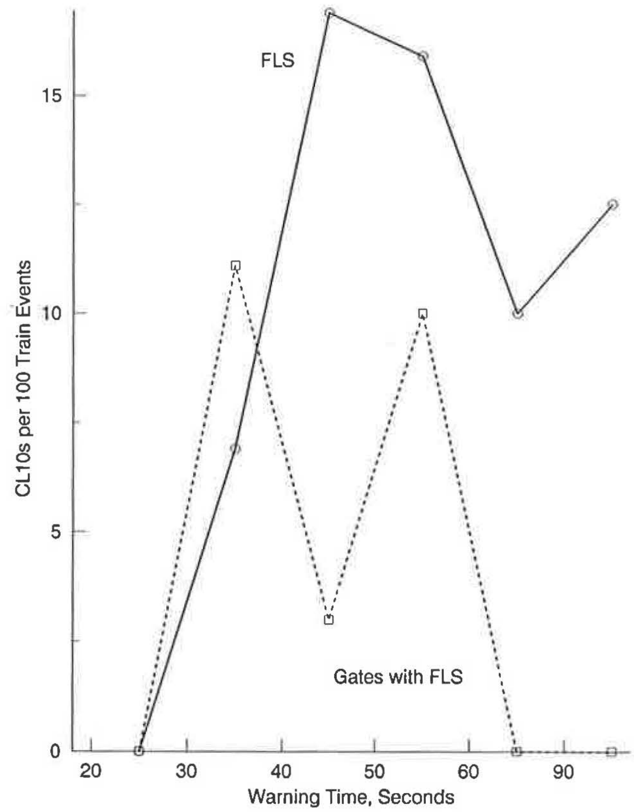


FIGURE 8 Relationship between number of CL10s and warning time.

Warning Time Effects on Exposure Times

Exposure time is the time that a crossing vehicle is on the tracks and directly exposed to a potential car-train accident. Because certain vehicles are required to stop at all crossings, it follows that minimum warning times should be greater than the exposure times of these vehicles. Figure 9 shows the exposure times observed at the study crossings. Exposure time was measured as the time it took a crossing vehicle to travel from the stop line to completely clear the tracks. The data in the figure are for all vehicle types, including large trucks and buses. Unfortunately, the sample included very few trucks and a single bus, thus the results predominantly represent passenger car (including pickups and vans) characteristics.

At the Cherry Street crossing, exposure times ranged from under 2 sec to just over 13 sec. Approximately 50 percent of the exposure times at Cherry Street were less than 4 sec, and about 90 percent were less than 9 sec. A few large trucks drove around the lowered gate arms at the Cherry Street crossing, and these trucks had the longer exposure times. The longest exposure time of 13 sec was experienced by a semi-trailer unit that stopped and then drove around the lowered gate arms.

At the crossings with flashing light signals, exposure times were much shorter and more consistent than at the Cherry Street crossing. Exposure times at the Ebenezer Road and Cedar Drive crossings were 1–11 sec; approximately 80 percent of the times were less than 3 sec. There were very few trucks or buses in the sample; however, a few of these large

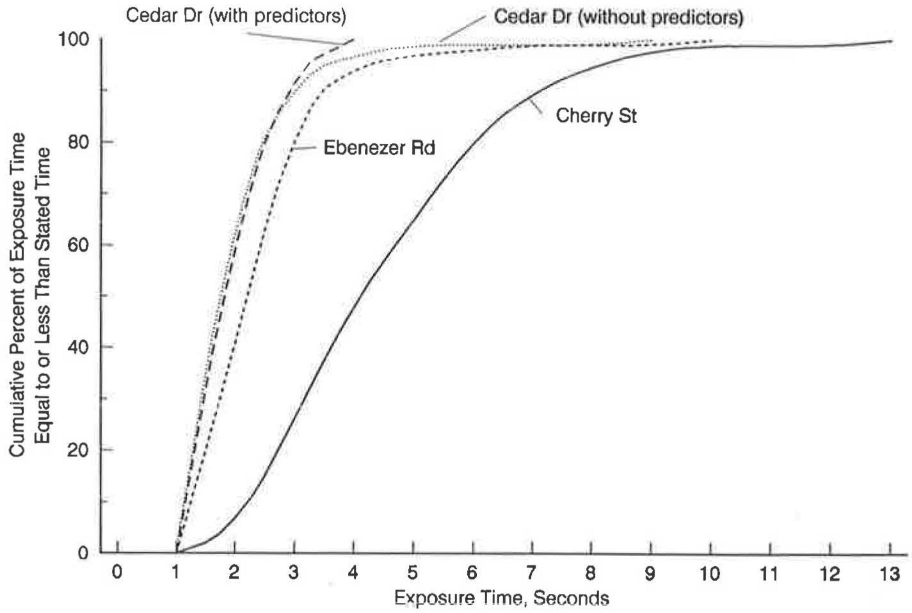


FIGURE 9 Exposure time characteristics.

vehicles did cross and they tended to have the longer exposure times.

Based on the data presented in Figure 9, warning time had no significant effects on exposure time at any of the crossings.

HUMAN FACTORS LABORATORY STUDY

The field study results strongly suggest that drivers have certain expectations and tolerance levels associated with warning times at active grade crossings. These expectations and tolerance levels undoubtedly influence drivers' crossing behavior and thus crossing safety. In order to explore warning time expectations and tolerance levels more fully, a human factors laboratory study was developed and conducted as part of the overall research effort. The specific objectives of the laboratory study were to

1. Determine the extent of variation among drivers with respect to their warning time expectations and tolerance levels;
2. Compare warning time expectations and tolerance levels at crossings with flashing light signals versus crossings with gates and flashing light signals; and
3. Identify general trends in warning time expectations and tolerance levels, and associate these trends to the driver behavior observed in the field studies.

Sixty driver subjects were shown videotapes of staged traffic control device activation events at active grade crossings. While individually viewing an activation event, each subject was asked to indicate: (1) when he or she would expect a train to arrive at the crossing; and (2) when the elapsed time without a train arriving had become too long. One-half of the subjects viewed a videotape that showed the activation sequence at a crossing with flashing light signals. The other half viewed a videotape showing the activation sequence at a crossing with gates and flashing light signals.

Flashing Light Signals

The mean expected time to train arrival for the flashing light signals was 14.5 sec. This mean time is slightly (5.5 sec) less than the 20-sec minimum warning time currently required at crossings with flashing light signals. The fact that drivers, on the average, expect warning times to be less than the minimum time supports the need to keep warning times as short as possible. The relatively short expected train arrival time probably accounts for the high percentages of drivers who cross as the warning times increase above 20–30 sec.

The mean excessive elapsed time was 39.7 sec for the flashing light signals. This time is consistent with driver behavior observed in the field studies, i.e., crossing violations were very frequent when the warning time exceeded 40 sec, whereas violations decreased as the warning time dropped below 40 sec. It is also significant to note that the mean excessive time was approximately 25 sec greater than the mean expected train arrival time, supporting the premise that there is a range of warning times that minimizes unwanted crossings and reinforces the credibility of the warning system.

The range of excessive elapsed times was 26.0–57.8 sec, a spread of 31.8 sec. The low end of this range, i.e., 26.0 sec, is only 6 sec higher than the 20-sec minimum warning time currently required. This supports the conclusion that the majority of drivers would not lose confidence in flashing light signals if warning times were kept at or slightly higher than the current minimum value of 20 sec.

Gates with Flashing Light Signals

The mean expected time to train arrival for the gate with flashing light signals was 30.6 sec, including the gate delay and descent time. This mean time is approximately 10 sec higher than the 20-sec minimum warning time required at active crossings. Thus, warning times at gated crossings at or only slightly higher than the current minimum allowable warning time of 20 sec should promote driver confidence and respect.

The mean expected train arrival time, including the gate delay and descent time, was significantly longer than the mean time for flashing light signals. However, after the gate descent time is subtracted, the mean expected train arrival for a gated crossing is approximately the same as for a crossing with flashing light signals. The mean expected train arrival time for the gated crossing, excluding gate delay and descent time, was 13.2 sec, compared to 14.5 sec for the crossing with flashing light signals. Thus, it is concluded that drivers generally do not consider the gate delay and descent phase in terms of their warning time expectations at gated crossings. That is, they anchor their expectancies to the end of the gate delay and descent phase rather than the beginning of the phase.

The range in expected times to train arrival for the gated crossing was 20.9–47.4 sec including the gate delay and descent time, and 3.5–30.0 sec excluding the gate delay and descent time. This spread of 26.5 sec is not significantly different than the spread of 26.3 sec for the flashing light signals. However, it is important to note the lower limit (i.e., 3.5 sec) of the range after subtracting out the gate delay and descent time. Apparently, some drivers may grow impatient if the train does not arrive almost immediately after the gates are fully lowered. This is consistent with the field studies that found that some drivers drive around the gates almost immediately upon arriving at the crossing if the train is not imminently close.

The mean excessive elapsed time for the gated crossing was 66.2 sec including the gate delay and descent time, and 48.8 sec excluding the gate delay and descent time. These mean times combined with the expected train arrival times support the premise that there is an optimal range of warning times for gated crossings that minimizes gate violations and maximizes driver confidence and respect for the traffic control system. Based on laboratory study results, this range would be 20–60 sec, including the gate delay and descent time.

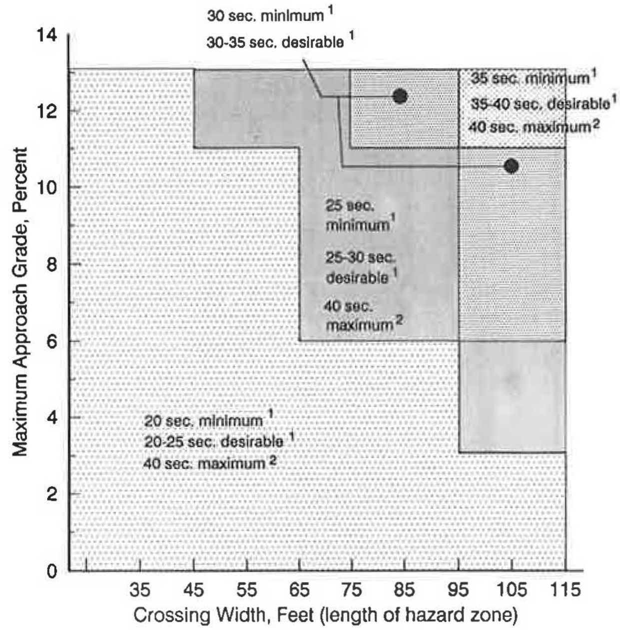
Even excluding the gate delay and descent phase, the mean excessive elapsed time for the gated crossing was significantly higher (at the 99 percent confidence level) than the mean time for the crossing with flashing light signals (i.e., 48.8 sec for gates versus 39.7 sec for flashing light signals). The difference of about 10 sec is consistent with driver behavior observed in the field studies and with the generally more restrictive appearance and legal status of gates. Drivers apparently tolerate longer total warning times at gated crossings before losing confidence in the traffic control.

GUIDELINES FOR WARNING TIMES

On the basis of the results of the field and laboratory studies, guidelines were developed for minimum, maximum, and desirable warning times at grade crossings with active traffic control. Guidelines for gate delay and descent times also were developed.

Flashing Light Signals

Figure 10 shows the suggested guidelines for warning times at crossings with flashing light signals. These values are consistent with the current minimum warning time of 20 sec in this country, and with warning time practices in many foreign countries (*6*). They are designed to (1) provide sufficient time for stopping; (2) minimize vehicles crossing during the warning period; (3) minimize number of CL10s and maintain adequate clearance times; (4) minimize unnecessary driver delay; and (5) promote driver confidence in flashing light signal systems. The values also provide safe clearance time for those vehicles that by law must stop at all crossings. Safe clearance times are reported by Bowman and McCarthy (*7*).



[1] Times shown apply to crossings with no twin or triple tractor-trailer combinations. If twins or triples are expected, times should be increased by 10 percent.

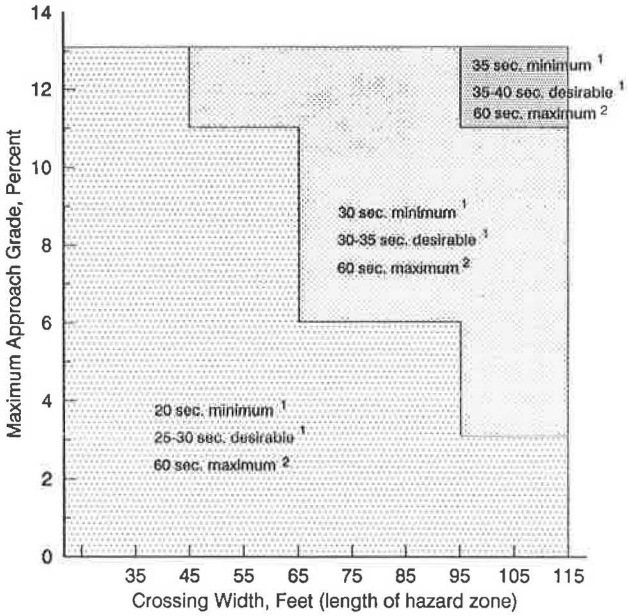
[2] If more than 10 percent of the warning times exceed 40 seconds, installation of motion sensors or train predictors is recommended. If motion sensors or predictors cannot reduce warning times to the desired level, installation of gates should be considered.

FIGURE 10 Suggested warning time guidelines for crossings with flashing light signals.

The suggested minimum warning times range from 20 to 35 sec depending on the width and grade of the crossing. These values should be increased by 10 percent if twin or triple tractor-trailer combinations are present. The suggested ranges of warning times are relatively narrow, i.e., 5 sec. These narrow ranges are strongly supported by the research results. Recognizing practical limitations of train operations and train detection hardware, some longer warning times would be allowed. However, if more than 10 percent of the warning times exceed 40 sec, then the installation of motion sensors or train predictors is strongly urged. The 10 percent value is somewhat arbitrary; however, it is intended to define the upper limit of occasional excessive warning times. If motion sensors or predictors are not effective in limiting the warning times to the desired range, then the installation of gates should be considered.

Gates with Flashing Light Signals

Figure 11 shows the suggested guidelines for warning times at crossings with standard gates and flashing light signals.



[1] Times shown apply to crossings with no twin or triple tractor-trailer combinations. If twins or triples are expected, times should be increased by 10 percent.

[2] If more than 10 percent of the warning times exceed 60 seconds, installation of motion sensors or train predictors is recommended. If motion sensors or predictors cannot reduce warning times to the desired level, installation of four-quadrant gates should be considered.

Note: All warning times include gate delay/descent time.

FIGURE 11 Suggested warning time guidelines for crossings with gates and flashing light signals.

These values are consistent with the current minimum warning time of 20 sec in this country, and with warning time practices in many foreign countries (*6*). They are designed to (a) provide sufficient time for stopping; (b) minimize gate violations; (c) minimize CL10s and maintain adequate clearance times; (d) minimize unnecessary driver delay; and (e) promote driver confidence for gates with flashing light signals. The values also provide safe clearance time for those vehicles that by law must stop at all crossings.

The suggested minimum warning times range from 20 to 35 sec depending on the width and grade of the crossing. These values should be increased by 10 percent if twin or triple tractor-trailer combinations are present. The suggested ranges of warning times for gated crossings are relatively narrow, i.e., 5 sec. These narrow ranges are strongly supported by the research results. Recognizing practical limitations of train operations and train detection hardware, some longer warning times would be allowed. However, if more than 10 percent of the warning times exceed 60 sec, then the installation of motion sensors or train predictors is strongly urged. If motion sensors or predictors are not effective in limiting the warning times to the desired range, then the installation of four-quadrant gates would seem appropriate. However, at this time four-quadrant gates are not adopted in the MUTCD.

The gate delay and descent time should not be too long or drivers will try to beat the gate. The following guidelines for gate delay and descent times are suggested:

1. The total gate delay and descent period ideally should be 10–12 sec and should not exceed 15 sec.
2. The gate delay time (i.e., the time that the flashing light signals are activated before the gates are activated) should be approximately 3–4 sec. Slightly longer times may be justified if vehicle approach speeds are above 60 mph. This is consistent with accepted traffic engineering principles for the warning phase at signalized intersections, and would be appropriate at gated crossings (*8*).

ACKNOWLEDGMENTS

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Model of the Effects of Rail-Highway Grade Crossings on Emergency Access

TIMOTHY A. RYAN AND EVERETT C. CARTER

The purpose of this research was to develop a simple model describing the impacts of rail-highway grade crossings (RHGCs) on emergency access. Linear cities and two-dimensional cities with square grid roadway networks are considered. For the purposes of the model, maximum response time from the emergency services base stations to the most distant point in the service area was minimized. The model indicates that the introduction of an RHGC into an optimized condition requires each base station to be relocated toward the RHGC, to again achieve optimal conditions. It also reveals that the impacts of a rail line through a city vary greatly with the orientation of the rail line relative to the roadway grid. Suggestions for further model extension are presented.

A rail-highway grade crossing (RHGC) is an at-grade intersection of one or more railroad tracks and a roadway. At such a crossing, railroad vehicles and roadway vehicles must share the right-of-way. RHGCs are unusual in transportation engineering in two respects—first, at an RHGC, two different types of traveled way intersect and must time-share the right-of-way. This is not unique—where highways and waterways intersect at drawbridges, right-of-way is also time-shared. Secondly, at an RHGC, right-of-way is allocated between two competing flows by a continuous favoring of one flow (rail traffic) over the other (highway traffic), without regard to the volume of traffic on the highway. This last aspect is unique to RHGCs.

This continuous favoring of rail traffic results in delays to highway users. Such delays have quantifiable costs, including the time of the delayed motorists, additional vehicle operating costs, and costs of additional air pollution. These costs can be substantial, but are generally not catastrophic.

A special type of delay cost is incurred, however, when an emergency vehicle is delayed at an RHGC. Delays to emergency vehicles can, in the most extreme cases, result in the loss of human lives. In less severe cases, these delays can result in additional property damage (as in the case of fire apparatus being delayed in reaching the scene of a fire). These costs are not obvious, and frequently go completely unnoticed until they are incurred.

A review of the professional literature was conducted; no information related directly to this topic was found. This paper briefly documents a model describing the impacts of RHGCs on emergency access.

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MODEL DEVELOPMENT: LINEAR CITY CONDITIONS

Ideal Conditions, Without RHGCs

Assume the existence of a completely isolated linear city. The city is one block wide, with a single roadway extending through the entire length of the city. All emergency services (fire or medical) must be provided from within the city, because it is completely isolated. Further, assume that the demand for emergency services is distributed uniformly across the city, and that there are no impediments to transportation at a maximum speed of v in the city; in other words, the city is an ideal transportation surface. Emergency services must be provided from a single base station, and maximum response times (travel times to points most distant from the base station) are to be minimized. For the purposes of this model, minimizing the maximum response time was preferable to minimizing the average response time, to maintain consistency with typical fire protection agency policies. In Baltimore County, Md., for example, an effort is made to have first-due units in urbanized areas a maximum distance of 1.5 mi from the most remote points in their service areas.

In a setting such as this, shown in Figure 1, the logical choice for a single base station for emergency services would be C, the geographic center of the city. From such a base station, all points in the city would be reached within T, the maximum tolerable response time. Relocation of the base station to another point, say $C1$, would, of course, shorten some response times, but it would also raise some response

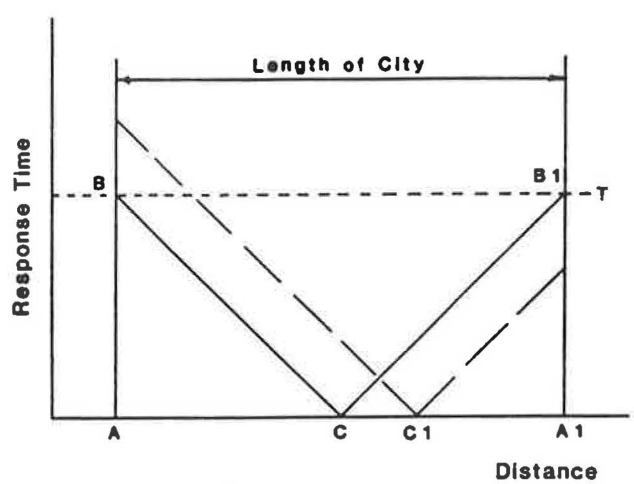

FIGURE 1 Response time versus distance, one base station.

times to intolerable levels. This is also shown in Figure 1 (dashed line); the absolute value of the slope of each line in this figure is v.

Strictly speaking, it may be proven that C is located at the center of the city. Because the absolute values of the slopes of the service time lines (BC and $B1C$) are identical, angle BCA is identical to angle $B1CA1$. In addition, $AB = A1B1$; furthermore, by definition, angle BAC and angle $B1A1C$ are identical, each being a right angle. Because two of the angles in triangle ABC are identical to two of the angles in triangle $A1B1C$, the third angle in triangle ABC (angle ABC) must also be identical to the third angle in triangle $A1B1C$ (angle $A1B1C$). Because the three angles and one side are identical for the two triangles, the other two sides of each triangle must be identical as well. Thus,

$$AC = A1C$$

and

$$AC + A1C = A1A$$

By substitution,

$$2AC = A1A \qquad (1)$$
$$AC = A1A/2$$

Thus, C is the center of the city.

Inclusion of RHGCs

Let us now relax the assumption of the ideal linear city, and assume that a single-track railroad extends across the entire city at any location other than the center of the city. Assume further that each time an emergency vehicle needs to cross this track it is blocked by a train, and is delayed for a time d. The results of this condition are shown in Figure 2, with the base station located at C, the original optimal location, and with the rail line located at F. Figure 2 shows the following:

1. Response times are completely unaffected for locations on the same side of the track as the base station.
2. All locations on the opposite side of the track from the base station will suffer an increase in response time. In fact, some locations beyond F will have response times greater than T (the former maximum).

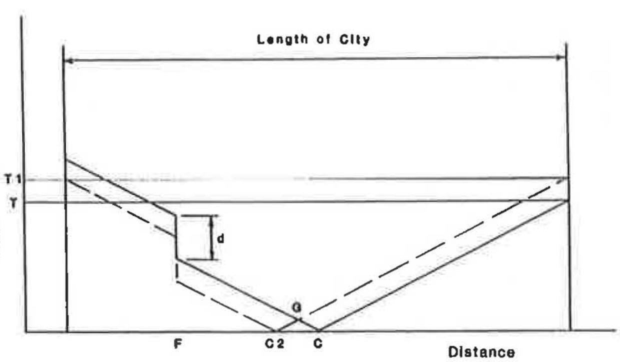

FIGURE 2 Response time versus distance, one base station with RHGC.

In order to optimize this modified system, and meet the objective that maximum service time is to be minimized, T needs to increase at both endpoints of the city. Because there are two endpoints, each needs to accommodate one-half of the delay d caused by the track. Thus, T will increase by $d/2$. In addition, because distance is directly related to travel time by v, the optimal location of the single base station will shift toward the railroad track as follows:

$$s = v(d/2) \qquad (2)$$

where s is the distance of the shift and the other variables are as defined before. The new optimal location will be $C2$, as shown in Figure 2.

The resultant response times are also plotted in Figure 2 (dashed line), and show that the maximum response time increases by $d/2$, to $T1$. In fact, response times increase for most individual points in the city, decreasing only for the areas between the track and G (the point at which the response time line for the original base station intersects the response time line for the relocated base station).

Thus, on the whole, the presence of the railroad causes a deterioration in response time for emergency service for the city as a whole.

Ideal Conditions, with Two Base Stations

Let us return to the ideal city shown in Figure 1, and eliminate the assumption that a single base station is needed. We replace this assumption with one that states that two base stations are to be used. Because the objective is still to minimize the maximum response time, the stations should be located such that

1. At the common boundary of their service areas, T is equal for each station. The value of T in the two base station scenario will be different, of course, than the value of T in the single base station scenario.
2. Response time at each of the city limits is T. For this city, the quarter points ($C3$ and $C4$, each located one-fourth of the length of the city from the city limits) are the optimal locations, as shown in Figure 3.

In essence, the two base stations simply divide the city in half. It may be proven that point H in Figure 3 is located at

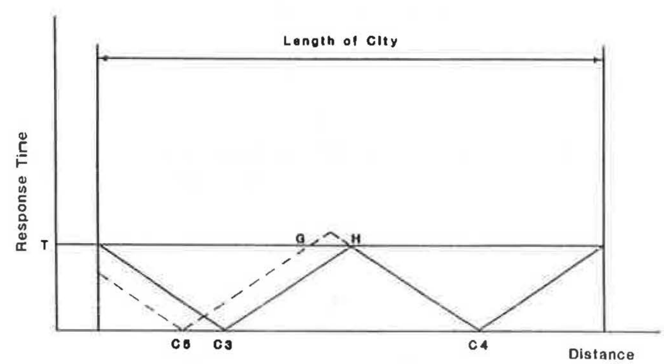

FIGURE 3 Response time versus distance, two base stations.

the center of the city by an approach similar to that used to show that point C in Figure 1 is located in the center of the city. Once it is established that H is located at the center of the city, the proof that $C3$ and $C4$ are located at the quarter points is identical to the proof that point C in Figure 1 is located at the center of the city.

Relocation of either base station will result in a response time exceeding T for some locations in the city. For example, a relocation from $C3$ to $C5$, as shown in Figure 3, will cause section GH to have response times that are larger than acceptable.

Inclusion of RHGCs

If, under the two base station scenario described above, a single-track railroad is assumed to cross the linear city at a point J located between the two base stations, and if it is further assumed that this railroad causes a delay of d for each emergency vehicle attempting to cross it, the condition shown in Figure 4 results. Figure 4 is quite similar to Figure 2, of course; the only difference is in the number of stations, and in the magnitude of the changes required to recalibrate the system.

Because there are four service area endpoints (two per service area), each needs to accommodate one-quarter of the delay d caused by the track. Thus, T will increase by $d/4$. Each base station will shift as follows:

$$s = v(d/4) \tag{3}$$

The new maximum response time is greater than the original maximum response time by $d/4$ in each station's service area. Thus, the presence of the RHGC causes a deterioration in response time not just for the area on the distant side of the RHGC from the station, nor just for that particular station's service area, but for the city as a whole. The only exceptions to this deterioration are some locations in the immediate vicinity of the relocated base stations; these locations will actually have decreased response times.

General Conditions for Linear City

It can be shown mathematically that, for n base stations, the optimal spacing between base stations is L/n, with the distance from the edge of the city to the nearest base station being given by $L/2n$. In addition, the shift toward a single RHGC by each base station in order to correct for a delay of d is

$$s = vd/2n \tag{4}$$

where all variables are defined as before. In addition, the change in maximum response time zT will be

$$zT = d/2n \tag{5}$$

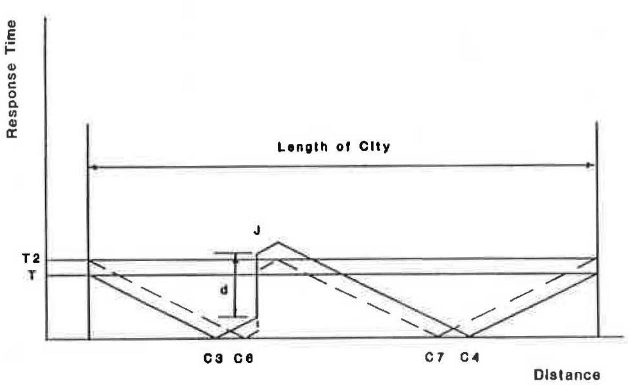

FIGURE 4 Response time versus distance, two base stations with RHGC.

MODEL EXTENSION: TWO-DIMENSIONAL CITY CONDITIONS

The preceding discussion is limited, of course, by the assumption that the city is linear. (It is also limited by the assumption that base stations can be instantaneously relocated, and that the demand for emergency services is uniformly distributed across the city. However, these assumptions are held for the following discussions as well.)

Conditions Without RHGCs

Let us now assume that the city in question is two dimensional; that is, having length and width but not height. Let us further assume that this city is not a transportation surface, but has a right-angle grid street system, and that this grid has been laid out such that each block in the grid is a square. Intersections are assumed to have no impact on response time, even if a turn is involved. All streets are two-way, and each block requires x units of time to traverse. Finally, let us assume that driveway entrances to the street network can be made only at the middle of a block, and that the maximum tolerable response time T is equal to $3.5x$ time units. (The coefficient 3.5 is arbitrary, and has been chosen for ease of presentation.)

Figure 5 shows these conditions and the service area for a single base station located at A. The service area thus defined is a diamond, with a triangular area equivalent to ¼ block

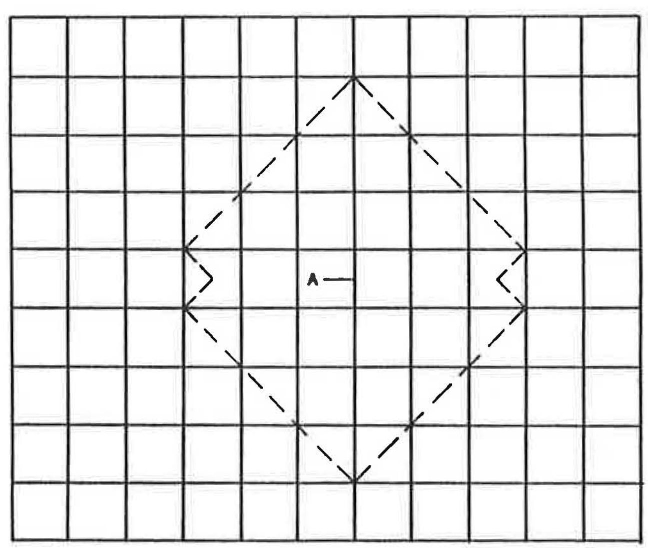

FIGURE 5 Service area boundaries with midblock entrance.

missing from each of the two points of the diamond on the axis at a right angle to the orientation of the access roadway for the station.

These triangular indentations pose a problem in efficient allocation of stations while fulfilling the objective function that $3.5x$ is the maximum acceptable response time. As Figure 6 shows, if the sides of the diamond are used as the boundaries between adjacent service areas, four stations will surround a small diamond that has an area equal to one block, and in which response times are greater than $3.5x$ by, at most, one time unit. Of course, the problems posed by this small diamond can be solved by locating the stations closer together or by redefining the maximum acceptable response time as $4.5x$. In either of these cases, however, the long sides of each service area will have response times less than T, thus resulting in an inefficient use of resources.

The cause of the triangular indentations in each station's service area is the assumption that entrances to the roadway network can occur only at midblock. If this restriction is lifted, and entrances to the network are permitted at intersections, the service areas for $T = 4x$ shown in Figure 7 result, and the indentation problem disappears. (The coefficient has been changed in order to allow service area boundaries to occur at intersections.) For simplicity in modeling the impacts of RHGCs, only this latter access scenario is considered in the following discussion.

Conditions With RHGCs

Let us now consider RHGCs in this analysis, again assuming that every emergency vehicle is delayed for a time period d at each RHGC. The impacts of RHGCs on emergency access are entirely dependent on the location and orientation of a rail line relative to the boundaries of each station's service area. For example, a rail line such as that shown in Figure 8 will have absolutely no impact on emergency access, because the rail line runs along the boundaries of service areas. Thus, no emergency vehicle needs to cross an RHGC. (In fact, this is a method used in practice quite frequently by providers of emergency services; service area borders are often defined by geographical barriers, such as rail lines or rivers.)

A rail line oriented as shown in Figure 9, however, will have a profound impact on emergency access. This line intersects each service area boundary at a 45-degree angle. Furthermore, because the line runs along one street in the right angle grid, there is no simple way to avoid it. Service areas could be redefined to set the track as a boundary, but this would result in inefficient shapes for those service areas abutting the track.

For ease of presentation, let us assume that $d = x$. The hatched areas in Figure 9 are those that can no longer be served within $T = 4x$, with all stations in their original optimal location. Assuming that no stations to the left of the track are relocated, the $T = 4x$ criterion can be met efficiently if all stations to the right of the rail line are moved a distance

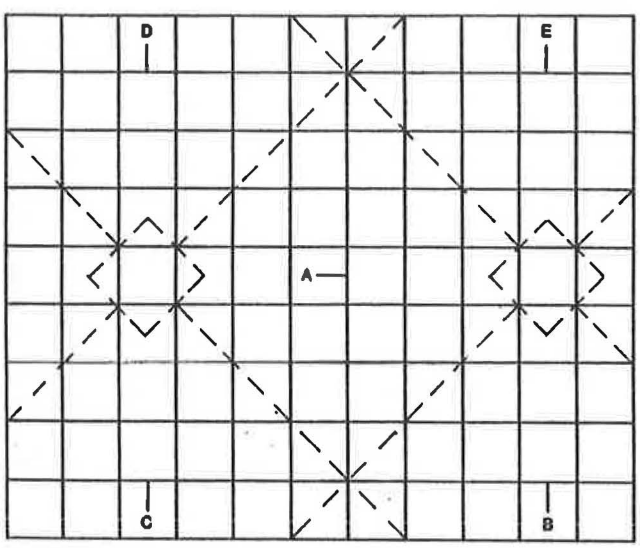

FIGURE 6 Multiple service area boundaries with midblock entrances.

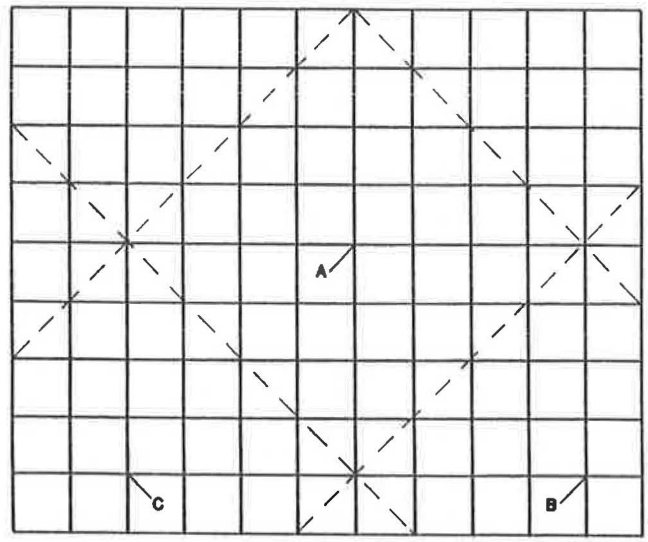

FIGURE 7 Multiple service area boundaries with intersection entrances.

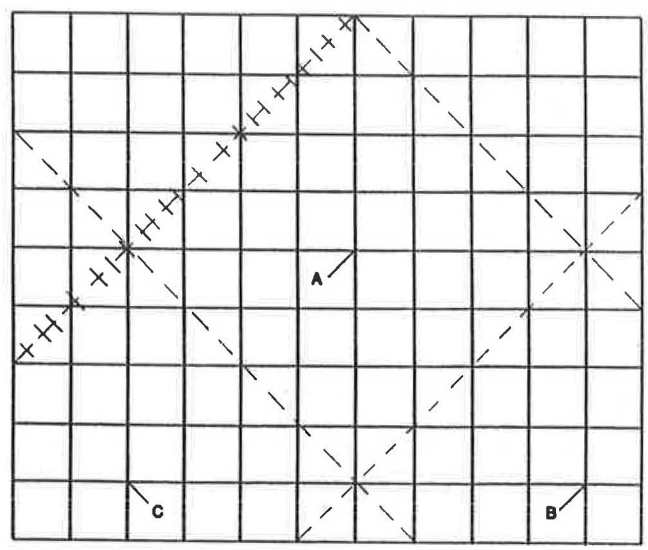

FIGURE 8 Rail line along service area boundaries.

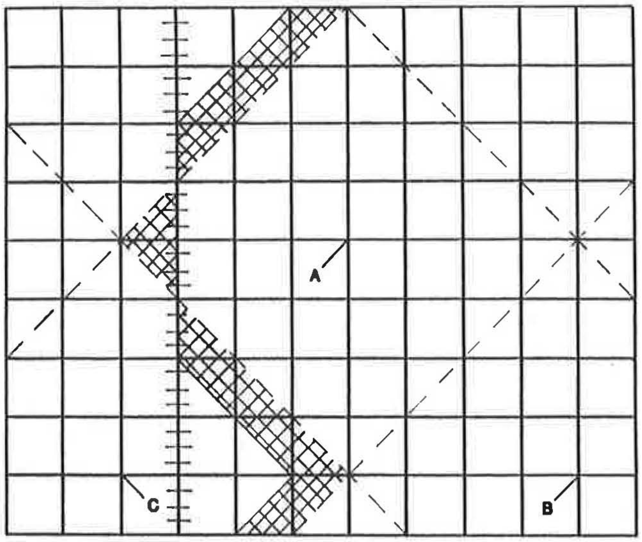

FIGURE 9 Rail line at angle to service area boundaries.

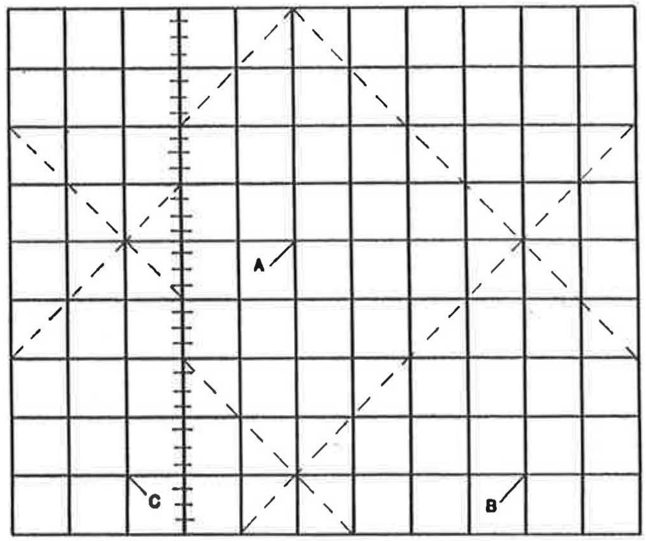

FIGURE 10 Relocation of base stations to right of rail line.

d closer to the rail line. This condition is shown in Figure 10. Of course, the shapes of the service areas through which the rail line extends are no longer diamonds. In addition, such a simplistic decision to shift all stations on one side of the RHGC ignores the system-wide effects. Recall that, in a linear city, all stations would be relocated to equalize the additional travel time through the system. Ideally, the same strategy should be followed in a two-dimensional city; however, unless each station moves a whole number of blocks, the stations will not access the grid network at intersections, resulting in inefficiencies, as described earlier.

As the preceding discussion indicates, the presence of a rail line through a two-dimensional city has the potential for causing severe complications in efforts to optimize the system.

Effects of Grade Separations

Let us now modify the type of system shown in Figure 9 to allow for a grade separation at G, as shown in Figure 11. The presence of this grade separation allows an emergency vehicle to bypass a blocked RHGC. Of course, the advisability of such a bypass depends on the relationship between the length d of the blockage and the time required to divert the emergency vehicle to the grade separation and back to the desired route. Clearly, unless the diversion time is less than d, there is no point in diverting the emergency vehicle.

If d is actually so large that it is advisable to use the grade separation in all cases, the portion of the service area for which $T \leq 4x$ is reduced, as shown by the dashed line in Figure 11. (Factors that could make d very large include a derailment, an accident at an RHGC, and switching operations.) In the event of such a long blockage, the grade separation at G becomes, in effect, a second base station for the area to the left of the track. That is, all emergency vehicles destined for this area must pass through point G, $2x$ time units after leaving the actual base station.

From the preceding discussion, it is clear that if a grade separation is available within a service area, the ideal location

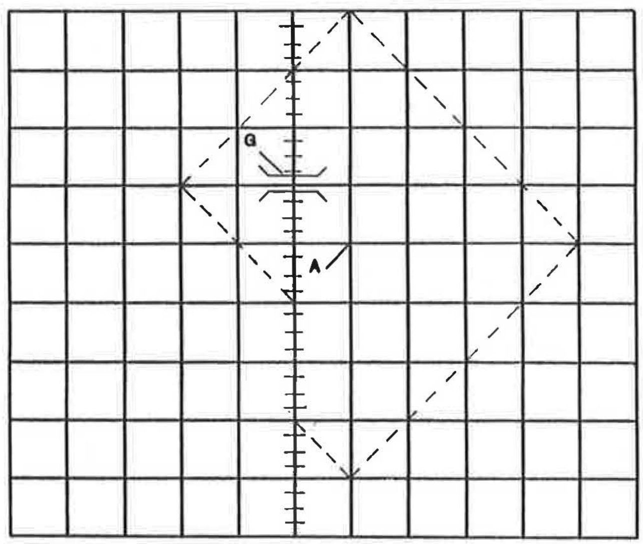

FIGURE 11 Grade separation at *G*.

for it is immediately adjacent to the base station. This location would eliminate the problem of travel time to the secondary base station. Thus, in Figure 11, under ideal circumstances, the service area would be shifted upward and to the left by one block.

SUMMARY

Given a city in which the demand for emergency services is uniformly distributed, and in which emergency vehicles travel at a uniform speed of v, the following conclusions may be drawn from the preceding discussion:

1. In a linear city of length L, n base stations should be optimally located such that L/n is the spacing between stations, and such that $L/2n$ is the distance from the edge of the city to the nearest base station.

2. In the same linear city, the presence of a rail line that causes a delay of d to all emergency vehicles can most optimally be addressed by moving all base stations by $vd/2n$ closer to the rail line. The maximum response time T will increase for each service area by $d/2n$.

3. In a two-dimensional city with a right angle grid roadway network, the optimal location for emergency vehicle access to any given block is at a corner. Midblock access results in inefficient station locations or in sections of the city not being served within time T.

4. In the same two-dimensional city, the impact on emergency access of the presence of a rail line is entirely dependent on the orientation of the rail line relative to service area boundaries. If the rail line runs along service area boundaries, there is no impact on emergency access. If the rail line cuts through service areas, however, significant impacts may result. If the rail line runs parallel to one of the axes of the grid, the system-wide impacts and optimal strategy to address those impacts are the same as for the linear city. However, the need to have base stations access the network only at intersections complicates implementation of this strategy.

5. The use by emergency vehicles of a grade separation at a random location within a given service area, in effect, results in a second base station. The optimal location for a grade separation is adjacent to the base station, so that the second base station impacts disappear.

SUGGESTIONS FOR FURTHER MODEL EXTENSION

There are, of course, numerous potential extensions of the model that are suggested, as follows:

Traffic Engineering Extensions

These potential extensions involve the inclusion of traditional traffic engineering parameters in the model. For example, the imposition of penalties in response time due to required turns or the presence of intersections would be useful. The model could also be expanded to three dimensions by consideration of roadway grades. In addition, explicit recognition that RHGCs are not always blocked by trains (and thus do not always delay emergency vehicles) and consideration of the stochastic nature of blockage times would enhance the model.

Basic Parameter Extensions

Perhaps the most interesting extension of the basic model would be one that recognized that demand for emergency services is not uniform across a service area, but rather varies in density. It should be noted, however, that such an extension is pointless under the original goal of minimizing maximum response time; the extension would have to include modifying the goal so that average response time is to be minimized.

Combination with Other Models

The basic model presented in this paper assumes that the locations of all RHGCs and grade separations are fixed; the only variable is the location of a given base station.

It would be useful if this model could be used in conjunction with other transportation planning models to determine the societally optimal locations for base stations, RHGCs, and grade separations. For example, construction of a grade separation only one block away from its originally proposed location might reduce emergency access costs substantially, while increasing construction costs and costs to normal roadway users only minimally.

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