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Analysis of Highway Accidents, Pedestrian Behavior, and Bicycle Program Implementation

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Investigation of Accidents on Alabama Bridge Approaches

DANIEL S. TURNER AND NEILON J. ROWAN

As part of a research project to examine bridge accidents in Alabama, an investigation was conducted to ascertain the effects of the approach roadway on bridge accident rate. The objective of this study was to determine whether the accident rate increased near bridges, and if it did, to determine whether the increase could be described by a standard statistical distribution. A sample of approach accidents was prepared by matching county, highway, and milepoint numbers from Alabama's bridge inventory and accident files. This difficult matching process was necessary because state accident-investigation forms do not record structure numbers for bridges that are involved in collisions. More than 24 000 accidents on state-route highways between 1972 and 1979 were used in the study. A unique distribution of accidents was observed at bridge ends. The average accident rate doubled over a 0.35-mile distance at the approach to a structure. This increase could not be identified as any standard statistical distribution, primarily because of investigating officers' preference for recording accidents to the closest one-tenth milepoint. Tenth-milepoint locations dominated the data and masked the true distribution. An examination of accident codes revealed that many Alabama bridge accidents are apparently investigated incompletely, identified improperly, recorded erroneously, or ignored due to limited space on the investigation forms.

A survey of the nation's 564 000 bridges shows that 100 000 structures are seriously deficient (1). Various estimates have indicated that at least 50 000 bridges need widening or replacing (2,3). Such statistics lead to a staggering estimate of the cost of correcting all bridge deficiencies. The 1980 Surface Transportation Act established funding at more than a billion dollars per year for bridge rehabilitation and replacement but, even at such an accelerated rate, it may take 25 years to cure the problem (4).

In Alabama, literally hundreds of bridges are candidates for federal funds. But, from its share of the billion federal dollars allocated to the states, Alabama can afford to replace only a few. The problem is to choose the most dangerous bridges so that they can be replaced first.

OBJECTIVES

This report documents one phase of a bridge accident investigation, an examination of the contribution of approach roadways to bridge accidents. One-way and two-way traffic structures on Alabama state-route highways were included in the study. Underpasses and culverts (with earth cover) were not identified as bridges.

Specifically, this portion of the research was designed to answer questions such as

1. Does the roadway accident rate increase near bridges?
2. If such an increase occurs, can it be described by a statistical distribution?
3. Should approach accidents be included in hazardous bridge studies? and
4. Can a statistical distribution be used to define how much of the approach roadway should be considered?

BACKGROUND

Bridges are inherently more dangerous than the roadways on which they are located. Mitchie generalized that bridges are 50 times more hazardous than roadways (1). He used 1975 data to compare the ratio of fatal ran-off-road, hit-fixed-object type of collisions to gross roadway mileage (5,6). He then found a similar ratio for fatal bridge or bridge-barrier accidents to cumulative bridge mileage. The bridge fatal accident rate was found to be 50 times larger. Although specific inferences should not be drawn from such a generalized analysis, it does serve to demonstrate the drastic increase in the potential for accidents caused by the structures.

Prediction of Bridge Accidents

Two procedures have been suggested as ways to predict accidents: (a) observations of driver behaviors and (b) analysis of historical accident data. Bridges are known to exert an influence on the behavior of drivers as they approach and to cause both lateral displacements and changes in speed. The lateral movement case has been recognized and studied for some time (7-9). Typically, these studies involve observation of a vehicle's lateral position at some distance from a bridge, then a second observation near the structure. The movement of the vehicle toward or away from the centerline has been shown to be a general indicator of how dangerous drivers perceive the bridge to be. Unfortunately, no strong correlation has been identified between lateral movement and bridge width, nor has the relation between lateral movement and accident rate been quantified. A logical assumption would be that these movements could cause an increase in traffic accidents on bridge approaches.

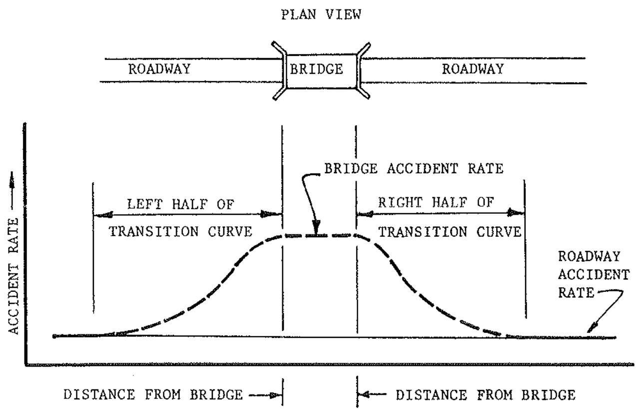
The second method of predicting bridge accidents is by use of historical accident data. In recent years, researchers have examined accident records rigorously in an attempt to isolate those factors most significant in causing bridge accidents. Bridge width, approach-roadway width, sight distance, traffic volumes, alignment, approach barrier, bridge rail, traffic control devices, approach speeds, and pavement surface conditions have all been shown to contribute to accidents. The complex interaction of the multiple contributing factors has made it difficult to define a single method to realistically predict bridge accidents. There has been a general agreement on major factors such as the primary importance of relative structure width and traffic volumes; however, the majority of factors that influence bridge accidents has not been quantitatively defined. At least four of the research projects used accidents on bridge approaches during their studies (8,10-12). Lengths of 500-1200 ft were most commonly used in these projects.

The examination of literature showed that bridges have higher accident rates than the roads on which they are located, that vehicles frequently shift lateral position as they approach structures, and that previous researchers have used various approach distances in analysis of bridge accidents.

Accident Rate Transition

The exact role of the bridge approach (and departure) has not been previously defined. The accident rate does not change abruptly at the beginning of the structure. Rather, a transition must occur as vehicles approach the more dangerous location. A logical assumption would be that the increase in accidents would follow some statistical pattern, such as the normal distribution shown in Figure 1. The figure illustrates that a normal curve may be split at the mean value and one-half placed on each

Figure 1. Suggested accident rate for transition curve.



end of the bridge to form a smooth transition. The mean value for the distribution would be the rate of bridge accidents, although it is possible that approach accidents might occur more often than collisions on the structure. The tail of the distribution would approach the roadway accident rate. The area under the curve would represent the excess accidents (beyond the roadway rate) caused by the bridge structure. Knowledge of the existence, magnitude, and character of the statistical distribution of accidents on bridge approaches would lead to vastly increased accuracy in bridge studies.

STUDY PROCEDURE

To carry out the study, it was first necessary to identify approach collisions. A computer program was prepared to compare accident milepoints and bridge-end milepoints. The program gathered data from the Alabama bridge inventory file, including highway number, county number, milepoint of bridge beginning, and bridge length. The highway, county, and milepoint numbers form a unique designation in the Alabama numbering system. This combination was compared with the highway, county, and milepoint numbers on accident records to match accidents to bridges. For purposes of this study, the approach was defined as the direction of increasing milepoints, and the departure was defined as decreasing milepoints.

During the course of a normal accident investigation, Alabama law-enforcement officers are directed to specify the accident location by highway and milepoint. The officer's training requires that such information be recorded to the closest one-hundredth of a mile (13). A comparison of such accurate data for accident milepoints and bridge-end milepoints should produce a good distribution of distances for an analysis of approach accidents. Accident data were used for all state-route highways for the period 1972 through 1979 to ensure a large and meaningful sample.

The milepost-matching procedure was not without problems, however. One of the complicating factors is that an accident that occurs between two closely spaced bridges occurs on the approach to one bridge and on the departure of the other. Establishment of which bridge was the most significant in causing the accident is very difficult. A bridge could cause an erratic maneuver that results in an accident at the following bridge. In that case, existing records would not assign the accident to the correct location. In addition to the previously described data, travel direction, distance between structures, and many other causal factors would have to be examined to see which of the bridges instigated the accident. Even if a method could be selected to review these data and assign locations, the complexity in developing a computer program makes the procedure unattractive. For study purposes, individual accidents were assigned to the nearest bridge-end milepoint.

To identify exact accident locations, the roadway was searched in incremental lengths away from bridge ends. The unit length was selected as 0.05 mile after some initial analyses indicated that such a length was appropriate. The computer would read the bridge data and calculate beginning and ending milepoints for the approach, for the bridge, and for the departure. The program would then search a sample of 24 000 accident records by milepoint to determine how many occurred at the particular bridge. Next, a new bridge record would be input and the process would be repeated. After all bridge records were examined, the total approach, bridge, and departure accidents were output for the incremental approach length under consideration. The program was repeated for several approach lengths up to 0.35 miles to develop the desired distribution of accident distances from bridge ends.

STUDY RESULTS

The computer program was run for 0.05-mile increments seven different times. When the accidents within 0.35 mile of bridge ends had been merged with the appropriate structures, the results were tabulated for analysis. For example, during the initial computer run for a 0.05-mile increment, 696 accidents were found on bridges, 575 were found on approaches, and 477 were found on departures. When the program was executed with an increment of 0.10 mile, 1027 approach accidents and 1024 departure accidents were noted. The additional collisions noted in the second run represented accidents that occurred between 0.05 and 0.10 mile from bridges.

Initial Analysis

The results of the computer analysis are displayed in Table 1 and Figure 2. The sample contained 24 000 accidents that occurred on state routes in Alabama. More than 25 percent (6049 out of 24 000) were found to be within 0.35 mile of a bridge. Of the 6049 matched collisions, 696 occurred on bridges (approximately 3 percent of all accidents). The number that occurred on approaches, 2645, was almost exactly the same as the 2708 that occurred on departures. Assuming that these accidents comprise a normal distribution, the mean location would be 0.004 mile from the departure bridge end, and the standard deviation would be 0.18 mile. Such characteristics seem to reflect the type of distribution assumed by Figure 1.

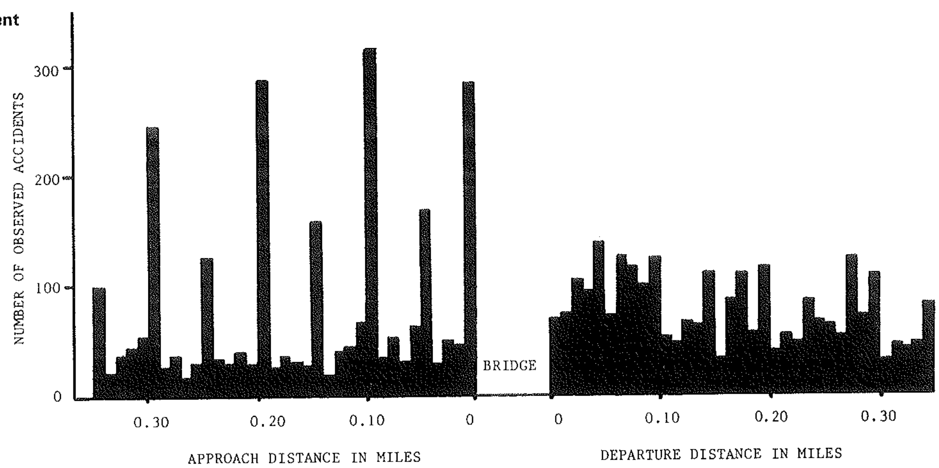
Distribution Patterns

Two things are immediately noticeable about Figure 2. First, the approach accidents follow an unusual and repetitious pattern. This pattern can be traced to an obvious cause. Investigating officers tend to record accident milepoints to the nearest 0.1 mile. This would seem natural since most mileposts are at mile intervals and automobile speedometers measure in tenths of miles. Most officers probably locate the accident milepoint by driving from the milepost to the accident while observing the automobile speedometer. In Figure 2, officers clearly favor use of 0.1-mile distances, and about half as many accidents are recorded in between the tenth-mile locations as officers estimate to the closest 0.05

Table 1. Tabulation of bridge approach and departure accidents.

Distance From Bridge End (mile)	Observed Accidents Approach	Observed Accidents Departure	Distance From Bridge End (mile)	Observed Accidents Approach	Observed Accidents Departure
0.00	135	0	0.19	25	57
0.01	147	70	0.20	290	114
0.02	46	72	0.21	33	38
0.03	50	104	0.22	40	54
0.04	30	94	0.23	30	47
0.05	167	137	0.24	33	83
0.06	33	71	0.25	127	66
0.07	26	127	0.26	29	64
0.08	52	115	0.27	18	53
0.09	33	100	0.28	35	123
0.10	316	123	0.29	27	72
0.11	66	52	0.30	246	109
0.12	44	48	0.31	54	31
0.13	40	65	0.32	44	46
0.14	19	62	0.33	37	42
0.15	154	109	0.34	21	45
0.16	28	44	0.35	101	82
0.17	31	84	Total	2645	2712
0.18	38	109			
Subtotal	1455	1586			

Figure 2. Approach and departure accident distribution.



mile from automobile speedometers. A relatively small number of accidents were recorded to the closest 0.01 mile. This finding imposes a constraint on any statistical inferences drawn from the data.

Approach and Departure Differences

The second thing that draws immediate attention in Figure 2 is the distinct difference in the approach and departure observations. The departure distribution seems to follow the type of random pattern that would be anticipated from accident statistics rather than the tenth-mile pattern so obvious on the approach. This is not the case, however. The departure dispersion was caused by the manner in which the bridge-ending milepoint was calculated. All bridge-beginning stations are recorded in the Alabama bridge inventory file to the nearest tenth mile, as reflected by the approach accident pattern. The computer calculated the bridge-end station from the starting point plus the bridge length. The majority of bridge-ending points thus fall on hundredth-mile stations, instead of tenth-mile stations, like bridge beginning points. If ending stations are calculated to hundredths and accidents are to the tenth, a different pattern could be expected from that of the bridge approach.

The overall distribution of Figure 2, although difficult to analyze, indicates an increase in accidents at bridges. On the approach side, the accidents occurring at each 0.1-mile location, as well as at the midpoints between these locations, increase as the bridge is approached. A similar, although not as obvious, arrangement may be discerned from a study of the departure side.

Larger Class Intervals

The use of 0.01-mile increments tended to confuse rather than simplify the analysis. For that reason, the data were grouped into 0.05 mile units to aid in the interpretation. Figure 3 and the table below represent such a grouping.

Study Interval Miles From Bridge End	Adjusted No. of Bridge Accidents Approach	Departure
0.000-0.050	575	477
0.051-0.100	460	536
0.101-0.150	323	336
0.151-0.200	412	408
0.201-0.250	263	288
0.251-0.300	355	421
0.301-0.350	257	242
Total	2645	2708

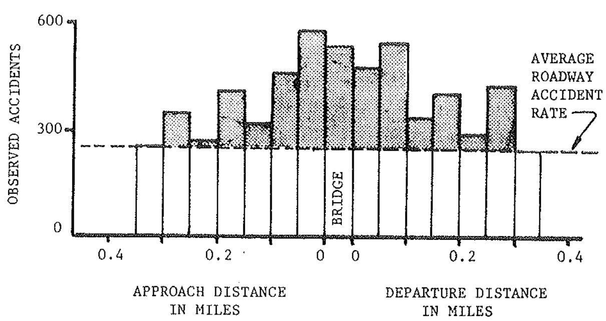
The preponderance of accidents recorded at the one-tenth points is still evident even when the data are grouped. The symmetrical pattern of Figure 3, with every other bar raised, clearly reflects the officer's preference for tenth milepoints. The number of accidents on bridges and the average bridge length in the sample were used to calculate the rate for accidents that occur on the structure, which is shown by the dotted line on the figure. The dotted line agrees nicely with the adjacent approach and departure rates. The number of accidents decreases as distance from the bridge end increases. This is the anticipated result and represents the transition from the bridge rate to the roadway rate. The type of transition is not intuitively obvious from either Figure 3 or the table above. A chi-square test was performed on the hypothesis that the data were taken from a population that has a normal distribution. The hypothesis was rejected. A similar test indicated that the distribution was not linear.

Control Group

In order to further examine the approach and departure distribution and to estimate the number of accidents that would have occurred at study sites if bridges had not been present, a control distribution was established. An equivalent amount of randomly selected highway, county, and milepoint numbers was designated as theoretical bridges and were computer matched against the original sample of accident records. Two things were accomplished: (a) a control distribution was obtained for comparison with the bridge accident distribution, and (b) an average roadway accident rate was obtained for randomly selected sites. Table 2 contains the random control site results. The tenth-point accidents are even more pronounced than the bridge-site accident distribution. This suggests that officers are slightly more prone to pinpoint the location of bridge accidents than roadway accidents. Collisions are not grouped around the control sites as are bridge-approach accidents. Thus, the control site distribution accomplishes the first objective by demonstrating the uniqueness of the bridge approach distribution.

Although the same number of bridges were used for the control group, the randomly generated highway-county-milepost numbers did not always correspond to hazardous locations on Alabama highways. This produced a smaller sample size for merging of control site bridges with accident data.

Figure 3. Excess accidents caused by bridges.



THE SHADED AREA INDICATES "EXCESS" ACCIDENTS CAUSED BY BRIDGES.

Excess Accidents Caused by Bridges

A better analysis of bridge approach and departure accidents might be to examine only those locations where the accident rate is higher than the average roadway rate. Since the roadway rate was determined through the control group, the excess accidents caused by bridges could be identified.

The excess accidents associated with bridges do not seem to fall into any conventional distribution. The table below lists the number of accidents in each distance interval around bridges.

Study Interval (miles from bridge end)	Excess Observed Accidents: Approach	Excess Observed Accidents: Departure
0.000-0.050	321	223
0.051-0.100	206	282
0.101-0.150	69	82
0.151-0.200	158	154
0.201-0.250	9	34
0.251-0.300	101	167
0.301-0.350	3	0
Total	867	942

The mean accident location was 0.013 mile on the departure side and the standard deviation was 0.147 mile. These values are very close to the values for the initial distribution. For the distance class used, the one-tenth point collisions continued to dominate. Two attempts were made to overcome the tenth-milepoint bias of the data and identify the actual distribution.

Smoothed Distribution

The initial effort involved smoothing the sample by distributing the one-tenth point accidents to adjacent intervals. The logic behind the smoothing was that officers recorded the locations as the closest tenth point, but an accident so recorded would have an equal probability of actually oc-

Table 2. Accidents at control site.

Distance From Bridge End (mile)	Observed Accidents: Approach	Observed Accidents: Departure	Distance From Bridge End (mile)	Observed Accidents: Approach	Observed Accidents: Departure
0.01	4	10	0.16	5	8
0.02	2	7	0.17	0	19
0.03	1	4	0.18	3	11
0.04	1	9	0.19	1	31
0.05	9	0	0.20	174	29
0.06	1	8	0.21	1	1
0.07	1	10	0.22	0	5
0.08	3	16	0.23	3	4
0.09	2	39	0.24	0	7
0.10	215	25	0.25	16	13
0.11	2	1	0.26	2	8
0.12	0	2	0.27	0	6
0.13	2	4	0.28	0	11
0.14	2	12	0.29	1	7
0.15	21	12	0.30	3	28
Subtotal	266	159	Total	475	347

Figure 4. Grouped accidents.

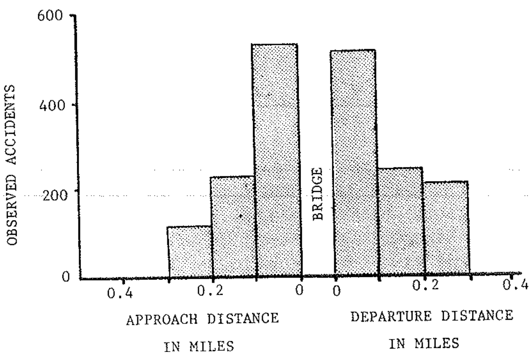


Table 3. Accident codes for approach accidents.

Alabama Accident Code	Frequency	Percentage
No entry	146	22.2
Inattention	121	18.4
Run off road	6	0.9
Run off road and overturn	1	0.2
Before overturning	42	6.4
Skidding	38	5.8
Other collision	227	34.2
Passing	25	3.8
Avoiding other vehicle	9	1.4
Before vehicle submerging	3	0.5
Hit bridge rail	44	6.7
Hit bridge abutment	34	5.2
Hit culvert or headwall	4	0.6
Hit embankment or ditch	18	2.7
Hit guardrail	19	2.9
Hit median barrier	3	0.5
Hit other object	19	2.9
Hit animal	29	4.2
All other codes		9.7

curring at any of the adjacent hundredth points on either side of the one-tenth point. The technique used to distribute the data focused on the difference in the number of observations in a particular interval and the observations in the intervals on either side. The tenth point had a weight of 0.5 and the adjacent intervals had weights of 0.25 each for distributing the extra tenth-point accidents. The smoothed accidents are shown in the table below. The total departure accidents, total approach accidents, mean accident location (0.013 mile from departure end of bridge) and standard deviation (0.149 mile) are almost identical to the distribution prior to smoothing.

Study Interval (miles from bridge end)	Smoothed Accidents Approach	Departure
0.000-0.500	283	243
0.051-0.100	201	217
0.101-0.150	126	150
0.151-0.200	98	106
0.201-0.250	69	97
0.251-0.300	53	92
0.301-0.350	37	42
Total	867	947

The smoothed accident tabulation was tested to see if it conformed to a recognizable statistical distribution. The chi-square test was applied at a 95 percent confidence level to the hypothesis that the sample came from a normally distributed population. The hypothesis was convincingly rejected, primarily due to the large irregularity near the outer edge of the observed accident distribution. The sample was also compared with Poisson, Erlang, and binomial distributions with the same result. Finally, the approach and departure tabulations were each compared with the Poisson and a negative exponential population. The comparisons were again rejected, although the negative exponential distribution came closer to matching the sample than did any previous distribution.

Larger Data Intervals

An alternative to smoothing the data is the use of larger increments for the frequency tabulation. This was done by using a 0.10-mile grouping. The results are shown in the table below and Figure 4.

Study Interval (miles from bridge end)	Observed Grouped Accidents Approach	Departure
0.000-0.101	527	505
0.101-0.200	227	236
0.201-0.300	118	201
Total	864	942

The large increase in collisions at the bridge ends is immediately obvious from the figure. The first 0.10 mile dominates the drawing. It also appears that the figure could be reasonably approximated by a statistical distribution. The chi-square test was used to investigate the normal, Poisson, Erlang, and negative exponential distributions. All comparisons were rejected, as had been the case for previous attempts to identify the data sample as a standard statistical distribution.

TYPES OF ACCIDENTS

An additional investigation was conducted to see whether the character of accidents changed from bridge approach to departure. The descriptive codes on Alabama accident reports, collision diagrams, and explanatory reports were used to compare accidents at the three locations (13,14). The approach and departure collisions were virtually identical in nature, as would be expected. Approximately 10 percent of these accidents were identified as types that might be associated with bridges. Examples are hit bridge rail, hit bridge abutment, before vehicle submerged, and hit headwall. A categorical grouping of approach accident codes is shown in Table 3. About 6 percent of the accidents that occurred between the bridge approach beginning and ending milepoints were coded as hit bridge rail or hit bridge abutment. This is 50 percent as large as the number of corresponding type accidents for bridges. An additional 5 percent of the table was coded such that bridge accidents were implied; however, the vast majority of the collision codes was either noncommital or suggested something other than bridge accidents. In comparison with approach accidents, there were slightly fewer entries for the following codes for bridge accidents: no entry, hit tree, hit pole, inattention, and during passing.

That most bridge accidents are apparently not identified as such is significant. Many collisions caused by bridges are probably not identified by officers due to the single data entry point on the investigation forms. A complex accident may have several causes, or a sequence of events may precede the wreck. The investigating officers choose and record only one. Officers apparently do not place great emphasis on identifying the events that surround an accident, since one-fourth of the forms had blank entries. Other reasons for the discrepancy between the number of bridge-associated accidents and the corresponding coded descriptions could be erroneous data code entries, misidentified milepoints, and incomplete investigations.

CONCLUSIONS

During the investigation of the effects of bridge approaches on accident rate, 24 000 accidents that occurred in Alabama between 1972 and 1979 were compared by highway, county, and milepoint for 960 bridges. The most significant findings were as follows:

1. Researchers have previously recognized that bridge approaches cause an alteration of driver behavior through modification of vehicle lateral placement and speed. Other researchers have used various a roach distances during bridge accident analyses. None of the previous research had identified a quantifiable relationship between approaches and accident rates.
2. Although Alabama investigation forms provide for the recording of accident locations to the closest hundredth of a mile, officers record them to 0.1-mile points on more than half of the cases. Around bridges the tendency to measure them more closely (to the 0.01 mile) is increased.
3. The recording of accident location data to the closest 0.1 mile tempers the significance of any statistical analysis applied to such data.
4. There is an inherent difficulty in assigning an Alabama accident to the correct structure where bridges are in close proximity due to overlapping approaches and departures.
5. One-quarter of the traffic accidents in Alabama occur within 0.33 mile of a bridge.
6. An analysis of nonbridge control sites indicated that there is a unique distribution of accidents for bridge approaches and departures.
7. There is a definite transitional increase in accidents on bridge approaches and departures. The maximum rate occurs at the bridge abutment and is more than twice the rate of the adjacent roadway.
8. The increase in accidents apparently reaches 0.35 miles from bridge ends. The precise beginning of the transitional pattern could not be identified due to the complexity of approach and departure overlap.
9. The grouped tabulation of approach distances (see Figure 3) could not be identified as any standard statistical distribution. Normal, Poisson, Erlang, negative exponential, binominal, and linear distributions were rejected. A negative exponential distribution came closest to matching the data. The exact distribution was masked by the tenth-milepoint predominance, extensive overlap of approaches and departures at the tail of the distribution, and inability to establish the absolute base accident rate for the roadway. The distribution could not be identified in spite of repeated frequency groupings and smoothing attempts.
10. The character of accidents that occurred on approaches is virtually identical to that of accidents that occurred on departures. The nature of collisions on the structure is slightly different from that of approaches and departures, with a greater emphasis on hit bridge rail and hit bridge abutment types of accidents.
11. For accidents that occurred on bridge structures as identified by milepoint, only 12 percent are directly labeled as bridge hits by the data coded on accident investigation forms.
12. Many bridge accidents are apparently incompletely investigated, not properly identified, erroneously recorded, mislocated, or ignored due to limited room for identifying information on accident investigation forms.

ACKNOWLEDGMENT

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Study of Ran-Off-Roadway Fatal Accidents in Louisiana

OLIN K. DART, JR., AND LAWRENCE S. McKENZIE

Thirty ran-off-roadway single-vehicle fatal accidents in an eight-parish area of Louisiana were studied by an interdisciplinary team. This team consisted of a highway engineer and the research director (authors of this paper), an automotive specialist, a psychologist, a sociologist, and two state police officers. Three-fourths of these accidents occurred on curved two-lane rural highways. More than one-half of the accident sites had roadside hazards within 20 ft of the pavement. No vehicle defects were found to cause any accident. Young, white, male drivers who have no more than a high school education and came from a working class family were particularly susceptible to an accident of this type. An epidemiological conceptual model of accidents was adopted to evaluate the interaction of the agent (vehicle), the host (driver), and the environment (physical and social). The evaluation showed that, although the vehicle and highway can contribute to the severity of an accident, the human factor is the major contributor to the accident. Countermeasures to reduce the number of ran-off-roadway accidents will require strategies for changing the behavior of susceptible drivers, primarily through law enforcement and educational programs. Reduction of the severity of these accidents will require manipulation of situational (engineering) factors related to the vehicle and physical environment.

Within the past decade the relative number of single-vehicle fatal accidents has been increasing in the nation and state. Single-vehicle accidents currently account for approximately 40 percent of highway fatalities nationwide and almost as large a percentage in Louisiana.

Ran-off-roadway fatal accidents in rural areas of Louisiana have attracted the attention of the Louisiana Highway Safety Commission because of their increase since 1973 (Figure 1). Because of the seriousness of the problem and in light of the paucity of information regarding this particular type of accident, the Louisiana Highway Safety Commission initiated a preliminary research project of an interdisciplinary nature designed to shed light on the factors related to single-vehicle ran-off-roadway fatal accidents.

STUDY OBJECTIVES

The Louisiana Highway Safety Commission research project had several specific objectives, all of which related to reducing highway accident numbers and fatalities in the state and nation.

The first objective was to develop a comprehensive interdisciplinary research approach to the investigation of accidents. This was because accidents appeared to occur because of human responses and behavior as well as because of mechanical and environmental factors. Therefore, social scientists were included on the study team to broaden the base of accident investigative procedures.

The second objective was to establish procedural guidelines to ensure the collection of as much data as possible pertinent to the study from each accident investigated. Certain data critical to the determination of the cause of accidents were not normally heretofore collected. For example, very little information on the behavior of the driver prior to an accident or on the sociocultural background of the driver was routinely obtained.

The third and final goal was the development of strategies and recommendations for accident prevention programs. Given that human behavior and physical factors combine in some way to produce an accident, it was considered reasonable to assume that educational processes could be devised to raise the consciousness of drivers to accident-causing factors. Thus, the goal was to obtain as much background information on the driver of vehicles involved in accidents as possible.

STUDY PROCEDURE AND METHODOLOGY

The study was designed as a pilot investigation. We decided that the first 30 subject cases, which occurred in rural areas of the State Police Troop A region after July 1, 1977, would satisfy the requirements for a preliminary study. (Rural was defined to include all territory outside the incorporated boundaries or limits of towns that have a population of 2500 or greater.) The methodological procedures for the study were designed accordingly.

The study area served by Louisiana State Police Troop A is shown in Figure 2. This area contains widely differing geographic features. On the west side of the Mississippi River and south of Baton Rouge, the terrain is typical of flood plains and contains extensive swampy areas. By contrast, the part of the Troop A region that falls to the north and east of Baton Rouge is characterized by rolling hills and expanses of pine and hardwood forests. Driving conditions are therefore somewhat different in the two parts of the region. Beyond geographic differences, there are also rather distinct cultural variations in the region. The inhabitants of the delta areas along the Mississippi River tend to reflect the Cajun culture. That is, they are likely to be of French or Spanish descent, Catholic, and to have distinct social patterns in such things as food habits and music. Those persons who live in the uplands are more likely to be of Anglo-Saxon or Scotch-Irish heritage and to belong to protestant denominations. They are characterized as more puritan in their life-styles.

The location of the 30 case accidents is shown in Figure 2 so that an association with geographic and cultural features can be made. A rather easily followed definition of an accident was essential since cases had to be determined to be suitable for the study:

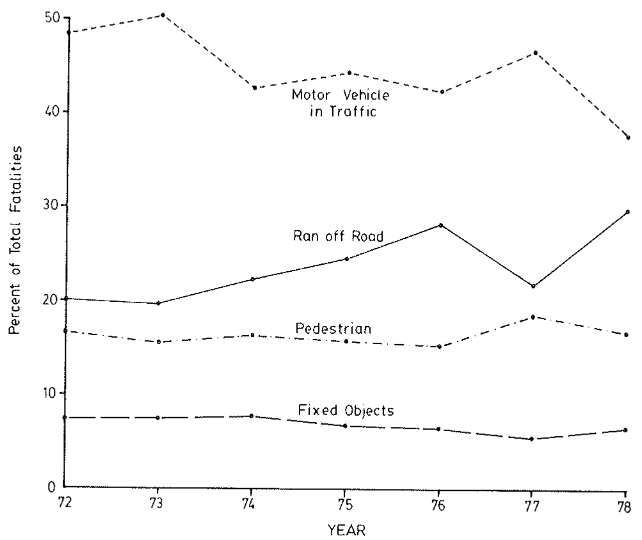
1. The accident must have occurred within the geographical boundaries of the study area,
2. The accident must have produced a fatality,
3. Only one vehicle was involved in the accident, and
4. The final accident requirement was that the subject vehicle must leave the roadway totally (pavement or other surface) at some time during the development of the accident.

Since one of the objectives of the study was to develop an interdisciplinary approach to accident investigation, a team of specialists was recruited to investigate each accident. This team included professionally trained and experienced individuals from the disciplines of psychology, sociology, civil engineering, and automotive mechanics. The interdisciplinary group, as a team, was given the responsibility for identifying data requirements, collecting relevant data, and analyzing the data relative to each accident, both individually and collectively. A research specialist coordinated the team's work and two specially trained state police sergeants were assigned to work with the team. The latter gave special assistance in the reconstruction of each accident and in verification of the data collected at accident sites.

Before the field part of the study was launched, a protocol was established to alert and notify the accident investigation team of the occurrence of an accident. All state police troopers were advised to notify headquarters whenever they believed a fatal accident met the study criteria. If the accident reported was determined to meet the criteria of the study, one of the two sergeants assigned to the study team was contacted by state police headquarters or by the Louisiana Highway Safety Commission and given the circumstances of the case. The project director was then contacted and he alerted other team members.

Note that several accidents (5) occurred on parish (county) roads during the study period. These cases were investigated by sheriffs' deputies rather than by state police. In such instances the team did not receive direct notification of the accident. The first information on the accident was received through news media or from the fatality accident reporting system (FARS) officer at the Louisiana Highway Safety Commission. This procedure had a drawback in that an extended period of time elapsed between team notification and the occurrence of the accident. Once it was determined that such an accident met the criteria of the study, the same follow-up procedure was initiated.

Figure 1. Proportion of highway fatalities by select accident categories, Louisiana, 1972-1978.

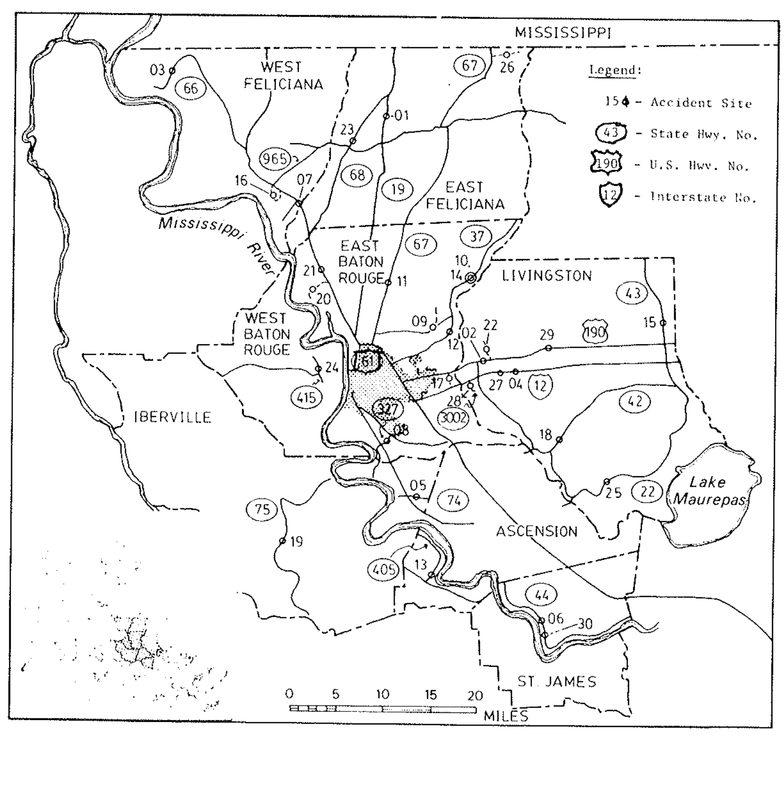


The investigative procedure on an accident may be briefly described as follows. Primary data on each case were collected at the scene of the accident. Standard vehicle accident report forms were used to obtain basic information from the driver, occupants of the vehicle, and witnesses. These forms were completed or checked by one of the state police sergeants assigned to the team. The project director and the engineer generally accompanied the troopers to the accident scene. They made first-hand observations and participated in the collection of data. In certain cases the team psychologist also visited the scene and interviewed the principals involved.

A survey crew under the direction of the project engineer was dispatched to each accident site as soon as possible to map and locate relevant features implicated in or caused by the accident. The project engineer also obtained the road construction plans and accident history records of accident sites from the Louisiana Department of Transportation and Development. Some time after the accident, the subject vehicle was inspected for mechanical defects by the automotive specialist. Usually this was done at the place where the vehicle had been towed.

Detailed information about each driver was obtained at a later date through personal interviewing. The interviews were conducted by using a questionnaire prepared by team members in consultation with representatives of the Louisiana Highway Safety Commission. In all but two cases interviews were taped. At a later date the tapes were studied

Figure 2. Study area and location of 30 case accidents.



carefully by team members. In those cases where the driver was killed in the accident, interviews were conducted with persons who knew him or her well. Usually this was next of kin, but on occasion more distant relatives, close acquaintances, or surviving occupants of the vehicle provided information on drivers.

Blood alcohol test results and driver record files for drivers were obtained through the state police, as was vehicle licensing information.

All information obtained about the accident was employed to reconstruct the sequence of events that led to each accident. A psychological and sociological profile of each driver and the mechanical condition report of each vehicle were prepared as part of the study procedure.

During the course of the study, periodic conferences were held to discuss accident findings and research problems. These conferences included all members of the research team plus representatives of the Louisiana Highway Safety Commission. The discussions that took place aided greatly in placing each accident in a truly interdisciplinary perspective.

Altogether, a total of 32 cases were studied from May 1977 to September 1978. The first two cases were worked in a pretest phase of the study. The purpose of the pretest was to verify and correct deficiencies in the methodology and protocol and to refine research procedures.

The research phase of the study began July 1977 and ended September 1978. As noted, a total of 30 cases met the accident criteria and were investigated. These cases were and are considered as a pilot research effort in use of a case-history and an event-reconstruction approach. The findings and analyses presented in the remainder of this report must be interpreted accordingly.

ACCIDENT CHARACTERISTICS

Although no 2 of the 30 accidents occurred in precisely the same way, there were similarities among a number of the accidents. These similarities provide points of departure for the understanding and study of ran-off-roadway single-vehicle fatal accidents. The number of cases is too small for statistical generalization; however, we think that the findings generated are highly enlightening.

Place and Time of Accidents

At least one accident occurred in each of the eight parishes in the study area, nine each occurred in the parishes of East Baton Rouge and Livingston (see Figure 2).

Of the 30 accidents investigated, nearly one-half (43 percent) occurred on a Saturday and approximately three-quarters (73 percent) occurred during the weekend (Friday-Sunday). The preponderance of accidents on weekend days was determined to be related to trip plans associated with leisure activities. This finding suggests an association of accidents with type of trip and helps explain the greater probability of alcohol as a contributing factor. Nearly one-half (47 percent) of the accidents covered in this study occurred after 9:00 p.m. but before 6:00 a.m. With only one exception all of the accidents during this time period (night) were on weekends. Forty percent of the case accidents occurred between 3:00 and 9:00 p.m. These accidents were nearly evenly divided between weekdays and weekend days.

Driver Characteristics

In addition to normal demographic data, the sociologist and psychologist on the study team compiled additional case history data on most of the 30 drivers involved. The median age of the drivers was 22; 40 percent were younger than 21 and only two were over 40. Only two drivers were female. Only 20 percent were black compared with 35 percent of the area population.

Well over half of the drivers had not finished high school. Fifty-four percent were classified as semi-skilled or unskilled laborers, none as professional. Thirty-five percent were single, twenty-three were married, and the rest had unstable family life.

Almost none of the drivers participated in formal organizations. Seventy-five percent belonged to a church, but only 5 were active members. Over half were engaged in outdoor recreational activity to a moderate extent. Fifty-eight percent led rather normal social lives. All drivers came from a working class background.

Among the deviant and substance-abuse behavior found, eighty-nine percent of the drivers consumed alcoholic beverages; fifty-eight percent were heavy drinkers. One-third used drugs, mostly marijuana. More than half smoked. Seventy-seven percent had one or more traffic violations. Well over half had unstable work habits; 40 percent were highly unstable.

Attitudinally, the drivers were classed as nonpathological (60 percent). These drivers are not deliberate risk takers and are not overtly hostile to the law and society. They are disinclined to drive defensively with a view to preventing accident situations from occurring; they are inclined, instead, to place undue reliance on quick reaction time to get them out of difficulty.

Quite another attitudinal pattern was evidenced by those drivers classified as macho (18 percent). These were deliberate risk-takers. Drivers classified as sociopathological (14 percent) presented personal histories wherein active hostility was directed toward social convention and authority figures (father, law enforcement officers, or teachers).

Other contributory psychological factors were inexperience (only one case), fatigue (40 percent) (established for five and surmised for seven cases), psychotic episodes (two cases), acute stress without psychotic episodes (seven cases), and depressive personalities experiencing acute stress just before accident (four cases).

Characteristics of Vehicles Involved

Vehicle characteristics were derived from information collected at the scene of the accident and from follow-up inspection conducted after the vehicle had been removed from the accident scene. Chevrolet made 57 percent of the vehicles. Sixty percent were two-door sedans. Median vehicle age was six years. The owners drove 70 percent of the vehicles. None of the accidents was attributed to vehicle defects. Two-thirds were being driven above the speed limit. The seat belts in over half the vehicles were removed or rendered unusable.

Roadway Characteristics

The general highway characteristics include roadway type, previous accident history, geometric design features, and the presence of roadside obstacles. For the 30 accident sites in this study, 83.3 percent were state maintained and all but two were two-lane, two-way highways.

Most of the highway sections (±1 mile) did not

Figure 3. Left-hand curve site of two ran-off-road fatal accidents.

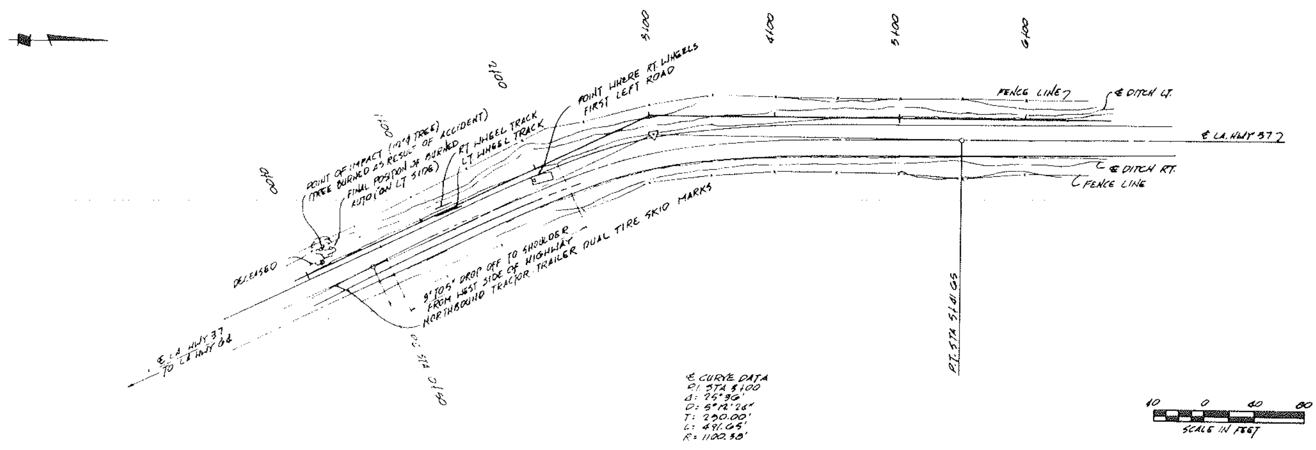
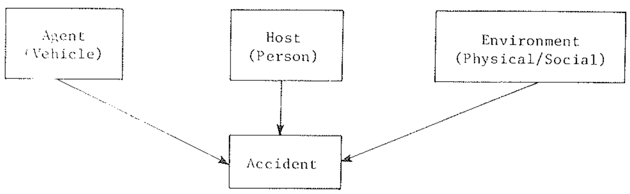


Figure 4. Epidemiological model for ran-off-road, single-vehicle fatal accidents.



have a serious accident history (3 years before study case). Twenty-eight percent had more than three accidents per year. Thirty-two percent had no previous ran-off-road accident. Only two sites had more than one ran-off-road accident.

Eighty-seven percent of the highway sections had at least one substandard design element. Sixty-three had substandard pavement width, 70 percent had substandard shoulder width, and 53 percent were deficient in both pavement and shoulder widths. Seventy-seven percent were curves (6° median curvature): 39 percent had excessive curvature and 44 percent had inadequate superelevation. Roadside hazards and obstacles (i.e., sharp ditches, trees, poles, culverts, or fences) within 20 ft of the pavement edge were found at 57 percent of the highway sections. Twenty-three percent of the accidents happened at bridges, and 71 percent of these locations had inadequate guardrails.

Of the curved roadways involved, 65 percent curved to the left. This corresponds to previous studies of this subject like that of Wright and Robertson (1). An example of such a site is illustrated in Figure 3. This location was the only section where two fatal accidents occurred during the course of this study.

ANALYSIS OF ACCIDENT CAUSAL FACTORS

Obviously many accidents represent a complicated sequential chain of events that can be related to several contributory factors or variables. These factors are exceedingly complex in that they range in degree from the physical (road or weather) to the mechanical (characteristics of the vehicle) to the human (personality traits and social and cultural background). Because of the way these variables interact, accidents appear to be capricious in and of themselves. But, accidents are a recurring phenomenon and, as such, have a relatively high degree of predictability. However, prediction addressed to accidents has usually been limited to the number and types of accidents, number of fatalities, and immediate characteristic of drivers. This study, within the limits of its pilot or case nature and localized setting, presumed to go somewhat further and to isolate, identify, and describe the predictable aspects of the precrash milieu that more or less predestine the accident. Said another way, a certain combination of physical, mechanical, and human factors is necessary for an accident to happen. The objective was not only to determine the nature and importance of these contributory factors but also to gain an understanding of the likelihood of interaction between them that would culminate in an accident-causing syndrome.

Conceptual Framework

The conceptual perspective that provided the initial orientation for the analysis of study findings is known as the epidemiological model. This model has been used successfully in many previous studies (2). Epidemiology, as an investigative and analytical technique, was developed primarily in the medical field. This conceptual approach has been modified for application to nondisease injury and fatality experience such as accidents. The rationale for the use of the model in accident investigations is that human behavior and experience follow some sort of epidemiological pattern.

The epidemiological conceptual model of accidents that was adopted focused on three basic components: (a) a host, (b) an agent, and (c) an environment (Figure 4). Within the context of the type of highway accident under consideration, the host was the person or persons involved in the given accident, the agent in the accident was the motor vehicle, and the environment was the sum of the physical and social conditions present that contributed to the accident. The physical parameters of accidents included weather conditions and the roadway conditions and design. The social parameters included the set of sociological and psychological variables that comprised the social (or human) aspects of the accident.

In a given accident all three sets of variables were investigated. The relevant information necessary to ascertain or infer causal relation was collected. The epidemiological model provided the frame of reference for the analysis that follows. This analysis was designed to answer questions related to the cause of accidents and to predict future accident occurrence.

Relative Importance of Factors that Contribute to Accidents

The first analytical procedure was to determine which of the overall components or factors appeared to be predominant in the cases under study. Thus, careful attention was given to the agent (vehicle), environment (physical and social), and host (driver). The factors discovered fall far short of telling the whole story of accident causation when considered independently. Until the factors are related in some sort of interaction matrix that provides an understanding of the relative importance of each component in the specific accidents and for accidents in general, the picture of accident causation will remain unclear. In keeping with this thought, each of these sets of components was considered in turn and evaluated.

Vehicle

The vehicle was the agent in which the host (driver and passengers) experienced an accident that resulted in a fatality. Logically, there is justification for suspicion of mechanical malfunction or failure as the major contributing cause of accidents. For none of the 30 vehicles associated with accidents was there positive and unmistakable evidence of primary responsibility for the accident. Although some of the vehicles were not in the best mechanical condition (three had more than 100 000 miles) and about half had tires that showed heavy wear, no clues were found that would suggest breakdowns or failures possible of leading to an accident. The vehicles, in and of themselves, were simply the agent for the accident and not a major contributory factor to the accident. This is not, of course, to rule out some breakdown once the sequence of the accident was under way. Obviously, worn tires and tied down seatbelts could add to the severity of the accident. All in all, in terms of the epidemiological model, there was no justification for assigning vehicles more than a minor role in accidents. This conclusion suggested that a close look be given to the environmental and host elements of the model.

Physical Environment

Two variables in the physical environment were carefully investigated, including ambient conditions and roadway conditions. In both instances, some evidence suggested association with accidents.

The most obvious clue to ambient conditions was that a large percentage of the accidents occurred at night. Night driving tends to be more hazardous than day driving. Other than this variable, no aspect of ambient conditions appeared to contribute to accidents. There were no incidents of storms, very little rain involvement, and no fog conditions were reported.

Roadway conditions at some sites were determined to be potential contributors to accidents. We found that 26 (87 percent) of the sections of roadway where accidents occurred had at least one substandard geometric element. These included any one or a combination of narrow pavement, narrow shoulders, sharp ditches close to pavement, trees and utility poles close to shoulders, excessive curves, and superelevation problems. Curves to the left were the most troublesome sites.

Roadway defects could not, except circumstantially, be given direct accountability for accidents. But there was little doubt that in some instances the unforgiving nature of the road contributed to the sequences that ended in fatalities. A tired driver, an inattentive driver, or a driver under the influence of alcohol or drugs was more likely to have an accident on a substandard roadway than on one that was in keeping with the safety standards recommended by highway engineers.

Social Environment

It was quite clear that drivers that had certain sociopersonal characteristics and certain social backgrounds were involved more frequently in accidents. The more immediate, specific sociopersonal indicators of accidents were age, sex, race, educational attainment, and working class background. Taken together these characteristics were important predictors of the drivers involved in the accident cases studied. In this regard, it appeared that a young, white, male driver who had no more than a high school education and who came from a working class family was particularly susceptible to an accident.

Psychological traits of drivers that were found to be related to accidents included substance abuse (primarily alcohol), speeding, deviant behavior (such as arrest records and unstable work habits) and certain attitudinal patterns (macho, hostility, or inattentiveness). In only one or two instances were psychopathological conditions suspected.

Beyond the sociodemographic and psychological characteristics noted above, the social environment surrounding an accident included another important social variable. This was the immediate precrash activity of the drivers. Weekend accidents related closely to precrash activity. Approximately three-quarters of the accidents were associated with leisure precrash activities, most of which involved the consumption of alcoholic beverages. The investigation turned up cases where the subject driver had left a lounge, party, outdoor recreational activity, or was simply joy riding and drinking. In only two cases were drinking drivers involved in work-related precrash activities. Both of these cases occurred after 5:00 p.m.

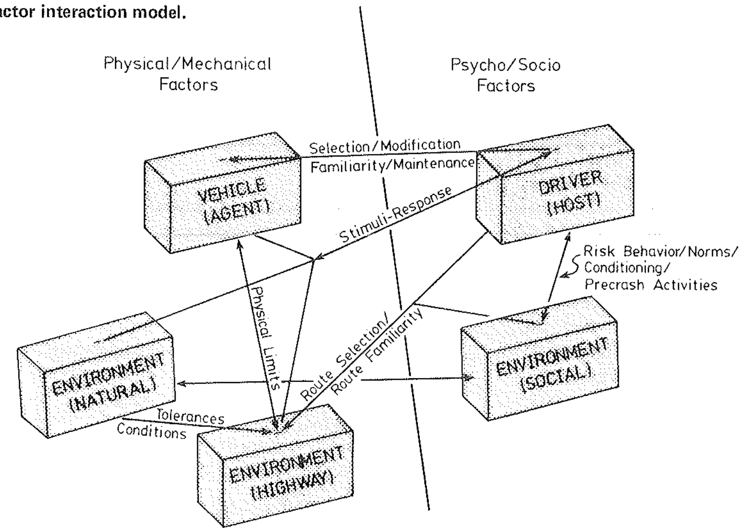
Drivers

In an overview sense, the sociodemographic and psychological characteristics of drivers, including their styles of life, tended to follow three major patterns. The first type of driver was termed the macho type. Many of the drivers had internalized acts such as drinking and speeding as an acceptable way to prove their manliness. Obviously such ideas are traceable to a socialization process (i.e., learning experiences received in the home, in work groups, or in other social groups). The young appear more susceptible to such patterns of behavior, which bring recognition and acclaim among their peers, than do older persons. At any rate, drinking and certain types of risk-taking could be seen as a part of the style of life of the macho driver.

The second type of driver clearly identifiable was classified as an inadequate performance type. This class of driver was generally described as moderate, dependable, and conservative in his or her behavior. The cause of their accidents was a breakdown in their performance as drivers that was not directly associated with drinking or speeding. Beside inexperience, fatigue and lapse of attention apparently accounted for inadequate performance, especially in more-demanding situations. The number of drivers in this class was not large, about one out of five, or 20 percent.

The third type of driver isolated was termed a physical or psychological breakdown type. Such drivers had undergone a complete physical or psycho-

Figure 5. Accident factor interaction model.



logical collapse that rendered them helpless as drivers. For example, one person apparently fell asleep while driving and a second seemed to have a mental breakdown that placed him in a trance. In the cases of three drivers, there was some suspicion of a physical breakdown.

Overall Assessment and Conclusions

We must reemphasize that broad generalizations cannot be made from the study findings because of the limited number of cases and the limited area of investigation; however, the information gathered and analyzed was of sufficient nature to warrant the design of an accident factor interaction model. This model, shown in Figure 5, is based on the theoretical (epidemiological) model presented in Figure 4. The model shows how, in an analytical sense, a given accident is the result of the interaction of factors in the epidemiological model. This interaction, when it achieves a certain state, predestines an accident. In light of study findings, the human factor in the model (social environment and psychological characteristics of the driver) is depicted as the overriding interactional or causal variable in accidents in general.

Given that the engineering and maintenance of roads and highways, weather conditions, and vehicle malfunction are situational variables that are contributory to accidents and must be addressed if accidents are to be reduced, the basic thrust of a preventive effort must still be concerned with the human factor. In this regard, we must conclude that the socialization processes that an individual undergoes and the opportunity structure that he or she experiences largely account for the behavior that leads to an accident. In more precise terms, none of the drivers was from a middle or upper class background. This pattern may be unique to the area studied, but it is one that appears to need intensive study. It strongly suggests that the human factor is mainly responsible for accidents. Thus, accidents will persist at rates that are too high to tolerate unless remedial action can be taken to change or alter the behavior of accident-prone individuals.

RECOMMENDATIONS FOR COUNTERMEASURES

Despite the small sample size of this study, we can identify countermeasures and intervention measures that might reduce accident frequency. From the theoretical standpoint taken, accident reduction is seen to entail two major countermeasure options that have both short-term and long-term implications:

1. Education or control of the driver through law enforcement and
2. Protection of the driver (and occupants) by manipulating situational factors related to the vehicle and physical environment.

The short-term human factors immediately and directly identified with accidents included the following:

1. Alcohol and drugs,
2. Speeding,
3. Inattention,
4. Fatigue,
5. Inexperience,
6. Poor judgment, and
7. Psychopathology.

Short-term physical factors are road and vehicle design condition. Long-term human factors include particular socialization experiences of individuals or, more precisely, an inadequate sociocultural background. Long-term physical factors include the redesign and improvement in the standards of vehicles and roads.

Countermeasures to reduce accidents in the relatively near future must address the immediate human factor and short-term physical factors that cause accidents. Countermeasures aimed at long-term control must look at programs of an educational nature administered over time and to safer vehicles and roads. In recommending countermeasures of both types, we are keenly aware that what is involved is the alteration of human attitudes and behavioral patterns of drivers and many others. Many of these patterns are normative within given cultures and will be difficult to change. This will make intervention extremely difficult. In this regard some of the recommendations include control measures over which there may be serious political and social-philosophical conflict:

1. Substance abuse control--Individuals who insist on driving while under the influence of alcohol or drugs have been the subject of much study (3-6). Their preponderance among accident-involved drivers suggests the need of more-effective control. Early revocation of license and other severe penal-

ties should be carefully considered.

2. Inadequate performance control--Countermeasures for this type of driver must be directed toward reducing the incidence of careless driving, of driving while fatigued, and of overreaction by inexperienced persons. The only feasible countermeasure appears to be the imposition of serious penalties for individuals detected in violation of a traffic regulation because of such failures. This may appear a harsh measure for errors that are not necessarily willful, but the frequency with which such behavior is associated with accidents necessitates some preventative measure.

3. Vehicle control--The contribution of vehicle factors to accidents was minimal among the cases investigated. Yet, there appears to be some justification for tougher legislation and controls related to the regulation of tire tread depths, use of safety restraints, and general vehicle maintenance. The recommendation is for police to make vehicle inspection a higher-level priority and enforcement of violation penalties more stringent.

4. Physical environment control--The roadway sites where accidents occurred were determined to have a considerable number of substandard features. Many of these features, such as lack of proper guardrails or of railings on bridges, can be easily and quickly remedied. We thus recommend that measures be taken to correct these deficiencies along all roads as quickly as possible. Attention should also be given to improving traffic control devices and delineation of hazardous locations. Priority should be given to those roads that have higher rates of accidents. Other needs, such as improved shoulders, curves, and grades, should be identified and made a part of longer-term planning and development.

Short-Term Countermeasures

The issue of control of the problem driver has been debated since the beginning of automobiles and continues to be a controversial subject. However, legislative requirements for the revocation of licenses and other penalties that would effectively limit the number of problem drivers would undoubtedly reduce accidents. A screening mechanism for identifying problem drivers would have to be developed in order to implement this recommendation.

Speed has also been a perennial problem related to driver control. The most effective short-term countermeasure is more-stringent enforcement of posted speeds, particularly on two-lane highways. This measure entails more police personnel, but it should go somewhat further. Multiple offenders should receive penalties sufficiently severe (including the revocation of licenses) to alter their driver practices.

Long-Term Countermeasures

Because of the obvious association of accidents with the social and cultural background of individuals, a massive socialization effort must be mounted to change driver behavior. This means that educational material must be generated for use in schools and the media and comprehensive informational programs must be planned. In this regard, emphasis should be placed on reaching youngsters from working class families. Although such a program is ambitious in character, it is not unrealistic. Many examples of programs that have changed behavioral patterns, such as practices related to health and disease control, can be cited. However, such an effort must be planned for long-term benefits and is quite costly in nature.

Observations showed that the techniques used by police in the handling of driving violation cases are not maximally effective in dealing with some types of drivers. Thus, attention should be given to the development of a simple, rapidly administered, on-site screening technique to enable the officer to determine whether he or she is dealing with a macho or sociopathic personality. (This screening is analagous to procedures used in airport security screening that entails human factors.) Further, officers should be given instruction in suitable postapprehension behavior for such violators. This countermeasure would involve the use of social scientists in the development and validation of tests and in the conduct of training courses.

The recommendation for control of the physical environment is simply that all new and reworked roadways be made to conform to standards that will minimize accidents. This would include parish (county) roads as well as state and federal-aid highways.

Long-term planning in terms of vehicle control must involve careful study of the mechanical features that tend to minimize accidents such as rollovers. This type of measure would eventually involve the establishment of design criteria for manufacturers. Again, such an accomplishment is challenging and would require the enlisting of informed advice from appropriate specialists and the support of the populace at large.

CONCLUSION

This study has highlighted that ran-off-roadway, single-vehicle fatal accidents are attributable to vehicle, human, and physical and social environmental factors. These factors interrelate in various ways to trigger a particular accident. Almost always, however, the human factor is the major contributor to the accident. For this reason, the accident rate cannot be reduced unless strategies for changing the behavior of those individuals most prone to have accidents is initiated. The planning and implementation of programs to modify human behavior plus the action that must be taken to make the physical environment of roads and mechanical characteristics of vehicles less accident prone will require a sound fund of information and data. The implication of this preliminary investigation is that more systematized research be directed toward the accident phenomenon studied.

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Abridgment

Costs of Operating Aircraft for Rural Traffic Enforcement

RICHARD A. RAUB AND BOBBY L. HENRY

This paper describes the cost of operating airplanes for law enforcement, including traffic patrol, in rural areas. Included are all costs associated with ownership and operation of airplanes, cost for pilots and support personnel, and the ground-support costs associated with enforcement of traffic laws. Total costs are approximately $96/h. Of this, 54 percent represent direct operating costs for the airplane, including fuel, periodic maintenance, and depreciation. An additional 34 percent of the costs are for salaries of the pilots, and the remaining 12 percent cover the overhead costs. When used for line patrol, the airplane costs $1.33/mile. Based on productivity of the pilots in Illinois, the cost per stop initiated by aerial patrol is $35. The use of the airplane solely for enforcement of the speed limit at fixed locations costs approximately $22/stop. Line patrol of highways with aircraft can be cost effective when compared with the same type of patrol by the officer on the ground. For enforcement of the speed limit at selected locations, an aircraft is substantially more expensive than a comparable operation that uses a radar operator and chase cars. A team of officers, including a radar operator, can perform the same task at approximately one-half the costs.

Although the hourly operating costs of airplane operation are high, the speed and coverage of airplanes make them practical to use for certain types of rural law enforcement. Aircraft are particularly superior for coverage of large areas. The area viewed from an aircraft for manhunts, searches, and general surveillance far exceeds that from the ground. The equipment, however, must be operated for traffic law enforcement in order to help offset the cost of purchase and storage.

Except for a report completed for the Illinois State Police (ISP) in 1979 (1), most other studies of aircraft costs have included only the direct costs of operation. Costs for fixed-wing aircraft ranged from $7.00-$43.76/h (2,3). Hourly costs for operating helicopters ranged from $23.01-$119.64 (2-5). The higher costs of helicopters tend do limit their use to metropolitan areas where the ability to hover and land at practically any location help outweigh the higher costs. The primary defect with the study for the ISP in 1979, which included an hourly operating cost of $137.42, was that it examined such costs under a specific operating policy. The costs in this report, which were derived from the methodology of the 1979 report, are presented in a more general fashion.

OPERATING COSTS

The operating costs for the aircraft include costs for depreciation, hangars, commodities, fuel, oil, and maintenance. Personnel costs are separated in the table below into the fixed cost of the chief pilot and secretary and the hourly costs of the 14 police officers who are the pilots:

Operating Cost	Item
Fixed	Chief pilot and secretary
	Hangar and office insurance
	Charts and other
Variable	Pilot salaries
	Depreciation
	Fuel and oil
	Periodic maintenance
Ground support officers	Drivers
	Assistants

Excluded from the cost of the police officers is their training and supervision on the basis that these same costs would be incurred if they were not flying. On the other hand, costs for pilot training are included. Finally, the costs of ground support are added. Such support is required to cite a traffic violation, investigate a disabled vehicle, or handle an accident reported by a pilot.

Fixed costs for the ISP aerial patrol in FY 81 (July 1, 1980 to June 30, 1981) were $83 950. Approximately 50 percent of those costs were for personnel. Variable costs added another $594 750, for a total annual cost of $678 700. Of the variable costs, 38.3 percent were costs for pilots. During FY 81, the seven aircraft in the fleet were flown 7080 h in law enforcement. More than 1000 additional hours were flown for maintenance, proficiency checks, training, and meetings, but these are considered a fixed cost of operation. Thus, based on the 7080 h of operation, Table 1 shows an average hourly cost of $95.80.

Added to the costs of operating the aircraft are those of ground assistance associated with enforcing

Table 1. Summary of airplane costs (FY 1981).

Cost	Total ($)	Cost per Hour of Law Enforcement ($)	Percentage of Total
Fixed	83 950	11.86	12.4
Variable			
Pilots	227 970	32.20	33.5
Depreciation	72 290	10.21	10.7
Fuel and oil	166 340	23.49	24.5
Periodic maintenance	128 150	18.10	18.9
Total	678 700	95.86	

Table 2. Cost of aerial patrols.

Item	Line Patrol[a] ($)	Speed Enforcement[b] ($)	Item	Line Patrol[a] ($)	Speed Enforcement[b] ($)
Airplane	450.54	249.24[c]	Cost per hour	159.36	247.54
Ground	164.97	105.76	Cost per stop	35.33	22.30
Court	133.46	140.08	Cost per mile	1.33	
Total	748.97	495.08			

[a]Cost calculations based on 4.7 h of work, during which 565 miles are driven and 21.2 stops made.
[b]Cost calculations based on 2.0 h of work, during which 22.2 stops are made (three officers each assist at 3.7 stops/h).
[c]The airplane also flies 0.6-h trip to and from the zone.

Table 3. Cost for alternate modes (ground officers).

Item	Line Patrol[a] ($)	Radar Team[b] ($)	Item	Line Patrol[a] ($)	Radar Team[b] ($)
Costs			Cost per hour	29.59	96.10
Ground	156.07	141.00	Cost per stop	49.32	11.86
Court	6.70	51.19[c]	Cost per mile	1.09	
Total	162.77	192.10			

[a]Cost calculations based on 5.5 h of work, during which 149 miles are driven and 3.3 stops made.
[b]Cost calculations based on 2.0 h of work, during which 16.2 stops are made (three officers each make 2.7 stops/h).
[c]Radar operator and arresting officer must attend court.

traffic laws and other services to the motoring public. How the assistance is given makes a difference in how the cost is assigned. The table below shows the costs when an officer leaves a patrol, assists the pilots, then returns. It also shows the cost where the officer serves as an interceptor for speed enforcement. The costs are based on personnel costs of $12.11/h and automobile costs of $0.28/mile. Some costs are also associated with driving to make the stop or to set up a speed-enforcement detail.

Item	Cost ($)
Single Traffic Action	
Driving	2.80
Officer's time	5.05
Total	7.85
Two-Hour Speed Enforcement Detail	
Driving and set up	11.03
Officer's time	24.22
Total	35.25

One final element is the cost of court attendance. Data from ISP records show that the officer spends an average of 2.1 h in court/appearance. If the officer is a pilot, that appearance often involves a flight to and from the county where court is held. Therefore, the cost for the pilot also includes flight. On the average, 3.9 percent of all traffic citations are contested. In 1980, of the 26 600 citations issued by pilots, 1040 were contested. The table below shows that the average court cost for each citation is $1.58 for the ground officer and $4.73 for the pilot.

COST FOR USE OF AIRCRAFT

ISP have used the aircraft for two different types of traffic enforcement: line patrol of Interstate highways and speed enforcement at marked zones (6,7). The cost for each of these operations differs. For line patrol, the pilot will cover 565 miles of highways and initiate 21.2 stops for traffic law violations. As shown in Table 2, a shift of line patrol that includes 4.7 h of flying will cost $748.97 or $159.36/h. On the other hand, a 2-h segment of speed enforcement by three ground officers who assist as interceptors will cost $247.54/h. The line patrol costs $35.33 for each stop made; speed enforcement costs $22.30 for each stop made.

COSTS FOR ALTERNATE MODES

A single officer on line patrol of an Interstate will cover 149 miles, will initiate 3.3 stops for traffic violations, and will cost $159.36/shift of 5.5 h. As shown in Table 3, the cost per hour and mile of patrol is less than that of the airplane, but the cost per action taken is 40 percent higher. However, to match the activity of a pilot in an aircraft would require 6.4 officers on the ground at a cost of $1041.73 compared with the cost of $748.97 for the airplane.

Item	Court Appearance: Ground Officer	Court Appearance: Pilot
Travel		
Time (h)	0.6	1.0
Cost ($)	15.17	95.86
In court		
Time (h)	2.1	2.1
Cost ($)	25.43	25.43
Total cost ($)	40.60	121.29
Cost per citation ($)	1.58	4.73

Although the line patrol by aircraft may be more cost effective than conventional ground patrol, the same does not appear true for speed enforcement. The average cost for a stop use of a radar operator and three intercepting officers, even though each officer is less productive, is estimated to cost approximately one-half the amount per stop as the cost of an airplane. For the airplane to achieve a common economic position with a radar and chase car team would require a minimum of a threefold increase in productivity on the part of officers who assist the airplane. Given the time required for an officer to process a citation, such a threefold increase would not appear possible. On the other hand, because the airplane is rarely detected (as opposed to a radar operator), it has an intrinsic value in terms of identifying flagrant violators who might otherwise avoid the radar operator.

SUMMARY

If the airplane is used for law enforcement purposes of any form, the base cost will be $95.86/h. This would apply to manhunts, surveillance, photographic sessions, and other related activity. Use for traffic enforcement increases the hourly cost, depending on the type of use. For patrol of a highway, including the activity generated, the costs rise to $59.36/h. The use of the airplane for speed enforcement is more expensive--$247.54/h (for a two-hour session). Its use is practical only because it is more covert than radar. The airplane is superior for manhunts and related activity. It appears also to be cost effective for line patrol. Without substantial improvement in productivity of the ground support, its use for speed enforcement may not be cost effective.

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Truck Safety, Regulation, Inspection, and Enforcement in Virginia

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An investigation of state and federal regulations, inspection programs, and enforcement activities regarding truck safety was carried out to ascertain whether there were problems with the state's regulations and activity in these areas and to suggest remedial measures for any problems identified. The research, carried out with guidance from a project advisory group, included a review of relevant literature; a questionnaire survey of state enforcement programs; observations of on-road safety inspections; a review and comparison of state and federal laws and regulations that govern the trucking industry, including those that deal with hazardous materials; and an analysis of available data concerning truck accidents, registrations, miles of travel, vehicle type, load carried, and percentage of overloaded trucks. It was concluded that some revisions to the regulatory provisions that govern the trucking industry and the transportation of hazardous materials in Virginia were warranted and appropriate. Recommendations for the revision of some of the state's regulations and enforcement program activities were offered.

A great deal of attention has been focused on the safety aspects of the movement of goods by heavy trucks. Both state and federal governments have shown concern about statistics that indicate a significant increase in the involvement of heavy trucks in traffic crashes and fatalities. The response at the federal level included the introduction of the Truck Safety Act of 1978 and the Trucking Competition and Safety Act of 1979. These represent an effort to reduce crashes, injuries, and property damage; to provide drivers of commercial vehicles with safe and healthy working conditions; and to increase compliance with current regulations. Legislation has also been introduced to set national truck weight and length limits.

A 1977 General Accounting Office report to Congress stated that 20 percent of all traffic deaths resulted from truck and bus crashes and recommended an increase in funds for safety activities (1). A 1979 study by the same agency determined that "excessive truck weight is a major cause of highway damage," but the study did not deal directly with the relation between truck weight and crashes (2).

A study by the National Highway Traffic Safety Administration (NHTSA) found that, between 1975 and 1978, fatal crashes that involved heavy trucks increased by 40 percent; 10 percent of all fatalities on the nation's highways were related to accidents that involved heavy trucks; and fatal injuries to the occupants of passenger cars that collided with heavy trucks increased by nearly 30 percent (3).

The popular press, newspapers, and magazines have given considerable attention to crashes that involve heavy trucks, especially when multiple fatalities have occurred or when hazardous materials have been involved. A number of exposé articles have detailed a calculated disregard for weight and safety regulations by certain truckers.

In light of the above, officials of the Commonwealth of Virginia requested a study of the state's safety, regulation, inspection, and enforcement programs that deal with heavy trucks.

METHODOLOGY

The initial task in this study was a review of the literature on safety issues concerning heavy truck transportation. The sources of literature included federal and state agencies, the trucking and insurance industries, private and university research groups, and congressional and legislative hearings. In an attempt to determine whether truck crashes constitute a significant hazard to the users of Virginia highways, national and state data on truck accidents were analyzed. A survey was made of state programs for the enforcement of regulations on truck weights, safety, and the transport of hazardous materials, and Virginia and federal regulations on the movement of goods by the trucking industry were examined and compared. Because of the special dangers that attend the transport of hazardous materials, a significant portion of the study dealt with regulations on the transportation of these materials.

Because of the complex nature of most of the issues concerning heavy truck safety, the study was carried out with guidance from a project advisory group whose members represented the trucking industry and a variety of state and federal agencies charged with regulating the industry.

LITERATURE REVIEW

In 1976, more than 4000 people were killed in motor vehicle crashes that involved heavy trucks in the United States. This number represented a 15.7 percent increase over the number killed in 1975 and accounted for 8.9 percent of all traffic fatalities (4). Crashes that involved trucks resulted in twice the number of fatalities per crash as crashes that involved only passenger cars. Although the proportion of heavy trucks in the vehicle population is small, their exposure is disproportionately great, and their increasing involvement in fatal traffic crashes is a major safety concern (5).

General Crash Experience

The most-frequent accident that involves a tractor trailer is a collision between such a unit and a passenger car; this is followed by single-vehicle crashes and collisions with other commercial vehicles (6). Because of the relatively large size and weight of heavy trucks, collisions between them and passenger cars are especially dangerous for automobile occupants (4). Only 20 percent of the people killed in all heavy truck crashes are truck occupants and 80 percent are occupants of passenger cars, pedestrians, and bystanders (4). Such statistics suggest the need for a special sensitivity to issues of truck safety.

Causes of Accidents

Human error has been identified as one of the primary causes of accidents involving heavy trucks. A study by the University of Southern California, in conjunction with the California Highway Patrol, of accidents that involved commercial vehicles found that 45.7 percent of the truck drivers involved were at fault. Among the chief causes were driving at an unsafe speed and making unsafe lane changes (7). An important contributory factor to human error is driver fatigue. Not only driving time, but irregular scheduling, the use of sleeper operations, and variations in regular daily sleep patterns have been connected with driver fatigue (8).

Mechanical failures and vehicle defects also play a significant role in crashes of heavy trucks; they were identified as the determining factor in 10.8 percent of the accidents in the California study. Brake failures and inadequate braking ability accounted for the greatest proportion of mechanical problems. Tire and wheel failures were also important vehicle defects (7,9,10).

ACCIDENT ANALYSIS

An analysis of nationwide Bureau of Motor Carrier Safety (BMCS) data and Virginia data on truck crashes revealed several characteristics of these accidents that could be used to generate countermeasures. Major findings were as follows:

1. New drivers are more likely to be involved in truck crashes than experienced drivers, and the crash rates for new drivers increase more rapidly than those for experienced drivers;
2. Most truck collisions in Virginia involved passenger vehicles; crashes that involved two trucks ranked second; and
3. The largest percentage of noncollision, single-vehicle accidents involved either running off the road or overturning on the roadway.

Although the data indicated considerably more passenger-vehicle-related crashes than truck crashes, due to the greater numbers of passenger cars, truck crashes tended to be more severe. Trucks were less likely than cars to be involved in nonfatal accidents; however, they are more likely to be involved in fatal crashes. Overall, the average truck has more accidents per year than the average car but has fewer accidents per mile of travel.

Crash-trend data showed that Virginia's truck crash rates increased significantly between 1975 and 1977. These increases far exceeded the rates of passenger cars during the same time span. It was also determined that Virginia's truck crash rates were increasing much more rapidly than were the national truck crash rates and the passenger car crash rates in the state. Finally, fatalities are increasing at a rate greater than that for injuries or total crashes in the nation. Thus, although the limitations of the data prevent a strong conclusion that Virginia currently has a serious truck accident problem, the problem clearly exists and is worsening at an increasing rate.

SURVEY OF ENFORCEMENT PROGRAMS

In Virginia, three state agencies have responsibility for enforcing state laws related to truck weight and safety and the transportation of hazardous materials. The State Corporation Commission (SCC), State Police, and the Department of Highways and Transportation all have roles. In addition, BMCS of the Federal Highway Administration (FHWA) has the responsibility for enforcing federal regulations on truck safety and the transportation of hazardous materials.

The Department of State Police and the Department of Highways and Transportation share the responsibility for enforcing Virginia laws on truck weights. The latter operates the equipment used to weigh trucks but does not have authority to issue citations or summonses for violations of weight limits. Consequently, a state police officer works with the weigh station personnel to write tickets and issue citations.

The state police may conduct weighing activities independent of the Virginia Department of Highways and Transportation. An officer may stop a truck he or she suspects is overweight and direct the driver to travel as far as 10 miles to a permanent weigh station. The police are authorized to weigh such trucks at any permanent station, even if it is not officially open at the time. If the distance to the nearest permanent weigh station is greater than 10 miles, the officer may weigh the truck on portable scales.

The SCC and state police have responsibilty for safety inspections of motor vehicles in Virginia. The SCC investigators do not work directly with the state police, although they do have contact with the police when working at the weigh stations. They have authority to enforce the laws, rules, and regulations that govern the operations of motor vehicles and authority to issue a summons or arrest any person found in violation. They may stop and examine the lading and documents of any motor vehicle, trailer, or semi-trailer that operates on any highway in the state.

The SCC investigators conduct safety inspections at the permanent weigh stations after first checking vehicles for SCC authority. Investigators will usually stand at the scales and give the trucks cursory visual inspections. When an investigator sees a truck he or she thinks is likely to have safety problems, he or she can order the driver to pull the vehicle into the inspection area for a thorough inspection.

The SCC and state police are also responsible for the enforcement of the hazardous materials regulations. They have authority to stop and examine the lading of any motor vehicle thought to be transporting dangerous articles to determine whether it is in compliance with the rules and regulations that govern the transportation of dangerous articles. These investigations are also carried out as part of the safety checks conducted at the permanent weigh stations.

To evaluate Virginia's truck safety enforcement programs as compared with those of other areas, a questionnaire was sent to highway officials of the other 49 states and the District of Columbia. The questionnaire contained questions about programs for enforcing regulations on truck weight and safety and the transport of hazardous materials. Responses, which were received from 44 states and the District, gave a fairly complete picture of enforcement activities.

Weighing Operations

All of the respondents had some sort of truck weighing program, and most used both permanent and portable scales. Frequently, portable scales were used in conjunction with permanent scales in an effort to detect trucks that attempted to bypass the permanent scales.

Most states used the same basic equipment; however, the number of scales used and hours of operation varied greatly. One-third of the states operated at least one permanent scale seven days a week, 24 h/day, and more than two-thirds of the states had permanent scales open at least five days a week. Many states also used irregular scheduling, particularly for mobile weighing teams.

Both state police and highway or transportation departments played a significant role in the operation of weighing programs. More than half of the states named the state police and roughly 40 percent named the highway or transportation department as the agency responsible for the program. State regulatory commissions and motor vehicle agencies also had responsibility for weighing programs.

Effectiveness of Weighing Programs

An attempt was made to determine the relative effectiveness of the truck weighing programs. The number of trucks weighed varied tremendously. At one end of the spectrum, Virginia weighed more than 7 million trucks; at the other end, the District of Columbia weighed only 2240. However, because the volume of truck traffic varies considerably from state to state, the number of trucks weighed does not indicate a program's effectiveness.

Violation rates were also examined and tended to increase as the number of trucks weighed decreased. This would be expected because a program that has a reputation of not identifying violators of weight limits cannot be expected to deter truckers from running over the weight limit. When the probability of detection is low, the number of trucks that have loads in excess of the allowable limits tends to increase.

The measures that produced the most-consistent results involved comparison of the number of vehicles weighed to the number of commercial and private trucks registered in the state, the amount of diesel fuel consumed, and the number of truck miles of travel as estimated by FHWA. If the effectiveness of a truck weighing program increases as the percentage of trucks weighed increases, which should be true as more trucks that carry weights over the limit should be detected, these ratios should indicate the relative effectiveness of truck weight-enforcement programs. Therefore, program effectiveness increases as these ratios increase.

Table 1 shows the results of these calculations, with the states ranked from best to worst. Note that most of the states that have permanent scales that operate seven days a week, 24 h a day were among the states with the most-effective weigh programs. Also, the five states that had no permanent weigh stations were among the states cited by the U.S. Department of Transportation for inadequate weight enforcement and were the lowest-ranked states according to these calculations.

All of these rankings have certain problems. One is that the numbers used in the computations are proxy values and, therefore, are not completely accurate representations of the amount of truck traffic in a state. In addition, not all of the states that have permanent scales weigh every truck that passes the scales. Consequently, the number of trucks weighed for those states is lower than if all trucks were weighed, even though those allowed to pass the weigh stations are probably under the weight limits.

On-Road Safety-Inspection Programs

Thirty-six states conducted an on-road safety-inspection program. Unfortunately, many states lacked data on the number of trucks inspected so no attempt was made to determine the effectiveness of these programs.

Almost one-half of the states that performed safety inspections indicated that they did so in conjunction with weighing operations. This provided the opportunity to make a cursory visual inspection for obvious safety problems prior to performing a complete safety inspection. In the inspections themselves, most states focused on easily accessible equipment, such as brakes, tires, and lights.

More than 80 percent of the states cited the state police as having some responsibility for safety inspections and 50 percent said it was the sole responsible agency. Regulatory commissions were involved in roughly 30 percent of the states, and highway departments had some responsibility in about 20 percent.

The reported violation rate for safety inspections was far greater than the violation rate for truck weighings. For the 19 states that had data on safety inspections, the average violation rate was

Table 1. Ranking of state weight programs by effectiveness.

State	Scale Classification[a]	Weighing/ Registrations	Weighing/ Fuel Consumption	Weighing/ Vehicle Miles
Alabama	N,C	44	42	42
Arizona	P,C	25	28	26
Arkansas	P	3	4	2
California		20	20	19
Colorado	P	15	9	10
Connecticut	C	29	32	33
District of Columbia	M	33	41	39
Florida	P	11	11	16
Georgia	M	21	23	23
Hawaii	N,C	32	29	31
Idaho		31	31	28
Illinois		9	13	13
Indiana		23	26	22
Kansas	P	27	25	25
Kentucky		26	24	27
Louisiana	P	6	7	6
Maine	N,C	37	33	36
Maryland	M	28	27	30
Massachusetts	C	43	44	44
Michigan		18	18	20
Missouri	P	12	14	12
Montana	P	19	17	14
Nebraska		14	12	11
Nevada	N,C	41	40	37
New Hampshire	M	39	36	40
New Jersey	C	36	39	38
New Mexico	P	2	2	1
New York	N,C	40	38	41
North Carolina		7	10	9
North Dakota	P	10	6	5
Ohio		13	15	17
Oklahoma	C	38	34	35
Oregon	P	16	16	15
Pennsylvania	C	42	43	43
South Carolina	M	22	22	24
South Dakota	C	30	30	29
Tennessee	P	4	8	7
Utah	P	5	3	3
Vermont	M	34	35	34
Virginia	P	1	1	4
Washington	P	8	5	8
West Virginia	M	24	21	21
Wisconsin		17	19	18
Wyoming	P,M	35	37	32

[a]P = At least one permanent scale is operated 24 h/day, 7 days/week; N = no permanent scales; C = cited by U.S. Department of Transportation in February 1978 for inadequate weight enforcement; and M = classified as marginal in weight enforcement by U.S. Department of Transportation in February 1978.

20.5 percent. Rates ranged from a low of 0.03 percent to a high of 92.5 percent.

Hazardous Materials

Only 24 states actively enforced their regulations on the transportation of hazardous materials, and one state reported that it had no such regulations. Thus, fewer states had hazardous materials programs than had either weighing or on-road safety-inspection programs.

Most states conducted random inspections on the road as opposed to systematic roadway inspections or terminal inspections. Also, fewer states inspected private carriers than they did for-hire carriers because some state agencies had no authority to inspect private carriers.

In two-thirds of the states, the state police had some enforcement responsibility, and state regulatory commissions and highway or transportation departments each had some enforcement responsibility in one-third of the states. The enforcement responsibility tended to be more fragmented than that for the other programs because it was often shared among agencies that deal with health, environmental protection, and emergency services.

In summary, the data obtained on the questionnaire disclosed that all of the states that responded (45) had some sort of weighing program, 36 states had an on-the-road safety-inspection program, and 24 states had a hazardous-materials inspection program.

REGULATIONS OF THE TRUCKING INDUSTRY

In Virginia, the SCC is vested with the authority to supervise, regulate, and control all public service companies that do business in Virginia. This control includes the authority to regulate the transportation of passengers or property for compensation by motor carriers, unless the carrier is specifically exempt. Motor carriers are required by Virginia Code Sections 56-278 and 56-288 to secure approval from the SCC to operate in the state. The SCC also has authority over rates, routes, and schedules. In addition, it has appointed investigators to enforce its regulations under Title 56 of the Code and the general highway laws that apply to motor carriers under Title 46.1 of the Code.

Another general power of the SCC involves the investigation and reporting of accidents. Under Virginia Code Section 56-332, it has the authority to require motor carriers that do business in the state to report information concerning all crashes that result in injury to persons or in property damage of any kind. However, it does not require this reporting so as to avoid a duplication of recordkeeping by the Division of Motor Vehicles (DMV).

Weight and size limitations and equipment requirements are specified in the Virginia Code and apply to all vehicles that travel on Virginia highways, regardless of where they are licensed. Hauling or moving permits must be secured from the Department of Highways and Transportation for the operation of any vehicle or vehicle combination in excess of the statutory size and weight limits.

The transportation of hazardous materials is regulated pursuant to the Rules and Regulations Governing the Operation of Motor Vehicles Transporting Explosives and Other Dangerous Articles, promulgated by the SCC in 1958, which are being revised by the Department of Health under new legislative authority. Exemptions are permitted for materials transported in accordance with or exempt from federal regulations. The central purpose of the regulations is to prescribe the conditions under which dangerous articles must be loaded, transported, and unloaded. These conditions are designed to ensure that hazardous materials are handled and transported in a manner that is safe for the public and the motor carrier. Motor carriers of hazardous materials must also abide by all other laws and rules that govern transportation in Virginia.

Overview of Federal Regulations

In 1967 BMCS was established as a part of FHWA. Its primary function is to reduce commercial vehicle accidents, fatalities, injuries, and property losses. To encourage the safe operation of commercial vehicles, the Bureau also initiates research and development projects within FHWA. The jurisdiction of BMCS stems primarily from four pieces of legislation.

Congress originally passed the Interstate Commerce Act, which established the Interstate Commerce Commission (ICC), in 1887 and subsequently amended it several times, most recently in 1978. In 1935 Congess passed an amendment known as the Motor Carrier Act. Its purpose was to establish a uniform national system of motor carrier regulation. This Act authorized ICC to regulate the qualifications and hours of service of employees and to ensure the safety of operations and equipment of common, contract, and private carriers of property engaged in interstate commerce. The Act also gives ICC the authority to promulgate regulations, hold hearings, and conduct research. In addition, it defines that agency's very broad inspection and investigatory authority.

In 1966 the Department of Transportation Act established the U.S. Department of Transportation (DOT) and transferred the functions cited above from ICC to DOT. Subsequently, authority was delegated to BMCS to carry out the functions authorized by the Act.

The Noise Control Act of 1972, 42 U.S. Code Section 4917, empowers the Secretary of Transportation, in cooperation with the Administrator of the U.S. Environmental Protection Agency (EPA) to promulgate regulations that govern noise emissions from commercial vehicles operated by interstate carriers. In addition, the Act established inspection and enforcement powers within DOT.

The Hazardous Materials Transportation Act of 1976, 49 U.S. Code Section 1809, consolidated the general responsibility to supervise the issuance and enforcement of regulations on the transportation of hazardous materials within the Materials Transportation Bureau of DOT. BMCS, however, retained primary responsibility for originating regulations and carrying out the inspection, enforcement, and training functions related to motor carriers.

Comparison of Virginia and Federal Regulations

The Federal Motor Carrier Safety Regulations (FMCSR), 49 Code of Federal Regulations Sections 386-398, sets the boundaries for the inspection and enforcement activities of BMCS and provides a comprehensive set of definitions, standards, and procedures for all aspects of motor carrier safety. Drivers and trucks subject to the FMCSR include those that haul (a) cargo from overseas, (b) property from state to state, (c) cargo across a border, and (d) loads of interstate cargo within one state.

Although a number of states have adopted the FMCSR in whole or in part, Virginia has not. Neither has the state developed a section of the Virginia Code that deals specifically with motor carrier safety in a topical fashion. Although numerous aspects of the Virginia Code parallel the FMCSR, the lack of a topical approach makes it more difficult to assess the state's standards for motor carrier safety. It may also make it more difficult for state officials to educate carriers and enforcement personnel concerning safety standards. Consequently, some consolidation or reorganization of sections that affect motor carrier safety should be helpful.

The following sections discuss differences between state and federal regulations that affect the road operations of the trucking industry. Some of the differences prompted recommendations that the Virginia regulations be altered to conform with the FMCSR. Other differences are also noted where the significance cannot be ascertained without better data on truck accidents. At present we cannot determine which set of regulations better promote safety in transportation.

Qualifications of Drivers

The FMCSR requires that drivers be 21 years old, able to read and speak enough English to understand highway signs and communicate with officials, and able to operate a vehicle safely. In addition to the application process and the review of the driver's operating record, the driver qualification procedures include a road test, a written examination, and a physical examination.

The Virginia provisions differ in several respects. Virginia's minimum age for a chauffeur's license is only 18. The Code, although it requires a road test for drivers of vehicles of more than 40 000 lb, does not set forth requirements for this test. Also, the test may be waived if the applicant certifies that he or she has driven at least 500 miles in the type of vehicle he or she intends to drive. Apparently, the waiver is meant for drivers who have been licensed by other states, participated in motor carrier training programs, or driven with a learner's permit under the supervision of a licensed driver. However, the Code contains no provisions on how drivers can accumulate the 500 miles.

With regard to Virginia's lower minimum age requirements, there are no state crash statistics to indicate whether Virginia truck drivers under 21 years of age have a higher accident rate than older drivers. However, there are U.S. data to indicate that driver inexperience may be a causative factor in accidents [see Table 2 (6)]. In 1975 the BMCS considered lowering the FMCSR minimum age to 18, but decided against such action because available data indicated that persons under 21 lack the maturity, judgment, and skill to drive heavy trucks. In addition, researchers at the University of North Carolina have found higher accident involvements for young truck drivers. Finally, in 1978, the National Transportation Safety Board recommended that Vir-

Table 2. Involvements by driver experience.

Years of Driving Experience	1977	1976	1975	Change 1975-1977 (%)
0-1	14 182	10 603	9 357	+51.6
2-4	6 198	6 488	6 397	--3.1
5-9	4 830	4 024	3 969	+21.7
10-14	2 190	1 952	1 819	+20.4
15-19	1 131	1 141	1 219	--7.2
20+	2 032	1 795	1 958	+3.8
Total	30 563	26 003	24 719	+23.6

ginia eliminate the 500-mile waiver and expressed concern about the 18-year-old minimum age.

Driving of Motor Vehicles

The federal rules and Virginia laws that govern the operation of trucks are identical, or nearly identical, on many points. However, some differences exist and a significant one relates to use of seat belts. FMCSR requires use of seat belts but the Virginia Code requires only that seat belts be installed and explicitly states that the failure to use seat belts is not negligence.

The inspection provisions of FMCSR emphasize the pretrip inspection of safety devices by the driver. Although the Virginia Code requires a vehicle inspection every six months and prohibits operation with defective equipment, it does not require pretrip inspections. Although the Code implies a policy of pretrip inspection, state officials have no grounds on which to enforce a day-to-day accident-prevention program. Federal regulations, in contrast, require recordkeeping on pretrip procedures.

Parts and Accessories

Section 393 of FMCSR describes the scope of safety checks conducted by BMCS inspectors and establishes standards that equipment must meet. Again, the Virginia Code, under Title 46.1, parallels many of the regulations of FMCSR. One difference between federal and state rules concerns the stopping distance standards for brakes. FMCSR contains somewhat more stringent requirements. The federal regulations concerning tires are also more stringent. Federal tread-depth requirements are stricter and FMCSR also contains extensive provisions governing tire loads and pressures. Virginia has no standards for tire loads and pressures.

A final positive note is that both the SCC and state police truck-safety-enforcement teams are familiar with FMCSR and use them and BMCS inspection techniques as the basis for their truck-safety-enforcement programs. In view of the small size of BMCS enforcement staff, however, it is frustrating that state inspectors cannot cite obvious violations of federal law.

Reporting Accidents

Both federal and state requirements exist for reporting accidents; however, there are significant differences in the report forms used. The state police use a general field note form for all accidents, regardless of the type of vehicles involved. This form provides a great deal of information, but there are inadequacies in the data relevant to truck safety. As a result, the truck accident data for the state are insufficient for making generalizations in a number of significant areas of information.

Hours of Service

Under Section 395 of FMCSR, BMCS limits most truck drivers to a maximum of 10 h of driving time after accumulation of a minimum of 8 h off-duty. Additional regulations of driving time apply, based on the number of consecutive hours on duty and the number of consecutive days of operation. BMCS ensures compliance with these regulations by checking a daily log of hours that drivers must keep. If a driver is detected in violation, he or she may be placed out-of-service.

The only Virginia law governing hours of service applies to all drivers and states that it is unlawful to drive more than 13 h in a 24-h period. Apparently, this rule is invoked only for determining driver negligence after a crash, and the state has no requirements for keeping a log or other methods for enforcing the law.

In-Field Safety Checks

Both federal and state officials conduct on-the-road safety checks. BMCS inspectors have the authority to place vehicles out-of-service for violations of FMCSR until repairs are performed. In Virginia, however, SCC inspectors responsible for enforcement of state laws do not have the authority to declare unsafe vehicles out-of-service, although they can cite drivers and carriers for violations that can result in fines.

HAZARDOUS MATERIALS

Definitions of the term hazardous material generally tend toward extremes of either vagueness or specificity. Ideally, a compact definition of hazardous materials could be fashioned that would indicate whether a substance in question is hazardous or not. In practice, however, general criteria to fit all dangerous substances is difficult to develop. The annual introduction of chemicals alone accounts for nearly 500 new substances of varying characteristics and potential for harm. The definition must anticipate these substances and also apply to those already known. Because of the concern for identification and regulation of all applicable hazardous materials, the definition becomes either exceedingly specific, and resembles a listing of materials and their traits, or increasingly generalized in order to account for all possibilities.

The broad federal definition in the Hazardous Materials Transportation Act focuses not on the means by which harm occurs but rather on the fact that it does occur. Hazardous materials are defined as a substance or material in a quantity and form that may pose an unreasonable risk to health and safety or property when transported in commerce. This definition does not attempt to provide a functional guideline for determining whether a substance is harmful. Whether the risk is unreasonable is determined by the Secretary of Transportation through the hazardous materials regulations authorized by the Act. The regulations contain a list of 1200 substances judged to be capable of posing an unreasonable risk. This list includes those hazardous materials that are (or were) frequently transported and is used to determine whether a substance in question is a regulated material and to give the shipper guidance in labeling containers.

Federal and Virginia Regulations

Federal regulations govern materials transported in interstate commerce or in a manner that affects interstate commerce. Unless the Secretary of Transportation determines that a state's requirements afford at least as much protection as the federal regulations, and do not unreasonably burden interstate commerce, the federal regulations preempt inconsistent state requirements. To eliminate conflicts, Virginia regulations exempt substances transported in interstate commerce according to federal regulations or exempt from federal regulations.

Both Virginia and federal regulations prohibit the shipping or transporting of hazardous materials not in conformity with applicable regulations. However, the federal regulations also apply to persons that offer or accept nonconforming hazardous materials for transportation and to persons that represent or sell a package as in compliance with the regulations when it does not. Both sets of regulations contain certain exemptions, such as for U.S. military forces. Virginia specifically exempts flammable liquids from any regulation, although SCC does regulate the shipment of petroleum products.

Civil sanctions of up to $1000/day (state) and $10 000/day (federal) may be imposed for a violation of these regulations. Criminal sanctions are also available; violations are a misdemeanor in Virginia and a felony under the federal regulations.

Cargo Regulations

Virginia and federal regulations prohibit the transportation of hazardous materials in certain situations. For example, Virginia prohibits the transportation of explosives in passenger vehicles and federal regulations prohibit transport of any hazardous materials, with certain exceptions, on for-hire vehicles that carry passengers. Both sets of regulations prohibit the transport of certain combinations of hazardous materials. The federal regulations are far more specific and list 22 categories of prohibited combinations of hazardous materials.

Cargoes must also be loaded and unloaded in conformance with state and federal regulations. In addition, both sets of rules require the identification of cargoes. Federal regulations generally require that all individual containers be marked. In Virginia, if the entire cargo is of the same type of hazardous materials, then only the vehicle must be marked to indicate the contents. Compared with those of Virginia, the federal identification requirements are more detailed, broader in scope, and include additional placarding provisions.

Vehicle Regulations

Both Virginia and federal regulations govern the condition and construction of vehicles that transport hazardous materials. Virginia requires that trucks must be strong enough to carry the load and be in first class condition. Federal regulations place responsibility for the vehicle's condition on both the carrier and the driver and prohibit the operation of a truck in a hazardous condition.

Both federal and Virginia regulations require that the vehicle be inspected prior to each trip. As in other areas, the federal regulations are more extensive and detailed in specifying the items to be checked and the manner of inspection and recordkeeping. Both federal and state regulations are concerned with the electrical system, vehicle lighting, condition of the cargo area, use of certain materials in the construction of trucks carrying explosives, and the carrying of fire extinguishers. Again, the federal regulations that govern these areas are more detailed than those of the state.

The federal and state requirements on placarding diverge substantially. Although both require placards that indicate the contents of the vehicle, the federal rules specify 17 placard designations, but Virginia rules specify only 7. The federal rules are more detailed in specifying the design of the sign and its required visibility.

Driving Regulations

Regulations on the operation of a vehicle carrying hazardous materials cover the place, time, and manner of operation. For example, both federal and state rules discourage the unnecessary movement of hazardous materials through places where there are likely to be high numbers of people. Federal regulations require vehicles carrying hazardous materials to avoid heavily populated areas, unless there are no practical alternatives. Another example is Virginia's requirement that vehicles that carry explosives or a poisonous gas operate during daylight hours whenever possible.

Regulations that govern a multitude of other aspects of the transport of hazardous materials include those on following distances, parking, emergency stopping and signaling, procedures at railroad crossings, the proper documentation of the nature and quantity of hazardous materials carried, the use of intoxicants or narcotics by drivers, and sleep and rest periods.

Virginia regulations for petroleum trucks differ from other Virginia truck regulations in that they require the drivers of these trucks to be at least 21 years old rather than 18. Virginia also requires that drivers of hazardous materials be experienced, careful, capable, and able to read and write in English, and that they possess a valid chauffeur's license.

In general, the federal regulations on hazardous material cargoes, vehicles, and drivers are more thorough and more concerned with safety than comparable Virginia regulations. In addition, the possibility exists that hazardous materials may be transported on Virginia's highways by carriers or on vehicles not subject to federal regulations. Because of these factors, Virginia regulations on the transportation of hazardous materials, promulgated in the 1950s, are undergoing revision.

FINDINGS AND CONCLUSIONS

National accident data reveal an increasing incidence of crashes and fatalities that involve heavy trucks. These statistics are of concern to traffic safety officials because they represent an increasing hazard to truck drivers as well as to the safety of other highway users. Also, indications are that there is a problem in terms of the number of crashes per vehicle and that the problem is worsening at a rapid rate.

The relation between the length of experience of truck drivers and crash involvement is significant. Data indicate that truck drivers that have less than one year of experience with their employers had more crashes than drivers that had more experience. In addition, 18- to 21-year-old truck drivers had a substantially higher rate of crash involvement than did 25- to 40-year-old drivers.

Differences between the FMCSR and state trucking regulations on operator age, operator licensing, accident reporting, hours of off-duty and driving time, use of seat belts, and pretrip inspections are substantial. There also are differences between FMCSR and the Virginia regulations on braking distance standards, requirements for front tire tread, and tire load capacity and pressure for the most common sizes of tires.

In general, the federal regulations on the transportation of hazardous materials are more thorough and safety-oriented than comparable Virginia regulations. Also, the state imposes much lighter penalties for violations of regulations on the transport of hazardous materials than does the federal government.

Most states have truck weighing and inspection programs; however, the effectiveness of these programs varies widely. Based on an analysis of the data presented in this report, Virginia has one of the best programs in the nation.

Safety in truck transportation is of concern to individuals at numerous levels of government and private industry. In Virginia, available data indicate a need for close scrutiny of the involvement of trucks in traffic crashes. In addition, it has been determined that certain revisions to the regulations that govern the trucking industry and the transportation of hazardous materials by truck are warranted.

IMPLEMENTATION OF RESEARCH FINDINGS

In the time that has elapsed since the research was initiated and the report published, a number of events have occurred that can be traced directly or indirectly to the research reported here. The most-significant event has been the transfer of responsibility for the regulation of hazardous-material cargoes from SCC to the Department of Health. State regulations on the transport of these materials, which were promulgated during the 1950s, are being revised by officials of that department. Although the final version of the regulations has not been completed, significant changes have been made in the state's operational procedures.

A second major event has been the special attention given to truck safety by the state police. Through a safety grant, approximately 30 troopers have been to the Transportation Safety Institute for courses on truck safety and inspection. These troopers form a core of officers who are carrying out the state's truck inspection programs. In addition, the state police have proposed the establishment of an inspection division that will have two primary responsibilities:

1. The supervision of the current periodic motor vehicle inspection stations and
2. The performance of in-field inspections of trucks for compliance with vehicle safety and hazardous materials regulations.

And finally, the DMV has initiated procedures for the modification of the state accident report form, including the solicitation of input from various state agencies. The format and data items have not been made final, so it is possible that the new form will not require the recording of some essential truck data. If this is the case, then a supplementary form should be developed to aid in gathering these data.

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Selection Process for Local Highway Safety Projects

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This report presents a procedure (a) to identify accident problem locations, (b) to develop accident countermeasures, and (c) to rank highway safety projects according to their relative cost-effectiveness. The procedure is designed to be applicable to all highway operating agencies in order to assist them with the resource-allocation decision. The procedure was developed by the Oakland County Road Commission with assistance from staff of the Southeast Michigan Council of Governments and the Oakland County Transportation Systems Management Committeee. The procedure integrates techniques used by the Oakland County Road Commission to identify problem locations and develop project concepts as a means to evaluate those concepts for safety and other impacts. Although highway safety is the primary concern of the Oakland County Road Commission in developing this procedure, other variables (e.g., traffic congestion, air quality, and fuel conservation) can also be included in the process.

The Oakland County Road Commission (OCRC), in the face of growing liability exposure and an ever-increasing frequency of traffic accidents, has recently adopted highway safety as its number one priority. Traffic congestion and flow, although not ignored in the decisionmaking process, are to take a back seat to safety. As a result of this change in orientation, it became necessary to develop a new procedure for the allocation of resources that incorporates safety as the primary goal.

A substantial amount of research has developed means to identify hazardous locations and to evaluate projects in terms of cost-effectiveness, net benefits, and so on (1-5). Many of the approaches suggested are too complex to be implemented by local highway agencies, which have limited resources. Often researchers have described only part of the process that leads to the resource-allocation decision. For example, a number of reports concerning the identification of hazardous locations have been published over the years, but this activity is only one step in the decisionmaking process.

The purpose of this study is to present a comprehensive approach to the development and implementation of a highway safety project on the local level. The process described is designed to be applicable to all local highway agencies in order to assist them with resource-allocation decisionmaking.

The process was developed by OCRC with the assistance of staff from the Southeast Michigan Council of Governments (SEMCOG) and the Oakland County transportation systems management (TSM) committee. Some stages in the process have been used in the past by OCRC to assist in making decisions about safety improvements, but during the TSM planning process the various stages of the process were integrated and other factors were included.

In summary, the four stages of this process are as follows:

1. Identification of problem locations,
2. Development of project alternatives,
3. Evaluation of project alternatives, and
4. Project programming.

Although the process is not unique, the stages in the process present approaches that can be readily implemented by local highway authorities, regardless of size or sophistication. The process places emphasis on highway safety, but includes other factors related to traffic congestion, energy consumption, and economic and environmental concerns.

IDENTIFICATION AND EVALUATION OF SAFETY PROJECTS

Since OCRC established highway safety as its number one priority, numerous techniques have been used to identify problem locations and formulate project concepts. Many of the approaches used were too complex to integrate into daily operations. Others were very time-consuming or expensive in terms of the additional resources needed.

The approach presented in this study reduces the need for extensive data and additional resources. It is simple enough to be used daily as an operational tool.

Identification of Problem Locations

OCRC and most local highway authorities have at their disposal computer or manual files of traffic accidents within their jurisdictions. In Michigan, the Office of Highway Safety Planning maintains the Michigan Accident Locator Index (MALI), which can provide local highway agencies with site-specific accident statistics. Most other states have similar systems.

The statistics available through these systems or maintained manually provide the basis for the identification of problem locations. At OCRC three statistics are used during this stage of the decisionmaking process:

1. Average accident frequency per year at a site,
2. Average accident rate per million vehicle miles of travel (VMT) (for links) or million vehicles (for intersections) at a site, and
3. Percentage of injury and fatal accidents to total accidents at a site.

Three years worth of data are used to compute yearly averages so that the effects of one abnormal year on any of these factors is minimized.

Average accident frequency per year is the primary measure of a site-specific accident problem at locations that have similar traffic volumes. When two locations have similar traffic volumes, the one that has the greater accident frequency usually has a greater accident problem. Most locations that have high accident frequency can normally be associated with high traffic volumes, low average vehicle speeds, and a high percentage of property-damage-only (PDO) accidents. Due to the low severity rate of accidents at these locations, the level of societal costs and liability of the highway agency may not be reflected by high accident frequencies. Other measures should also be considered.

The accident rate per million VMT or million vehicles is used to control for the effects of traffic volumes on accident frequency. When two locations have dissimilar traffic volumes, the one that has the highest accident rate relative to the amount of traffic may have a greater accident problem. In other words, the frequency of accidents at this location could be abnormally high relative to the amount of traffic it carries.

Whereas, the accident frequency measure favors high-volume locations, the accident rate measure favors those that have low traffic volumes. For example, the accident and traffic characteristics of three intersections are given in the following table:

Figure 1. Oakland County traffic accident summary.

ACCIDENT RATE	ACCIDENT FREQUENCY (ACCIDENTS PER YEAR) 0-3	4-7	8-11	12-15	16-19	20-23	24-27	28-31	32-35	36-150
0.00 - 0.59	149	64	6	5	0	0	0	0	0	0
0-60 - 1.19	43	68	59	28	9	5	6	1	1	3
1.20 - 1.79	10	32	36	30	20	15	6	6	5	6
1.80 - 2.39	9	12	22	18	20	16	11	4	7	13
2.40 - 299	1	10	15	8	9	8	11	2	8	20
3.00 - 3.59	0	7	1	6	6	2	7	3	3	17
3.60 - 4.19	1	3	4	6	2	3	0	2	4	10
4.20 - 4.79	0	2	1	1	2	2	0	2	1	5
4.80 - 5.39	0	1	1	2	4	0	1	1	0	1
5.40 - 24.00	5	9	4	3	2	2	1	2	3	7

Priority 2 Priority 1

Intersection	Accidents per Year	AADT	Accident Rate (accidents/ million VMT)
A	100	100 000	2.74
B	80	20 000	10.96
C	2	200	27.40

If accident frequency is used as the only measure of an accident problem, then intersection A would be perceived as having the greatest accident problem. If the accident rate per million vehicles is used in an isolated manner, intersection C would be considered the worst.

To simplify the process of identifying accident problem locations, a 10x10 accident analysis matrix (6) can be devised, based on statistically determined intervals in accident rate and frequency. Separate matrices are used for intersections and road segments. Average yearly accident frequency and accident rate are used to plot road segments and intersections within the appropriate matrix cells. The highest priority locations are those plotted in cell (10,10). A diagonal reading across the matrix gives other priority groups. Figure 1 provides an example of such a matrix used by OCRC. By locating within Figure 1 each of the three hypothetical intersections in the foregoing example, intersection B is given priority over the others.

Once locations are ranked into these priority groups, they are ranked within each priority group by accident severity:

$$\text{Severity factor} = (\text{Fatal accidents} + \text{injury accidents}) \div \text{total accidents} \qquad (1)$$

The accident analysis matrix technique is a good indicator of priority locations, but must be followed up by other analyses to determine possible accident countermeasures and the relative cost-effectiveness of implementing those countermeasures at various locations.

Development of Project Alternatives

Once problem locations are identified by using the foregoing technique, OCRC assigns an interdisciplinary team to review each problem location and determine alternative project concepts. The project review teams are composed of staff from traffic engineering, design engineering, and transportation planning. The major objective of this approach is to mitigate all roadway and environmental characteristics that impact negatively on highway safety. The team-review approach uses the interdisciplinary expertise of team members to devise a variety of strategies for accident reduction. If constraints on staffing present a problem, the general approach can be carried out by an individual staff member. The approach is designed to be flexible, although specific guidelines for the location review should be devised by the implementing agency.

In the OCRC approach, a field review of the problem location is carried out and site conditions are noted, diagrammed, and photographed. Some problems in design or geometrics might be obvious; others may be more nebulous. Often a survey of property owners adjacent to the site is necessitated in order to determine the operational characteristics of traffic at the site. If time is not a constraint, a windshield survey can be taken to record drivers' reactions.

Information obtained during this stage of the team-review process includes the following:

1. Existing and expected traffic volumes,
2. Turning movement counts,
3. Existing right-of-way,
4. Signing and other traffic control devices,
5. Roadside obstacles,
6. Vehicle speeds,
7. Pavement or surface condition,
8. Shoulder width and condition,
9. Existence of on-street parking,
10. Sight distance,
11. Roadway design characteristics,
12. Roadway geometrics, and
13. Visual evidence of traffic accidents (e.g., scarred trees and scraped guardrail).

Simultaneous with the field review, an analysis of the accident history of the site is carried out. Each reported accident is investigated individually, and information for all accidents is tabulated. Collision diagrams are drawn and accident patterns are noted. The final step in the team-review process is to relate these accidents to the physical or operational characteristics of the site. By doing so, alternative sets of accident countermeasures can be determined for each particular location.

Cost estimates are assigned to each alternative project concept at a specific location. The project alternatives for a location normally range from low-cost alternatives to major reconstruction. If a project at a location is necessarily deferred, a set of interim accident countermeasures is devised to reduce accidents during the period of deferral. The product of this team-review process is a report that indicates existing conditions at the location and specifies the various improvement alternatives proposed.

Table 1. Safety improvement rating sheet for links.

Impact Criteria	Points Possible	Accident Frequency		
		High[a]	Medium[b]	Low[c]
Frequency reduction (%)				
>30	7.5	50+	20.0-49.9	<20.0
10-29	5.0	50+	20.0-49.9	<20.0
<10	2.5	50+	20.0-49.9	<20.0
Rate reduction (%)				
>30	7.5	26.0+	3.44-25.99	<3.44
10-29	5.0	26.0+	3.44-25.99	<3.44
<10	2.5	26.0+	3.44-25.99	<3.44
Severity accident reduction (%)				
>30	25.0	25+	6.0-24.9	<6.0
10-29	15.0	25+	6.0-24.9	<6.0
<10	5.0	25+	6.0-24.9	<6.0

[a]Multiply by 1.0. [b]Multiply by 0.5. [c]Multiply by 0.25.

Table 2. Safety improvement rating sheet for intersections.

Impact Criteria	Points Possible	Accident Frequency		
		High[a]	Medium[b]	Low[c]
Frequency reduction (%)				
>30	7.5	25+	10.8-24.9	<10.8
10-29	5.0	25+	10.8-24.9	<10.8
<10	2.5	25+	10.8-24.9	<10.8
Rate reduction (%)				
>30	7.5	3.50+	1.66-3.49	<1.66
10-29	5.0	3.50+	1.66-3.49	<1.66
<10	2.5	3.50+	1.66-3.49	<1.66
Severity accident reduction (%)				
>30	25.0	15+	5.0-14.9	<5.0
10-29	15.0	15+	5.0-14.9	<5.0
<10	5.0	15+	5.0-14.9	<5.0

[a]Multiply by 1.0. [b]Multiply by 0.5. [c]Multiply by 0.25.

Evaluation of Safety Project Alternatives

During the team-review process, an attempt is made to relate existing environmental characteristics of a location with the accident history at that location. The project alternatives developed must then be evaluated to determine the effectiveness of the proposed projects. At this point one of the group of alternatives at a specific location is chosen for implementation. Then each of the chosen alternatives is ranked among all projects according to its relative cost-effectiveness.

During the recent development of the Oakland County TSM plan, a procedure for evaluating and ranking project alternatives in terms of cost-effectiveness was devised. Although the process weighs highway safety above all other planning criteria, traffic congestion and delay, air quality, energy conservation, intermodal coordination, and social and economic impacts can be integrated. The process assigns points to alternative safety projects based on the relation between the amount of safety improvements the project provides and the existing level of accident experience at the project location.

Three variables are used to measure a project's impact:

1. Accident frequency,
2. Accident rate, and
3. Severe accident frequency.

Accident frequency is the average annual number of accidents at a particular location. The ranges indicated in Tables 1 and 2 (i.e., high, medium, and low) were determined by using three years of accident data for Oakland County roads and intersections. The high category indicates locations that experience a critical level of accidents. The medium category indicates locations that experience accident frequencies greater than the average for all locations. The low category includes those locations that have less than average accident frequency among all locations.

Accident rate is the number of accidents at a particular location relative to the amount of traffic at the location. Accident rate must be considered when reviewing locations that have dissimilar traffic volumes. For example, a 1-mile long road segment that has 10 accidents/year and 1000 vehicles/day has an accident rate of 27.40 accidents/million VMT, whereas a 1 mile road segment that has 10 accidents/year and 10 000 vehicles/day has an accident rate of 2.74 accidents/million VMT. The segment that has 1000 vehicles/day has a greater accident problem than does its more heavily used counterpart. Again, the high and medium category ranges have been determined from a review of data from all locations in Oakland County.

Severe accident frequency is the average annual number of accidents that result in personal injury or fatality. A reduction in the frequency of severe accidents has a dramatic impact on the reduction of cost to society, therefore, the benefits of a project are increased.

Tables 1 and 2 are used to determine the points of effectiveness associated with each project. For example, a project is proposed for a road link that has more than 50 accidents per year. The proposed project is expected to reduce accidents by 15 percent. Therefore, the project received five points for accident frequency reduction. This procedure is carried out for all three impact criteria to determine the final safety effectiveness score for a project.

The safety-effectiveness score is then divided by the estimated project cost and multiplied by one million to determine the cost-effectiveness of the proposed project:

$$\text{Cost-effectiveness} = a/b \times 10^6 \quad (2)$$

where a is the safety-effectiveness score and b is the estimated project cost. Projects that have the greatest scores are given priority for implementation.

In order to ensure consistency in evaluating alternative projects, a set of uniform accident-reduction factors (7,8) is used to determine a project's impact on accident frequency, rate, and severity. The accident-reduction factors shown in Table 3 are used by OCRC. Percentage reductions in various types of accidents are related to specific types of improvements. In addition, each accident type is associated with a severity factor so that reductions in severe accidents can be determined. The average percentage of severe accidents are as follows:

Accident Type	Average Severe (%)	Accident Type	Average Severe (%)
Right angle	42	Fixed object	36
Left turn	43	Overturn	62
Rear end	26	Pedestrian	97
Head-on	42	Bicycle	86
Side-swipe	15	Car-train	52
Parking maneuver	18		

In order to determine the estimated reduction in accidents the following formula is used:

$$R = \Sigma R_i \qquad (3)$$

where R is the total estimated annual accident reduction and R_i is the estimated reduction of type i accidents.

$$R_i = A_i \times P_i \qquad (4)$$

where A is the average annual type i accidents and P_i is the estimated fractional reduction of type i accidents.

$$P_i = 1 - (1-P_{i1})(1-P_{i2})(1-P_{i3}) \ldots \qquad (5)$$

where P_{i1}, P_{i2}, P_{i3} are the estimated fractional reduction of accident type i caused by improvements 1,2,3,....

The percentage reduction in accident frequency is determined by the following equation:

$$\text{Percentage reduction} = R/E \qquad (6)$$

where E is the existing frequency of accidents at a location. The percentage reduction in accident rate equals that for accident frequency. Therefore, no additional calculation need be performed to determine a project's impact on accident rate.

To determine the estimated reduction in severe accidents the following calculation is performed:

$$S = \Sigma S_i \qquad (7)$$

where S is the total estimated annual reduction in severe accidents and S_i is the estimated reduction in severe accidents of type i.

$$S_i = R_i \times Sr_i \qquad (8)$$

where Sr_i is the average percentage of severe type i accidents.

The safety project-evaluation process described above can be implemented easily by local highway agencies regardless of their size or sophistication. Access to a computer will facilitate the process.

Perhaps the biggest advantage to using Tables 1 and 2 is that they provide a rather simplistic approach that, with little explanation, can be used by nontechnical staff of small municipalities. For this reason alone, the tables should be retained. However, note that the selection of the number of columns (low, medium, and high) and the selection of the corresponding multipliers (0.25, 0.5, 1.0) was somewhat arbitrary. Although it was designed to

Table 3. Accident reduction factors.

	Accident Type										
Improvement	Right Angle	Left Turn	Rear End	Head-On	Side-Swipe	Parking Maneuver	Fixed Object	Overturn	Pedestrian	Bicycle	Car-Train
Traffic control devices											
Install new traffic signal	0.5		+0.5[a]						0.2	0.2	0.3
Install pedestrian signal									0.4	0.2	
Add separate left-turn phase											
With new left-turn		0.7	0.2	0.1	0.2						
Without left-turn lane		0.4									
Prohibit left turns		0.9	0.3						0.1	0.1	
Prohibit right turn on red	0.3		0.2		0.2				0.3	0.2	
Upgrade signals	0.1	0.1	0.2	0.1	0.1				0.1	0.1	
Improve timing and interconnect	0.1	0.1	0.2						0.1	0.1	
Install fully actuated signal	0.1	0.8	+0.5[a]		0.2				+0.1[a]	+0.1[a]	
Install 12-in lens			0.1								
Install advance warning flashers	0.3		0.3	0.1					0.1	0.1	0.2
Remove signal	+0.3[a]	+0.1[a]	0.9						+0.1[a]	+0.1[a]	
Upgrade signing	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
Install special curve warning signs				0.2	0.2		0.2	0.2			
Minor leg stop control	0.5	0.3	+0.2[a]	0.1					0.2	0.2	0.2
Install all-way stop	0.7	0.5	+0.5[a]	0.3			0.2		0.3	0.3	
Overhead lane signs			0.1		0.2						
Overhead warning signs	0.2	0.2	0.2								
Install yield signs	0.3	0.2	+0.2[a]						0.2	0.2	
Intersection directional and warning signs	0.2	0.1	0.2	0.1			0.1				
Edge markings							0.2	0.1			
Centerline markings				0.2	0.3						
No passing stripes				0.3	0.3						
Raised permanent reflectorized markers				0.2	0.2		0.1	0.1			
Railroad crossing gates											0.6
Channelization											
Add center left-turn approach lane											
With left-turn phase		0.7	0.2	0.1	0.2						
Without left-turn phase		0.5	0.2	0.1	0.2						
Add right-turn lane and deceleration lane			0.2		0.1						
Add passing lane			0.3								
Add continuous left-turn lane		0.3	0.5	0.2	0.3						
Extend lane drop and acceleration lane			0.3	0.1	0.3		0.1				
Add median and median barrier		0.5		0.5	0.3						
Other											
Remove on-street parking	0.1		0.1		0.3	0.9	0.4		0.3	0.3	
Revise driveways	0.1		0.1			0.2					
Remove fixed object							0.8				
Widen lane width		0.1		0.2	0.5	0.3	0.3	0.2		0.3	
Widen shoulders				0.1	0.1	0.3	0.2			0.1	
Install curbing							0.5				
Resurface			0.1	0.1	0.1	0.1	0.1	0.1			0.1
Deslick	0.1		0.4	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
Improve horizontal alignment				0.2	0.2		0.2	0.2			0.1
Improve vertical alignment				0.2	0.2		0.1	0.1			0.2
Illuminate	0.1		0.1	0.1	0.1		0.1	0.1	0.1	0.1	0.1
Improve superelevation				0.2	0.2		0.2	0.2			
Install guardrail							0.4				
Increase radii at intersection	0.1		0.2		0.1	0.1	0.1	0.1	0.1	0.1	
Improve sight distance at intersection	0.3	0.1		0.1	0.1	0.1			0.1	0.1	0.3
Widen bridge				0.4	0.4		0.4				
Pave approach	0.1		0.2						0.1	0.1	0.1

[a]Increase rather than reduction.

give more credit to increasingly worse locations, the tables could just as easily have been set up with only two columns (low and high) or a very large number of columns, each with multipliers that increase in value. The same arbitrary situation exists in the point spread for giving credit to the reductions (e.g., for frequency, 2.5, 5.0, and 7.5).

An obvious improvement to Tables 1 and 2 would be to develop a function that increases the multiplier or points proportionate to the increase in the scale under consideration (e.g., increase in frequency or increase in frequency reduction). Equation 9 provides such a function.

$$CE_{ij} = \left\{ [P_F(F_j/F_{max})(FR_{ij}/FR_{max}) + P_R(R_j/R_{max})(RR_{ij}/RR_{max}) + P_S(S_j/S_{max})(SR_{ij}/SR_{max})] \div C_{ij} \right\} \times 10^6 \quad (9)$$

where

- i = alternative improvement under consideration,
- j = location to be improved (i.e., intersection, curve, or link),
- CE_{ij} = cost-effectiveness of improvement i at location j,
- C_{ij} = cost of improvement i at location j,
- P_F = points (max) for reduction in frequency,
- F_j = frequency of accidents at j,
- F_{max} = maximum frequency possible at any location,
- FR_{ij} = estimated frequency reduction for i at j,
- FR_{max} = maximum possible reduction in frequency at any location,
- P_R = points (max) for reduction in accident rate,
- R_j = accident rate at j,
- R_{max} = max possible rate at any location,
- RR_{ij} = estimated rate reduction for i at j,
- RR_{max} = max possible reduction in rate at any location,
- P_S = points (max) for reduction in severity,
- S_j = number of severe accidents at j,
- S_{max} = max possible number of severe accidents at any location,
- SR_{ij} = estimated reduction in severity for i at j, and
- SR_{max} = max possible reduction in severity at any location.

As should be readily apparent, the first set of factors represents the potential credit for accident frequency, the second set for accident rate, and the third set for accident severity. For convenience, $P_f + P_r + P_s = 100$. The multiplier of 10^6 at the end is included simply to provide a meaningful cost-effectiveness number for easy comparison.

The establishment of the maximums (e.g., F_{max}) is not as critical as might appear, provided the same maximums are used for all comparisons. One approach might be to simply use the highest value for the group of alternative projects under consideration. For example, if 100 alternative projects were being considered, the location that has the highest frequency might be used in setting F_{max}. The same process would then be followed for all of the other maximums. Another approach might be to simply select maximums that are known to be unobtainable at any location. Again, the key is to use the same values for evaluating all alternative projects.

Although numerous values must be plugged into this equation, it is still simple enough that it can be programmed on many hand-held calculators for easy computation when a large number of alternatives are under consideration. It also provides a rational application of points or credits among alternatives and perhaps a better spread of resulting cost-effectiveness values.

Integration of Other Factors

During the development of the Oakland County TSM plan (9), the foregoing safety project-evaluation procedure was expanded to integrate other factors relevant to TSM project planning and programming. Although the enhancement of highway safety was retained as the primary criterion in the evaluation process, the following criteria were also considered (10):

1. Operations improvements, including reduction in traffic delay, importance of the project to the transportation network, and improvement in operations and roadway geometrics;
2. Improvement in air quality;
3. Reduction in fuel consumption;
4. Impact on other modes;
5. Impact on social and economic factors; and
6. Improvement in maintenance and service factors.

Points were awarded to projects for improvements in the traffic operations criteria that were weighted by the existing level of service (LOS) at the project location (11,12). The improvements in air quality and fuel conservation that result from a project were based on the reduction in traffic delay effectuated by the project. The other evaluation criteria were scored on a subjective basis. Cost-effectiveness for a project is determined by summing the effectiveness points assigned to the project, dividing by the estimated project cost, and multiplying by one million. Projects are then ranked by their cost-effectiveness and budget constraints are applied. Table 4 provides an example of the final product of this procedure.

Table 4. Highway projects listed by cost-effectiveness.

Project Location	Description	Safety	Traffic Operations	Air Quality	Fuel Conservation	Intermodal Impacts	Socioeconomic Impacts	Maintenance	Total	Cost ($)	Cost-Effectiveness
Elizabeth Lake–State to Telegraph	Interconnect signals	8.8	13.0	2	2	0	0	2	27.8	3 300	8424
Main–University	Remove parking, stripe for left-turn lane	23.8	13.0	5	5	2	2	3	53.8	10 000	5380
M-59–Crescent Lake	Add left-turn phase	18.8	3.0	0	0	0	0	0	21.8	10 000	2180
Farmington–Nine Mile	Widen for left-turn lanes	36.2	21.0	3	4	2	0	3	69.2	75 000	923
John R–Woodward Heights	Widen for left-turn lanes	36.2	3.5	1	1	3	0	2	46.7	130 000	359
John R–Nine Mile	Increase corner radii	5.6	6.0	1	2	3	0	2	19.6	55 000	356
Twelve Mile–Middlebelt	Widen for left-turn lanes	17.5	3.0	1	1	1	0	1	24.5	75 000	327
Ten Mile–Novi	Widen intersection	22.5	4.2	3	4	0	0	3	36.7	150 000	245
Pontiac Trail–Decker	Widen for left-turn lanes	5.0	2.5	2	3	0	2	3	17.5	80 000	219

CONCLUSION

The final product of this entire process is a list of projects ranked according to relative cost-effectiveness. By applying budget constraints to this listing of projects, a yearly or multiyear program is devised. The process explained presents a simple technique for facilitating the resource-allocation decision. It is designed to be applicable to all local highway organizations regardless of their size or sophistication.

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Analysis of Accidents in Traffic Situations By Means of Multiproportional Weighted Poisson Model

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This article describes a model that enables traffic engineers to get insight into the factors that influence the occurrence of accidents. This model has a multiplicative form and describes how the expected number of accidents depends on road and traffic characteristics. Because of the input of observations where no accidents occurred, a logarithmic transformation to linearize the model was impossible without biasing the estimates considerably. By introducing the maximum likelihood estimation theory, a model was developed that also analyses situations where no accidents occur. This method was first applied successfully in 1974 for the analysis of accidents on Dutch polderroads. This article also describes the results obtained by the method from a study that tries to establish a relation between road and traffic characteristics on one hand and the safety of cyclists and moped riders on the other. Influencing factors are (a) motor car, moped, and cycle traffic flows; (b) width of cycle lane and median width; (c) access roads to houses; (d) type of road surface of the cycle lanes; and (e) parking bays and bus stops. A further application is given by the study of interurban car traffic. Daily traffic flows proved to be the most important variable, followed by the presence of obstacles and intersections and crossings of various kinds.

Traffic accidents are caused by errors of judgment on the part of road users or by defects in vehicles. The occurrence of accidents is related to the psychological characteristics of the traffic participants as well as to the physical characteristics under which they take part in traffic. These physical characteristics are, for instance, the weather conditions (e.g., fog or slipperiness), the light or dark period of the day, and the road characteristics. One of the tasks of the traffic engineer is to examine whether the accident rate can be lowered by improving the traffic situation.

The occurrence of accidents can be analyzed by means of mathematical models. Regression analysis is often used; sometimes analysis of variance and factor analysis are also used to ascertain the effect of road and traffic characteristics (1-3). Some have used linear regression. Often, a multiplicative model is made linear (4,5).

The use of multiple linear regression implicitly assumes that the observation results are distributed normally. This assumption is not very realistic since the analysis is specifically concerned with traffic situations in which few accidents occur. The probability that the number of accidents would become negative is not negligible in that case.

The drawback of an erroneous assumption with respect to the sampling distribution is even greater in the use of the multiplicative model linearized by a logarithmic transformation. The logarithm of zero is not defined, and a zero observation can therefore not be included in the investigation. The zero observations are sometimes omitted from the analysis. This seems undesirable because traffic situations where no accidents occur are of a very real importance. Other devices are sometimes used; for instance, a small number (e.g., 0.5) may be added to all observations (6-8). Such a pretreatment of observations can greatly affect the estimate and is therefore undesirable.

For the method we propose it is not required either. This contribution deals with the weighted multiproportional Poisson model and illustrates this method with some applications. The number of accidents is used as the dependent variable, whereas the accident rate is not. In fact, the rate depends on the dimensions used. The lengths of road segments where accidents have been observed lead to the introduction of weighted models. Accidents are related to road and traffic characteristics by means of a multiplicative or multiproportional model. The accidents are assumed to be Poisson distributed.

MULTIPROPORTIONAL POISSON MODEL

The multiproportional Poisson model is based on two assumptions. First, it is assumed that accidents are Poisson distributed with some expected value. Subsequent accidents are not correlated and the time interval between two subsequent accidents has a negative-exponential distribution. Second, the expected number of accidents (μ) is multiplicative (i.e., the product of the effects of independent variables). This model is based on the analysis of higher-order cross-classifications to test whether the factors (roadway and traffic characteristics) of influence are independent. In the use of the accident model, many roadway and traffic characteristics must be included simultaneously in the analysis. The multiplicative model introduced here is a logical continuation of the analysis of cross-classifications that contain one or two roadway features; thus, all detailed information available may be analyzed. Oppe (9) gives some theoretical and experimental justification for the use of a multiplicative model.

In addition, some road segments, which have a certain combination of factors, may differ considerably in length (L) from other segments, which have a different combination of factors. The experimental design is not balanced. As a consequence of the governmental road design policy, these are combinations of road and traffic characteristics that do not exist (e.g., roads that have a small lane width but a high car volume). Moreover, observations from long road segments are more reliable than those from short segments. The literature on this subject pays little attention to the analysis of such weighted cross-classifications (6,10). A computer package like BMDP does not contain software for the analysis of weighted cross-classifications.

The presence of weight factors is a vital difference between the method being proposed and the standard log-linear analysis of cross-classifications. Note that the ratio between the number of accidents and the weighting factor is not suitable for analysis since, in that case, the analysis will depend on the dimensions used (11).

The form of the model is

$$\mu_{klmn} = a_k \cdot b_l \cdot c_m \cdot d_n \ldots \; L_{klmn} \tag{1}$$

where

μ_{klmn} = expected number of accidents in case the explanatory variables belong to the categories k,l,m, and n;
L_{klmn} = length of the segment that belongs to the categories k,l,m, and n (if necessary, weighted by period of observation);
$a_k, b_l, c_m, d_n \ldots$ = coefficients (estimate is indicated by ^);
a,b,c,d = factors (characteristics of the road and traffic situation); and
k,l,m, and n = classes with k = 1,2,3,4,...; l = 1,2,3,4,...; m = 1,2,3, 4,...; and n = 1,2,3,4,....

Interactions can also be taken into account. This means that the influence of several independent variables together differs from that of each separate independent variable.

Since it is possible to multiply the coefficients a_k by 100 and to divide the b_l coefficients by 100 without affecting the number of accidents, a normalization is used. The coefficients are not unique. The ratios between the coefficients of any factor are unique. In performing the computations this complication is taken into account. The influence of the traffic volume can be estimated by means of one of the traffic coefficients. It is also possible to include the traffic volume directly as an independent explanatory variable, if so required. In the latter case, L_{klmn} becomes equal to the product of volume, length, and observation period.

ESTIMATION EQUATIONS

The coefficients in the accident model are estimated on the basis of the maximum likelihood method. Maximization of the likelihood gives the estimation equations. As indicated above, the nature of the occurrence of an accident is a Poisson process. Consequently, the probability of y_{klmn} accidents at an expected value μ_{klmn} is given by the equation

$$\Pr[y_{klmn}] = [\exp(-\mu_{klmn}) \cdot \mu_{klmn}{}^{y_{klmn}}]/y_{klmn}\,! \tag{2}$$

The numbers of accidents (y_{klmn}) are assumed to be independent for all combinations of k,l,m,n,.... As a result, the value of the log-likelihood function (λ) becomes

$$\lambda = \sum_k \sum_l \sum_m \sum_n \ln \Pr[y_{klmn}] \tag{3}$$

In Equation 1 the coefficients should be chosen in such a way that the log-likelihood has a maximum value. Substitution of Equations 1 and 2 in Equation 3 gives the log-likelihood function:

$$\lambda = \sum_k \sum_l \sum_m \sum_n [-a_k \cdot b_l \cdot c_m \cdot d_n \ldots L_{klmn} + y_{klmn} \cdot \ln(a_k b_l c_m d_n \ldots L_{klmn}) - \ln(y_{klmn}!)] \tag{4}$$

The maximum value of the log-likelihood is found by determining the first partial derivative for each of the coefficients and by equating it to zero:

$$\partial\lambda/\partial\hat{a}_k = \sum_l \sum_m \sum_n (-\hat{b}_l \cdot \hat{c}_m \cdot \hat{d}_n \ldots L_{klmn}) + \sum_l \sum_m \sum_n (y_{klmn}/\hat{a}_k) = 0; \quad \forall k \tag{5a}$$

It is also (equivalently) true that

$$\partial\lambda/\partial\hat{b}_l = 0, \forall\, l \tag{5b}$$

$$\partial\lambda/\partial\hat{c} = 0, \quad \forall m \tag{5c}$$

$$\partial\lambda/\partial\hat{d}_n = 0; \quad \forall n \tag{5d}$$

A set of nonlinear equations is developed, with which the coefficients are determined.

$$\hat{a}_k = y_{k\ldots}/\sum_l \sum_m \sum_n \hat{b}_l \hat{c}_m \hat{d}_n \ldots L_{klmn}; \quad \forall k \tag{6}$$

$$\hat{b}_l = y_{.l..}/\sum_k \sum_m \sum_n \hat{a}_k \hat{c}_m \hat{d}_n \ldots L_{klmn}; \quad \forall l \tag{7}$$

$$\hat{c}_m = y_{..m.}/\sum_k \sum_l \sum_n \hat{a}_k \hat{b}_l \hat{d}_n \ldots L_{klmn}; \quad \forall m \tag{8}$$

$$\hat{d}_n = y_{...n} / \sum_k \sum_l \sum_m \hat{a}_k \hat{b}_l \hat{c}_m \ldots L_{klmn};\quad \forall n \tag{9}$$

In these formulas,

$$\sum_l \sum_m \sum_n y_{klmn} = y_{k...};\quad \forall k \tag{10a}$$

$$\sum_k \sum_m \sum_n y_{klmn} = y_{.l..};\quad \forall l \tag{10b}$$

$$\sum_k \sum_l \sum_n y_{klmn} = y_{..m.};\quad \forall m \tag{10c}$$

$$\sum_k \sum_l \sum_m y_{klmn} = y_{...n};\quad \forall n \tag{10d}$$

$y_{k...}$, $y_{.l..}$, $y_{..m.}$, and $y_{...n}$ are the observed marginal frequency distributions of accidents. The coefficients are determined by an iterative method in accordance with the Gauss-Seidel principle.

The method being proposed can be modified, if necessary; for example, the Poisson distribution of the accidents could be replaced with some other distribution (gamma, Erlang) if the empirical data would indicate so.

STATISTICAL TESTING

The estimators of the coefficients are stochastic variables. Each of these stochastic variables has a probability distribution, a mean, and a standard deviation. The smaller the standard deviation of the estimator, the more reliable a coefficient is considered to be. It is examined by testing whether certain assumptions concerning the parameters of a distribution (comprised in the null hypothesis) can be rejected in favor of the alternative hypothesis. Since a multiproportional model is being used here, it should be examined whether the ratio between the coefficients of each set of classes per factor (a_k/a_1, c_m/c_1, $d_n/d_1,\ldots$) differs significantly from one.

The variation in the coefficients can be determined by means of the matrix of the second derivatives of the log-likelihood function (λ). The negative expectation of the inverse of this matrix gives (asymptotically) the variance-covariance matrix. The square root of the value of the diagonal elements of this variance-covariance matrix gives the estimated standard deviation in the coefficients. The probability distribution of the ratios (a_k/a_1, c_m/c_1, $d_n/d_1,\ldots$) is skew. As the values of the coefficients are positive integer numbers, values smaller than or equal to zero cannot occur. The natural assumption to make in testing is that the estimated coefficients are log-normally distributed. It has been ascertained by Monte Carlo simulation that this assumption is very useful (12,13). Because the procedure of drawing random numbers requires lengthy calculations, the method with the second derivative is used. The normalization is done in such a way that only the estimated values of the normalized coefficients greater than one occur.

SELECTION OF FACTORS

In some studies we estimated the effects of a large number of roadway characteristics. The results of the simultaneous estimation can then be supported by some simple strategies. Depending on the problem, a distinction can be made between the various roadway characteristics. These are selected on the basis of the hypotheses that are to be analyzed. As a tool in selecting roadway features, the likelihood-ratio test statistic (G^2) is used [see, for instance, Bishop and others (6)].

$$G^2 \overset{\text{def}}{=} 2[\lambda^* - \lambda(\mu)] = 2 \sum y_{klmn} \ln (y_{klmn}/\mu_{klmn})$$
(sum over all the observations)

where

$\lambda(\mu)$ = value of the log-likelihood function with estimated coefficients ($\hat{a},\hat{b},\hat{c},\hat{d},\ldots$);

λ^* = highest attainable value of the log-likelihood; this value of the log-likelihood function is attained if the model results become equal to the observation results; and

G^2 = chi-square distributed if the number of observations is sufficiently large (i.e., asymptotic).

The value of G^2 is calculated for each separate roadway characteristic. It can be examined by testing whether the roadway feature in question contributes significantly to the explanation. This method can also be continued for combinations of roadway characteristics. Table 1 presents an example of this methodology. Average daily traffic volume, obstacle distance, and pavement, for instance, are significant. The legally permitted maximum speed and the gradient are not.

In this way the roadway characteristics can be classified according to their explanatory strength. It would be incorrect to regard these (simple) analyses as definitive, since the effects of all other factors are ignored and the observations are classified solely on the basis of the one factor considered. Although these simple analyses indicate the relative importance of the various factors, the weighted multiproportional Poisson model, which takes account of many factors simultaneously, must be considered decisive.

The test statistic G^2 can also be used when several factors are considered. In broad outline the procedure is as follows: After two factors have been considered and analyzed separately, this is repeated for the two factors together. The effect of the two factors together is then compared with the sum of the effects of the two factors taken separately. If, for instance, the effect of the two factors together is found to be considerably smaller than the sum of the two separate effects, correlation between these factors is obvious--they explain, in an (almost) identical way, the occurrence of accidents.

COMPARISON OF OBSERVED AND ESTIMATED RESULTS

The value of the application of the model is illustrated by comparing the observation results with the estimation results, for the bicycle traffic study (one-sided bicycle lanes). The estimation results have been taken from Table 2.

In practice, two-dimensional tables are often used to search for independent variables. In the table below the relation between the accident rate per kilometer per year and the volumes of car traffic and (motorized) bicycle traffic is shown. To allow comparison with the model results, all observation results were divided by the number of accidents in the upper left cell.

	Annual Accident Rate per Kilometer			
Volume of Motorized Bicycles	<2000 Motor Vehicles	2000-4000 Motor Vehicles	4000-6000 Motor Vehicles	>6000 Motor Vehicles
<250	1.00	0.91	0.77	0.94
250-700	0.78	1.50	1.23	1.47
700-1000	0.00	2.63	3.33	4.88
>1000	0.52	0.45	5.83	6.27

The number of accidents is expected to increase with the increasing volumes of the motor vehicles and cycles. In the above table, however, the first

Table 1. Example of methodology.

Roadway Characteristic	G^2	df	Significance Level
Average daily traffic volume	266.90	6	>0.999
Points of conflict	94.32	3	>0.999
Type of obstacle	102.00	6	>0.999
Horizontal curve	63.81	2	>0.999
Obstacle distance	75.33	4	>0.999
Parallel facility	54.64	4	>0.999
Environment features	40.41	3	>0.999
Median width	49.33	4	>0.999
Sight distance	49.30	5	>0.999
Profile narrowings	9.93	1	0.997
Truck percentage	12.74	3	0.995
Shoulder width	15.96	5	0.994
Pavement width	10.27	7	~0.80
Pavement	18.50	7	0.99
Lane width	10.13	6	0.90
Permitted speed	0.67	1	~0.60
Gradient	0.29	1	~0.40
Discontinuities	0.26	2	~0.20

line and the first column might suggest that the accident rate decreases with an increase of the traffic volume. The second column and last line also present a rather illogical picture. The reason is that other roadway characteristics also affect the occurrence of accidents. This effect cannot be demonstrated in a two-dimensional table.

In the first table in the next column, the model results are shown. The table is the result of multiplying the estimated coefficients for motor vehicle volume by those for motorized bicycle volume (Table 2). The effect of other roadway features was incorporated in the other estimated coefficients but is omitted from this table. The model results are well in line with the expectations. An increase in traffic volume results in more accidents.

Volume of Motorized Bicycles	Annual Accident Rate per Kilometer: <2000 Motor Vehicles	2000-4000 Motor Vehicles	4000-6000 Motor Vehicles	>6000 Motor Vehicles
<250	1.00	1.23	1.56	1.74
250-700	1.27	1.56	1.98	2.21
700-1000	2.99	3.68	4.66	5.20
>1000	3.92	4.82	6.12	6.82

Observed results are presented in the table below. In this table the width of the bicycle lane and the width of the median between bicycle lane and roadway are included:

Median Width (m)	Accident Rate by Bicycle Lane Width: <2.7 m	> 2.7 m
<2.3	1.00	1.68
>2.3	0.81	0.73

The wider a bicycle lane with a narrow median width, the more dangerous it is. This table has led to the hypothesis of interaction. Consequently, coefficients were estimated for each cell of the matrix (so 4). These were included in model results in the following table.

Median Width (m)	Accident Rate by Bicycle Lane Width: <2.7 m	>2.7 m
<2.3	1.00	0.85
>2.3	0.65	0.54

Table 2. Estimation results for roads that have a bicycle lane on one side.

Factor	Class	L	Y	C	t-Value Between Classes: Class 1	Class 2	Class 3
1. Motor vehicle volume							
<2000	1	147	66	1.00	–	–	–
2000-4000	2	150	98	1.23	1.29	–	–
4000-6000	3	123	102	1.56	3.02	1.68	–
>6000	4	129	255	1.74	3.02	2.31	0.85
2. Bicycle volume							
<250	1	130	105	1.00	–	–	–
250-700	2	192	120	1.27	1.83	–	–
700-1000	3	48	83	2.99	8.66	5.35	–
>1000	4	78	213	3.92	9.69	9.35	1.94
3. Access points, bicycle lane side							
<225 m	1	135	199	1.00	–	–	–
≥225 m	2	414	232	0.89	0.64	–	–
4. Access points other side							
<225 m	1	130	211	1.00	–	–	–
≥225 m	2	419	310	0.97	0.17	–	–
5. Median bicycle lane							
<2.3 m	1	215	281	1.00	–	–	–
≥2.3 m	2	334	240	0.65	3.13	–	–
6. Width bicycle lane							
<2.7 m	1	235	230	1.00	–	–	–
≥2.7	2	314	291	0.85	0.96	–	–
7. Pavement bicycle lane							
asphalt + concrete	1	318	307	1.00	–	–	–
brick pavement	2	231	214	1.21	2.03	–	–
8. Sight distance							
100 percent	1	338	308	1.00	–	–	–
<100 percent	2	211	213	1.15	1.23	–	–
9. Obstacle							
No obstacle	1	411	290	1.00	–	–	–
Others	2	61	73	1.15	0.85	–	–
Trees, continuous	3	55	120	1.57	3.84	1.87	–
Trees, discrete + lights	4	22	38	1.66	2.85	1.51	0.33

Note: L = total segment length (km) weighted by the analysis period, Y = number of accidents per class, and C = coefficients estimated.

If the cells outside the diagonal are multiplied (0.85 x 0.65 = 0.55), this value differs little from the estimated value of 0.54. Consequently, the assumed interaction apparently does not exist. In the final estimation (presented in Table 2), therefore, interaction is not present.

The estimation results show that wide bicycle lanes are safer than narrow ones and that a wide median is safer than a narrow one, which is entirely in line with the expectations. The difference between model results and observation results must be attributed to the fact that apparently, in the table that shows observational results, other independent variables (e.g., volumes) come into play as well.

APPLICATION IN SPECIFIC SITUATIONS

In the Netherlands the weighted multiproportional Poisson model was used for polderroads in 1974 (14), for interurban bicycle traffic in 1978 (15), and for interurban car traffic in 1979 (16).

Figure 1. Bicycle lanes on both sides (T-roads).

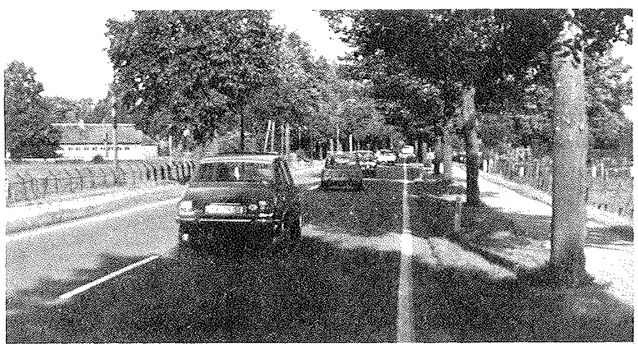


Figure 2. Bicycle lanes on one side (E-roads).

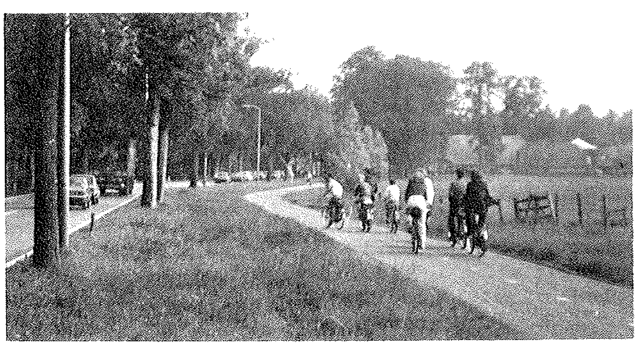
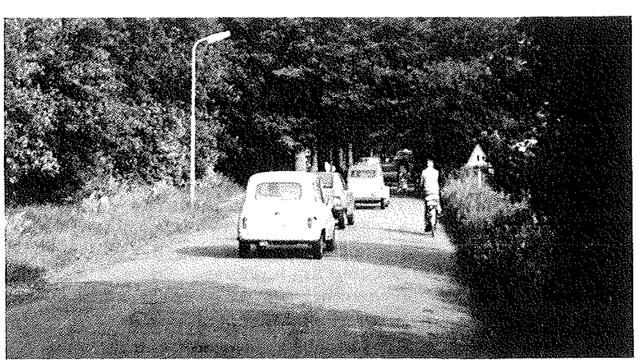


Figure 3. Road without bicycle lanes (Z-roads).



Interurban Bicycle Traffic

The study investigated 1774 accidents to motorized cyclists that resulted in severe injuries, some of which were fatal. The accidents have been taken from the national accidents survey of accidents per road segment. The roadway characteristics have been determined by a direct survey. The role of secondary and tertiary roads outside the built-up area (with a total length of 2439 km) in these accidents was studied. Some are roads with bicycle lanes on both sides (T-roads) (Figure 1), some are roads with bicycle lanes on one side (E-roads) (Figure 2), and some are roads without lanes (Z-roads) (Figure 3).

The inventory unit used was a road segment. A segment is a part of the road between intersections or junctions with public roads within which there are no changes in the most important characteristics of the road. It has a maximum length of 200 m. A distinction was made between roads without bicycle facilities, roads with a separate (i.e., reservation in between) bicycle lane on one side, and roads with separate bicycle lanes on both sides.

Some of the most important results are as follows:

1. The probability of accidents is greatly influenced by the motor vehicle volume. Average daily traffic volumes were used; the volume at the time of the accident could not be used because data were lacking. The influence of the volume on accidents is considerably greater for roads without bicycle facilities [factor 1 in Tables 2 and 3 (for illustration, only the results for E- and Z-roads are given)].
2. An increase in the bicycle volume greatly increases the probability of accidents (Tables 2 and 3, factor 2).
3. The probability of accidents is greater on roads with a wide bicycle lane and a narrow median than on roads with a less-wide bicycle lane and a wider median. This is particularly true for roads that have a bicycle lane on one side.
4. Roads that have many access points generate significantly more accidents than roads that have few or no access points (factor 3, Table 2, and factors 3 and 4, Table 3).
5. The influence of parking bays, bus stops, and so on is not significant on roads that have bicycle facilities on one side or on both sides as could be expected because there are no conflicts. On roads without bicycle facilities the probability of accidents is increased by more than 20 percent (see Table 3).
6. The influence of the presence or absence of an edge marking could not be proved (see Table 3, factor 6).

No significant influence could be demonstrated for other influence factors.

The model coefficients found are suitable for calculating the probability of accidents on a segment of a certain type. The method can also be used to determine whether no facilities, facilities on one side, or facilities on both sides are better.

Interurban Car Traffic

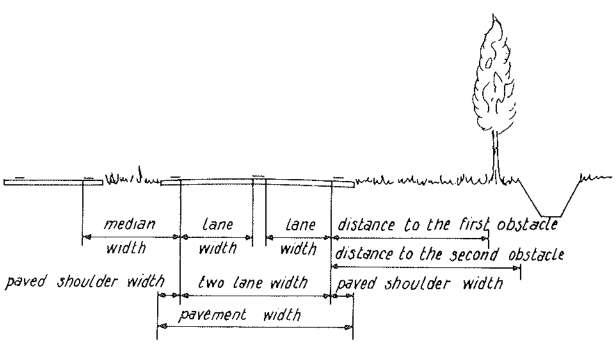
The following data are cited from Jager and Gijsbers (16). The study was concerned with 1545 accidents that caused severe injuries, some of which were fatal, on two-lane or equivalent roads maintained by the national or provincial governments (1300 km).

Table 3. Estimation results for roads that do not have bicycle lanes.

Factor	Class	L	Y	C	t-Value Between Classes Class 1	Class 2	Class 3	Class 4
1. Motor vehicle volume								
<1000	1	323	74	1.00	–	–	–	–
1000-2000	2	469	180	1.34	2.13	–	–	–
2000-3000	3	318	227	2.05	5.72	4.28	–	–
>3000	4	199	242	3.21	7.55	7.48	4.69	–
2. Bicycle volume								
<150	1	357	82	1.00	–	–	–	–
150-250	2	266	110	1.61	3.48	–	–	–
250-500	3	363	185	1.62	3.71	0.03	–	–
500-700	4	161	125	1.95	5.02	1.38	1.79	–
>700	5	163	221	2.85	6.94	4.16	5.57	3.47
3. Access points								
>500 m	1	356	110	1.00	–	–	–	–
225-500 m	2	399	195	1.41	2.42	–	–	–
<225 m	3	553	418	1.69	4.60	1.69	–	–
4. Two lanes width								
<5.0 m	1	231	87	1.00	–	–	–	–
5.0-6.0 m	2	611	304	0.90	0.70	–	–	–
6.0-7.0 m	3	394	285	1.09	0.55	1.68	–	–
>7.0 m	4	72	47	0.91	0.44	0.07	0.96	–
5. Pavement of the road								
Asphalt + concrete	1	1234	657	1.00	–	–	–	–
Brick pavement	2	74	66	1.22	1.37	–	–	–
6. Parking bays, bus stops								
Yes	1	1054	583	1.00	–	–	–	–
No	2	255	140	0.78	2.64	–	–	–
7. Marking								
Yes	1	644	376	1.00	–	–	–	–
No	2	664	347	0.97	0.41	–	–	–
8. Sight distance								
100 percent	1	675	308	1.00	–	–	–	–
<100 percent	2	634	415	1.11	1.56	–	–	–
9. Obstacle								
No obstacle	1	693	301	1.00	–	–	–	–
Others	2	212	135	1.19	1.74	–	–	–
Trees, continuous	3	253	175	1.31	2.57	0.95	–	–
Trees, discrete + lights	4	150	112	1.72	4.68	2.80	1.69	–

Note: L = total segment length (km) weighted by the analysis period, Y = number of accidents per class, and C = coefficients estimated.

Figure 4. Road characteristics of cross section.



To obtain a unit of analysis inventory, the road network was subdivided into lengths of approximately 100 m. The analysis was carried out on segments of roads, most of which were 200 m in length. The used road characteristics in a cross section are given in Figure 4. Table 4 presents the estimation results.

As may be expected, the average daily traffic volume (factor 1 in Table 4) is by far the most-important explanatory variable. The accident density is approximately a factor 3 higher on roads that have an average daily traffic volume of more than 9000 motor vehicles than on roads that have a volume lower than 3000 motor vehicles/24 h (provided all other roadway features are equal). The accident density hardly increases with an increase of the intensity over 10 000 motor vehicles/24 h. These analysis results scarcely differ from the conclusions that can be drawn from West German and Danish studies (5,17-19).

If the regression coefficients are normalized by traffic performance (roughly speaking this means dividing by volume), those roads that have a high volume are safer than roads that have relatively low volume, if all other features are the same.

The two-lane width does not significantly influence the accident density, whereas the pavement width and shoulder width do (factor 3), although the pavement width less so than the shoulder width. The two-lane width, therefore, seems to influence the accident density much less than does the paved shoulder width. The effects found for the shoulder width are significant in all cases. A paved shoulder width smaller than 0.85 m produces a greater probability of accidents than does a wider paved shoulder. The accident density is not significantly changed by increasing the width of shoulders of 1.8-2.0 m wide. The accident density is as much for roads that have a shoulder width of 1.8-2.0 m as for roads that have considerably wider shoulders. Within the classes smaller than 0.9 m, the widths of 0.4-0.5 m appear to be significantly safer than the slightly wider (0.6-0.8 m) or the slightly narrower (≤0.3 m) widths. The results found for the lane width are not in line with the conclusion drawn from other studies; i.e., that the traffic safety increases with an increase of the lane width.

The various researchers, however, do not come to the same conclusion concerning the relation between lane width and accident density: Foody and Long (20) have found a linear relation; Dart and Mann (3)

Table 4. Estimation results for lane-width study.

Factor	Class	L	Y	C	t-Value Between Classes Class 1	Class 2	Class 3	Class 4	Class 5
1. Motor vehicle volume									
<3000	1	256	172	1.00	–	–	–	–	–
3000-3999	2	177	156	1.23	1.83	–	–	–	–
4000-5999	3	282	329	1.55	4.48	2.37	–	–	–
6000-7999	4	238	329	2.07	7.59	5.30	3.56	–	–
8000-9499	5	123	236	2.72	9.83	7.56	6.32	3.18	–
≥9500	6	130	323	3.11	11.57	9.29	8.48	5.05	0.52
2. Truck percentage									
<20 percent	1	1003	1253	1.00	–	–	–	–	–
≥20 percent	2	204	292	1.14	1.89	–	–	–	–
3. Paved shoulder width									
≥0.85 m	1	131	136	1.00	–	–	–	–	–
<0.85 m	2	1077	1409	1.22	2.15	–	–	–	–
4. Obstacle distance									
Absent	1	39	37	1.00	–	–	–	–	–
2.5-3.5 m	2	1012	1197	1.21	1.13	–	–	–	–
<2.5 m	3	156	311	1.43	1.96	2.20	–	–	–
5. Median width									
Absent	1	653	703	1.00	–	–	–	–	–
≥4.0 m	2	412	612	1.15	2.29	–	–	–	–
<4.0 m	3	143	230	0.98	0.25	1.93	–	–	–
6. Type of obstacle									
Absent	1	764	829	1.00	–	–	–	–	–
Other	2	312	453	1.03	0.44	–	–	–	–
Open row of trees	3	87	145	1.26	2.41	2.04	–	–	–
Row of lighting columns	4	47	118	1.35	2.62	2.41	0.52	–	–
7. Sight distance									
≥900 m in both directions	1	263	258	1.00	–	–	–	–	–
Other	2	945	1287	1.14	1.91	–	–	–	–
8. Pavement									
Concrete	1	121	195	1.00	–	–	–	–	–
Asphalt	2	1086	1350	0.85	1.99	–	–	–	–
9. Horizontal curvature									
≥1500 m	1	982	1141	1.00	–	–	–	–	–
750-1499 m	2	128	198	1.25	2.29	–	–	–	–
≤749 m	3	96	206	1.64	6.29	2.70	–	–	–
10. Points of conflict									
Absent + crossings	1	1073	1261	1.00	–	–	–	–	–
Access points	2	111	206	1.32	3.45	–	–	–	–
Intersections	3	24	78	2.65	8.25	5.15	–	–	–
11. Profile narrowings									
Absent	1	1178	1487	1.00	–	–	–	–	–
Present	2	29	58	1.48	2.90	–	–	–	–

Note: L = total segment length (km) weighted by the analysis period, Y = number of accidents per class, and C = coefficients estimated.

and Nilsson (21) have found a parabolic relation; and Silyanov (22) has concluded a hyperbolic relation. On the whole the results for shoulder width and pavement width are in keeping with the conclusions drawn from the study of literature [Bitzl (17,18), Foody and Long (20), and Silyanov (22)].

The type of obstacle (factor 6 in Table 4) significantly influences the accident density. Open rows of trees and rows of lighting columns significantly increase the probability of accidents by 26 percent. With regard to the effects of the rows of lighting columns, a reservation should be made. In general, lighting is used along motorways only if the traffic volume in conjunction with the road situation calls for such a provision.

The obstacle distance (factor 4) significantly influences the accident density. The distance from the inner side of the edge marking of the lane to the first obstacle in the shoulder is used as a measure for the obstacle distance. The analysis makes obvious that the accident density decreases with an increase of the obstacle distance. The accident density for obstacle distances smaller than 2.5 m was found to differ significantly from that for obstacle distances greater than 2.5 m. Significant differences in accident density could be found neither for obstacle distances between 0 and 2.5 m nor for those greater than 2.5 m.

The presence of most types of conflict points (factor 10) significantly increases the accident density. After traffic volume, this road feature influences the accident density most. A distinction was made between the following points of conflict:

1. Pedestrian crossings and bicycle crossings, both with and without traffic lights, and crossings for mixed traffic;
2. Residential and agrarian access points and minor intersections; and
3. Type B intersections (intersections without road signs and without changes in their cross sections).

Access points and B-type intersections significantly influence the accident density. The presence of type B intersections increases the accident density considerably more than that of access points.

Horizontal curves (factor 9) that have a radius greater than 1500 m do not affect the accident density. With a decrease in the horizontal radius, the accident density increases significantly.

The sight distance (factor 7) significantly affects the accident density. A sight distance greater than 900 m is significantly safer than a more-restricted one. With a further decrease of sight distances smaller than 900 m, the increase of the probability of accidents was found to be insignificant. From other studies (5), it appeared that sight distances smaller than 400 m result in a higher accident density than do sight distances greater than this. The literature study showed that sight distances greater than 400 m hardly affect the accident density at all.

Beside the roadway features mentioned, the median width (factor 5), the type of pavement (factor 8), the truck percentage (factor 2), and the profile narrowings also affect the probability of accidents, though to a less extent. For some features no relation to the probability of accidents can be demonstrated. This applies to the legally permitted maximum speed and the presence of grades.

EVALUATION

The weighted multiproportional Poisson model presented here has yielded practical results when used in traffic situations with few accidents. Road and traffic characteristics affect the occurrence of accidents substantially. The estimated results enable the traffic engineer to design and evaluate safety measures.

ACKNOWLEDGMENT

M.C. Huisman was also closely connected with the development of the method. Tables and results were taken from the studies by Jager, Van der Wal, and Gijsbers, which were made for Rijkswaterstaat (Department of Public Works). We are grateful for having been allowed the use of these data.

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Conceptual Development of Exposure Measures for Evaluating Highway Safety

MYUNG-SOON CHANG

An overview of exposure measures such as distance, time, traffic volume, vehicle hours, and vehicle miles used in the past for evaluating accidents on highways and intersections is presented. Their inadequacy and insufficiency are discussed. The conceptual exposure measures for evaluating highway safety are presented for the sections between signalized intersections and at intersections. Exposure measure is suggested to include all highway and traffic elements that affect accidents in the highway-traffic-environment system. Also suggested is that the number of accidents is the square of the exposure measure that operates in the highway-traffic-environment system.

Accident information is required for a variety of safety activities undertaken by states and localities. This information assists in identification of safety problems, establishment of priority locations for safety improvements, and evaluation of specific accident countermeasures. Although this information is essential, some basic problems exist. Accurate accident data are difficult to obtain and, in some cases, totally unavailable. In addition, accident data must be combined with exposure measures in order to place the accident information in perspective so that the effects of various highway and traffic elements on accident risk can be explicitly compared within or between classifications of interest.

For a long time, highway engineers and researchers have realized the necessity of accident-exposure measures. Thorpe (1), in 1967, pointed out that the lack of knowledge on accident-exposure measures severely hampers accident-reduction efforts. Unless the exposure is known, the relative hazards of various situations cannot be compared.

To use accident data without using the appropriate exposure measure can be misleading. Council and others (2) reported that a simple tally of accidents indicates that daytime accidents are more frequent than nighttime accidents. However, when mileage driven during the two periods is considered, the indication is reversed and the risk of a nighttime accident is about twice that of a daytime accident. The use of the appropriate type of exposure measure not only clarifies the relation but sometimes alters the conclusion.

Carroll (3) explained that the primary use of exposure data was the identification of highway safety problems and evaluation of various countermeasures. Exposure data are needed to determine the optimum cost-effectiveness with respect to the classifications of the types of roadways, vehicles, accidents, and the environment.

Carroll and others (4) defined exposure as the frequency of traffic events that create a risk of accidents, measured in vehicle miles of travel (VMT). Vehicle mileage appears to be the prevalent choice to measure the amount of risk for accidents. However, a simple argument shows that this is neither the always acceptable choice nor necessarily the best choice. For example, a car that is driven slower than another car over the same distance, all other things being equal, will meet more on-coming cars than will the other and will therefore have more possibilities of getting into certain types of accidents. This example points out that the time spent on the road appears to be a better measure of the exposure than mileage. However, both are not perfect exposure measures: The same amount of miles or time spent on a road that has fewer intersections is less dangerous than the same exposure on a road that has more intersections, as evidenced by the lower accident rates on limited-access highways. Therefore, Joksch (5) points out that development of a measure of exposure that combines time or mileage with other relevant factors would be desirable.

Haight (6) refines exposure further by relating the size and power of vehicles in the traffic stream, the age and experience of the drivers, weather conditions, time of day, and various classes of accidents. Many factors of the road transportation system could reasonably enter into a definition of exposure. The unanswered problem is in determining what these factors are and what importance should be attached to each.

Exposure measures to evaluate the number of accidents experienced by an individual or group of individuals are not of interest in this study. In other words, the concept of accident proneness (7) (i.e., those situations where some individuals are more likely to have an accident than others due to some characteristic property of theirs) will not be considered.

Highways will be classified into two segments in this paper: sections between signalized intersections and sections at intersections. An overview of exposure measures used in the past for these two segments is presented. Conceptual exposure measures to account for accident risk in the highway-traffic-environment system are suggested.

EXPOSURE MEASURES USED BETWEEN SIGNALIZED INTERSECTIONS

Several exposure measures have been employed for the area between signalized intersections in the past. These exposure measures are mainly in the form of distance, time, traffic volume, and the interaction (or product) of these elements such as vehicle miles and vehicle hours.

Distance

The exposure measure in terms of distance is expressed in miles. Accident rate will be expressed in accidents per mile as follows:

$$R = A/L \tag{1}$$

where

R = accident rate,
A = number of annual accidents, and
L = section length (miles).

We assume by using this measure that longer sections have a higher risk of accidents than shorter sections. However, in highway environments, when the traffic volume on intersecting driveways and side streets increases, lengths of the noncontrolled sections become smaller due to the need for traffic control devices such as signals. In other words, the natural evolution of traffic development makes the shorter highway sections more dangerous than the longer highway section. This is easily seen in the dichotomy of urban and rural highways, in which the former has shorter sections and the latter has longer sections. In addition, the meaningfulness of the exposure measure is reduced as the distance over which the rate is computed becomes smaller. This means that, at a single point, the mileage-based accident rates are completely meaningless.

One possible improvement in the expression of an accident rate in terms of section length is one that involves both the length of section and the number of conflicting movements that operate along the sections. This will be accomplished by using measures such as the number of access points, access trips to and from commercial driveways, and the ratio of access trips to through trips (8).

Time

The exposure measure in terms of time is expressed as the number of hours driven, which can be derived mathematically by the division of section length by average speed as follows:

$$R = A/(L/S) \tag{2}$$

where S is the average speed in miles per hour, and others are defined previously.

The use of time as an exposure measure is based on the assumption that an increase in time spent on the road is accompanied by an increase in accident risk. However, higher speeds are, in general, more dangerous than lower speeds. Since estimation of speed on an individual basis is not of interest, estimation on a highway system basis will lead to vehicle hours of operation as follows:

$$R = A/(VM/S) = A/VH \tag{3}$$

where VM is the vehicle miles obtained by traffic volume times section length and VH is the vehicle hours spent on the highway system.

The exposure measure of vehicle hours takes into account the time drivers spend on the highway system. Although vehicle hours of operation is useful to analyze vehicle reliability among different transportation modes, it does not appear to be a good exposure measure for highway systems. Two reasons can be cited for its inappropriateness (9). The first is that not all time spent in travel is of equal accident risk. The second is that it tends to neglect those highway accidents that occur during relatively short time periods.

A possible alternative approach to expressing accident rates in terms of the effect of time spent on the highway would be a technique that adjusts the numerator as opposed to the denominator. In other words, the time spent effect can be appropriately taken into account by the classification of daytime versus nighttime accidents, dry-pavement versus wet-pavement accidents, and accidents caused by different vehicle types due to different speeds. Within this classification, vehicle hours may be analyzed as one of several factors.

Traffic Volume

The exposure measure in terms of traffic volume is usually reported as annual average daily traffic (AADT), peak-hour volume (PHV), and off-peak-hour volume (OPHV). The accident rate is defined as follows:

$$R = A/V \tag{4}$$

where V is the traffic volume in the form of AADT, PHV, or OPHV.

Numerous studies show high correlation among AADT, PHV, and daytime OPHV (8,11); therefore, the use of one of these three volume classes may be satisfactory.

The exposure measure of traffic volume takes into account the interaction effect among vehicles because it is assumed that accident risk increases as volume increases. However, as traffic volume increases toward capacity, accident risk decreases. In addition, as traffic volume increases, single-vehicle accidents, in general, decrease. For example, Chapman (9) reported the empirical relation by using New Zealand data, as follows:

$$p = \exp(-0.000\,865V) \tag{5}$$

where p is the proportion of single-vehicle accidents to all accidents and V is the traffic volume per hour in both directions.

The proper approach to express accident rate in terms of traffic volume is to separate the numerator into single-vehicle versus multiple-vehicle accidents, vehicle types such as trucks and cars, and combinations of these classifications. Within this classification, traffic volume may be analyzed as one of several factors.

Vehicle Miles

The most commonly used exposure measure in accident rates is vehicle miles. The expression for this measure is as follows:

$$R = A/VM \tag{6}$$

Note that the magnitude of the constant, such as 100 million vehicle miles or million vehicle miles, in no way affects the relative comparison of accident rates.

It is assumed in this exposure measure that accident risk increases as more vehicles travel more miles. However, conceptually the assumption itself appears to be incorrect. As is reported by many studies on accident likelihood, the probability or possibility of a driver who has extensive driving experience being involved in an accident is far lower than that of a younger driver who has less driving experience. Although part of the reason is attributable to driver characteristics, Greenberg (12) showed the existance of an accident-experience learning curve. He indicated that the number of accidents per mile decreases as the cumulative mileage increases and compares this with industrial learning curves of occupational injury as it relates to cumulative volume of production. In addition, an exposure measure of vehicle miles does not take into account the different risks associated with highway geometry and their interaction with traffic volume.

CONCEPTUAL DEVELOPMENT OF EXPOSURE MEASURES

From an overview, it appears that conventional exposure measures of accidents are either inappropriate or insufficient. Conventionally, an exposure measure is treated exclusively of highway geometrics and the roadway environment. The variables typically used for exposure measures were time, distance, traffic volume, and the product of these elements. However, as pointed out, many aspects of the highway-traffic-environment system enter into the exposure measure. The question is, What highway and traffic elements should be included in exposure measures and of what importance and relation are these variables to accident risk?

Previously, an induced exposure measure first suggested by Thorpe (1) and modified by Haight (6) had received attention due to the difficulty in estimating accurate exposure measures by driver and vehicle types. However, the induced exposure measure suggested can only indicate what importance each of the variables of interest has relative to other variables on accident risk. Aside from the validity of the assumption and limitations on the applicability of accident causative factors, it cannot determine the functional relation of the variables to explain accident risk. As pointed out by Carr (11), "The best measure of exposure is clearly some form of site-matching in a rigorously controlled, expert investigation." Accident rates are expressed as follows:

$$A/E(x) = f(x) \qquad (7)$$

where

A = annual number of accidents,
E(x) = exposure measure as a function of x,
f(x) = accident rate as a function of x, and
x = highway and traffic elements that affect accidents.

As mentioned, typical variables used for the exposure measure function [E(x)] were time, distance, traffic volume, and the product of these elements. However, many factors in the highway-traffic-environment system enter into exposure measures that represent potential accident risk.

Let E(x) in Equation 7 be a linear combination of variables related to the accident risk. Then,

$$E(x) = a + b_1x_1 + b_2x_2 + \ldots + b_nx_n \qquad (8)$$

where a, b_i are constant $(i = 1,\ldots,n)$ and x_i is the exposure variable $(i = 1,\ldots,n)$.

Then, each b_i will represent the different weights associated with accident risk that correspond to each exposure variable x_i. Furthermore, the function f(x) in Equation 7 is also composed of variables to indicate the different contribution of accident rate for each exposure variable that exists in the highway-traffic-environment system. Therefore, the two functions E(x) and f(x) should be equal because a variable could not possess different weights associated with accident risk. That is,

$$E(x) = f(x) \qquad (9)$$

Substitution of Equation 9 into Equation 7 yields

$$A = f^2(x) = E^2(x) \text{ or } \sqrt{A} = E(x) = f(x) \qquad (10)$$

Equation 10 shows that the annual number of accidents is the function of the square of the exposure measure that is the combination of variables. Note that the concept is equally applicable to both linear and nonlinear combinations of variables.

Empirical Evaluation of Exposure Measure Used Between Signalized Intersections

The concept is empirically supported from the study by Heimbach and others (8) by using the North Carolina accident data. The objective of the study was to investigate the effect of lane width on traffic operations and accidents on urban four-lane arterials. Sites were limited between signalized intersections that are not influenced by traffic signals. Accident data were classified into four groups based on the initial classification of 17 accident types. They were (a) all accidents, (b) flow-interruption accidents including rear-end accidents and accidents due to vehicles attempts to enter or leave side streets or adjacent land use activities, (c) lane-width accidents due to roadway geometry and maneuvering skills, and (d) lane-maneuver accidents as a subset of lane-width accidents to cover lane-encroachment and lane-changing accidents. It becomes apparent in grouping accident data that accidents that involve flow interruptions are not related to lane width.

The study analyzed four types of accident exposure measures. These are vehicle accidents per year, annual vehicle accidents per mile, annual vehicle accidents per 100 million vehicle miles, and the square root of the annual vehicle accidents.

By using multiple linear regression, the analysis obtained statistically significant independent variables and a coefficient of multiple determination (R^2) for various accident models. The dependent variables used are as follows:

RVAPY = square root transform of total vehicle accidents per year,
VAPY = total vehicle accidents per year,
VAPM = total vehicle accidents per year per mile,
VART = total vehicle accidents per year per 100 million vehicle miles,
RFIAPY = square root transform of flow-interruption accidents per year,
FIAPY = flow-interruption accidents per year,
FIAPM = flow-interruption accidents per year per mile,
FIART = flow-interruption accidents per year per 100 million vehicle miles,
RLWAPY = square root transform of lane-width accidents per year,
LWAPY = lane-width accidents per year,
LWAPM = lane-width accidents per year per mile,
LWART = lane-width accidents per year per 100 million vehicle miles,
RLMAPY = square root transform of lane-maneuver accidents per year,
LMAPY = lane-maneuver accidents per year,
LMAPM = lane-maneuver accidents per year per mile, and
LMART = lane-maneuver accidents per year per 100 million vehicle miles.

The independent variables used are as follows:

NNINT = number of side street intersections per mile,
ATCDW = number of access trips to and from commercial driveways,
ADT = average daily traffic,
HR = square root of the sum of the changes in horizontal direction,
VR = square root of the sum of the changes in vertical elevation,
LW = total traffic lane widths,
NINT = number of side street intersections,
TACR = total access trip conflict ratio (sum of access trips divided by ADT), and
NATCDW = number of access trips to and from commercial driveways per mile.

Table 1 shows that the models that involve the square root of the annual number of accidents (RVAPY, RFIAPY, RLWAPY, RLMAPY) not only had the greatest explained variation (see R^2) but also demonstrated the variables attributable to different accident types. For example, the flow-interruption accidents (about three-quarters of all accidents) that involved rear-end accidents and accidents due to vehicles entering or leaving side streets and driveways are not associated with highway geometric

Table 1. Comparison of accident rate models.

Grouping	Dependent Variables	Independent Variables Significant at $\alpha = 0.10$	R^2 (%)	Detransformed[a] R^2 (%)
All accidents	RVAPY	NNINT, ATCDW, ADT, HR, LW, VR	72	76
	VAPY	NINT, ATCDW, ADT	69	NA
	VAPM	NNINT, ATCDW, ADT	58	NA
	VART	TACR, NNINT, NATCDW	37	NA
Flow-interruption accidents	RFIPAY	NINT, ATCDW, ADT, TACR	69	76
	FIAPY	NINT, ATCDW, ADT	70	NA
	FIAPM	NNINT, NATCDW, ADT	56	NA
	FIART	TACR	23	NA
Lane-width accidents	RLWAPY	NINT, ATCDW, ADT, LW, VR, HR	73	65
	LWAPY	NINT, NCDW, ATCDW, ADT HR	63	NA
	LWAPM	NNINT, ATCDW, ADT LW	57	NA
	LWART	NNINT, NNCDW, LW, HR, VR	49	NA
Lane-maneuver accidents	RLMAPY	NNINT, ATCDW, ADT, LW, HR, VR	70	66
	LMAPY	NINT, ATCDW, ADT	61	NA
	LMAPM	NNINT, ADT, ATCDW	53	NA
	LMART	NNINT, ATCDW, LW	38	NA

[a]Detransformed is the process of converting RVAPY to VAPY by using significant figure.

elements. A rather significant association with those factors that interrupt steady flow such as (a) the number of intersections, (b) the number of conflicting movements due to commercial driveways, (c) the average daily traffic, and (d) the ratio of conflicting movements to ADT is indicated. However, lane-width accidents and lane-maneuver accidents, which are assumed to be due to not only traffic volume but also highway geometry, revealed exact relations by such variables as lane width, horizontal alignments, and vertical alignments. However, other accident-exposure measures, such as section length and vehicle miles of travel, failed not only to explain more variation but also to relate these relations accurately. The study revealed that accident rate per vehicle mile is the poorest model, in terms of both the least association with the variation for all four accident groupings and the misleading relation with classified accident characteristics contrary to the general tendency to take it as granted. If others would analyze the data by using only accident rates per vehicle mile, their conclusion would be erroneous and misleading.

The most important thing to note in Table 1 is that the different types of dependent variables are associated with different independent variables. Therefore, for different accident-exposure measures the countermeasure will be different. Thus, different exposures will adversely affect the efforts to improve safety.

Exposure Measures for Intersections

Each year about half of all accidents that occur in urban areas take place at intersections, and in rural areas about a quarter of all accidents are intersection related (13). Unsignalized intersections and signalized intersections have different risks of different accident types. Universally accepted is that, in general, proportionately more angle collisions take place at unsignalized intersections and more rear-end collisions take place at signalized intersections. The accident-exposure measure for intersections should reflect these characteristics, among others.

Exposure measures used in the past for intersections were based on the concept of conflict points, defined as the points or sections where two-directional traffic meets together. These will be points and sections where crossing, merging, and diverging maneuvers occur. Difference was found in the definition of conflicting maneuvers and the combined forms of traffic volume. Some treated all crossing, merging, and diverging movements as conflict traffic, and others considered one or more of them as conflict traffic. Also, some treated conflict traffic volume separately and others considered them as the sum of one or more approach-leg traffic volumes.

Note that these exposure measures, based on collision points, are only applicable to multivehicle accidents and not to single-vehicle accidents. Of course, single-vehicle accidents should be looked on as a function of a traffic volume not of a pair of conflicting volumes. In both cases, other elements that operate at the intersection, such as traffic control devices, speed, and geometry, should be examined.

Unsignalized Intersections

Grossman (14) defined collision points as conflict points in crossing maneuvers only. The exposure index at an intersection is defined as the total summation of the pairs of traffic volumes (ADT) at these collision points. Surti (15) used the same collision-points concept but added the merging maneuver. He used the product of the pair of traffic volumes at collision points by using peak-hour volume. However, he did not differentiate the different likelihood of accidents for different maneuvers.

Peleg (16) proposed collision points as the conflict points for crossing, merging, and diverging maneuvers. He considered an exposure measure as the product of the total number of vehicles per hour and the total number of collision points. However, this approach neglects that not all of the traffic at the intersection is in conflict at every collision point.

Some researchers approached intersection accident exposure as two intersecting conflict zones instead of conflicting points. From this concept, Chapman (17) proposed an exposure measure at a single conflict zone as follows:

$$E = [1 - \exp(-q_1 t)]\ [1 - \exp(-q_2 t)]\, T/t \tag{11}$$

where

E = accident exposure over time T,
q_1, q_2 = flows per unit time, and
t = time taken for a vehicle from direction 1 to pass in front of a vehicle from direction 2 plus the time for a vehicle from direction 2 to pass in front of a vehicle from direction 1.

Holland (18), who independently used a similar approach, added overall conflict zones within a four-leg intersection and derived the basic equation below for a range of volumes and turning flows.

$$E = KV_1{}^a V_2{}^b \tag{12}$$

where

E = accident exposure per time unit,
V_1, V_2 = hourly aggregate major and minor traffic volume, and
K, a, b = constants.

Richardson (19) generalized the Chapman (17) and Holland (18) approaches by allowing that either a direction A vehicle could conceptually hit a direction B vehicle or vice versa and the directional speeds could both be different. Richardson's theoretical exposure formulation is as follows:

$$E = T\Big[\{[1 - \exp(-q_A t_B)]\,[1 - \exp(-q_B t_B)]/t_B\} + \{[1 - \exp(-q_A t_A)]\,[1 - \exp(-q_B t_A)]/t_A\}\Big] \tag{13}$$

where

E = accident exposure over time T,
q_A, q_B = direction A and B flows per unit time, and
t_A, t_B = time for an A vehicle to clear the conflict zone and the time for a B vehicle to clear the conflict zone, respectively.

Hodge and Richardson (20) attempted to evaluate Richardson's (19) theoretical exposure formulation by using a simulation model. Their simulation results suggest that the exposure level between two crossing movements is simply proportional to the product of the intersecting volume. That is,

$$E = V_1 \cdot V_2 \tag{14}$$

The generalization of this equation would be

$$E = K(V_1 \cdot V_2)^c \tag{15}$$

where c is a constant.

Tanner (21) found that c equals 0.5, Leong (22) suggested c equals 0.42, and Hodge and Richardson (20) found c to be equal to 1. In summary, exposure measures suggested for unsignalized intersections in the past were as follows. For simplicity, these are shown in mathematical form.

$\Sigma(V_i + V_j)\ \delta_{ij}$ Grossman (14),
$\Sigma(V_i \cdot V_j)\ \delta_{ij}$ Surti (15),
$N\ \Sigma\ V_{ij}$ Peleg (16),
$(V_1 \cdot V_2)^a$ Tanner (21) and others, and
$V_1{}^b \cdot V_2{}^c$ Holland (18).

where

V = traffic volume,
i,j = traffic direction,
N = total number of conflicting points,
V_1 = major traffic volume,
V_2 = minor traffic volume,
a,b,c = constants, and
δ_{ij} = 1 if i and j are in conflicting points, 0 if i and j are not in conflicting points.

Note in the above summary that exposure measures suggested in the past were exclusive of intersection geometry and other traffic elements except traffic volume. Also note that the conventional approach treated crossing, merging, and diverging maneuvers as having the same accident risk.

Extension of Exposure Measures for Intersections

A logical assumption may be that different conflicting maneuvers have different accident risk. Two approaches may be possible to reflect this assumption. One is to differentiate the different traffic conflicts and the other is to differentiate the functional form of interactions between different traffic conflicts. For example, crossing maneuvers at intersections may have greater accident risk than other conflicting maneuvers and can be reflected by the product form while others are reflected as the summation form. Also necessary is that the intersection geometric features and other traffic elements, including traffic control devices, be included for measurement of intersection exposure. Therefore, the suggested measure of intersection exposure will be oriented as follows:

$f\{(V_i \cdot V_j)^p\ \delta_{ijk},\ (V_i + V_j)^q\ \delta_{ijk},$ and other intersection geometric and traffic elements$\}$

where

p,q = constant (probably, $0 < p < 1$),
k = type of conflict maneuver (crossing, merging, and diverging), and
δ_{ijk} = 1 if i and j having maneuver type k are in conflicting directions, 0 if i and j having maneuver type k are not in conflicting directions.

Signalized Intersections

At signalized intersections, the magnitude of accident risk depends not only on conflicting traffic volumes but also on site parameters such as signal phases, cycle length, splits, lens size, signal mountings, and the types of signal actuation. These components of traffic signals are found to be significantly related to traffic accidents at signalized intersections (23,24).

An accident exposure measure is desirable that can incorporate as many factors as is reasonable to distinguish varying accident experiences at signalized intersections. Thus, the suggested exposure measure at signalized intersections will be oriented as follows:

$f\{(V_i \cdot V_j)^p\ \delta_{ijk},\ (V_i + V_j)^q\ \delta_{ijk},$ other intersection geometric and traffic elements, and components of traffic signals and their operation$\}$

Note, again, that the exposure measures for both unsignalized and signalized intersections should be developed from the relation of $A = f^2(x)$ presented in Equation 10. A word of caution is added to the boundary of intersection accidents that is different from study to study. Some studies did not even mention what distance from the intersection was defined as the point where accidents become intersection related. The definition of intersection accidents with respect to distance to the boundary should be explicitly established for both the major and minor streets.

CONCLUSIONS

An overview of exposure measures such as distance, time, traffic volume, vehicle hours, and vehicle miles used in the past for evaluating accidents on highways and intersections revealed that they are inadequate and insufficient primarily for the following two reasons:

1. Conventional exposure measures were used exclusive of highway geometric and other traffic elements that affect accident risk and
2. Conventional exposure measures failed to recognize that accident rate is an equivalent expression of accident-risk exposure, which operates in the highway-traffic-environment system.

For sections between signalized intersections, the conventional exposure measure of using a single variable is to be replaced as an exposure measure that can encompass all highway and traffic elements that affect accidents. For intersections, the conventional exposure measure that treats the different traffic conflicts as the same accident risk is to be replaced as an exposure measure that can distinguish the propensity of accident risk of different traffic conflicts.

For both highways and intersections, exposure measure should contain all highway geometric and traffic elements that affect accidents. The number of accidents is the square of the exposure measure operating in the highway-traffic-environment system.

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Abridgment

Road Markings as an Alcohol Countermeasure for Highway Safety: Field Study of Standard and Wide Edgelines

NICHOLAS D. NEDAS, GERALD P. BALCAR, AND PRESTON R. MACY

Reflectorized road markings have long been recognized as a cost-effective means of reducing road accidents, particularly at night. In recognition, first, of the need to reduce the 50 percent of fatal road accidents that involve alcohol-impaired drivers, and second, of the high incidence of alcohol impairment at night, this study sought to determine whether wider-than-standard edgelines would serve as an alcohol countermeasure. A vehicle positional study was conducted on two-lane rural roads in northern New Jersey in which four edgeline width conditions (0, 4, 6, and 8 in) were evaluated. From lateral position measurements taken photographically every 100 ft, driver performance was analyzed by using six different methods. The 16 male test subjects each drove twice—once after they consumed placebo drinks [0.00 blood alcohol concentratrion (BAC)], and the other time after they consumed either placebo drinks or a controlled alcohol dosage (0.05 or 0.08 BAC). Prior research was corroborated in that the test subjects showed improved driving performance when edgelines were present and reduced performance when they were alcohol impaired. The presence of wider-than-standard edgelines was found to incrementally enhance the benefits derived from standard 4-in wide edgelines for both unimpaired and alcohol-impaired drivers. When alcohol is present, even at the relatively low BAC levels examined in this research, the visual communication link between the roadway and the driver is interrupted. The improved driving performance of the test subjects in the presence of wide edgelines indicates that strengthening the visual signal at the road edge may compensate to some degree for alcohol impairment and hence reduce the risk of accidents. Since the effects of alcohol on driver vision are similar to the effects of fatigue, drugs, and reduced visual ability due to old age, wide edgelines are likely to also benefit those with these other types of impairment.

Alcohol is a factor in up to 50 percent of all fatal accidents (1) and has historically been counteracted in the United States by highway safety programs based on law enforcement, health therapy, and public education. However, despite the best efforts of these programs, the alcohol problem still frustrates highway safety planners, and it is likely that, if major progress is to be made, new approaches to solving the problem must be developed.

Recent highway safety research has developed two basic propositions that are relevant when considering new solutions (2):

1. It is the ability of the driver to negotiate a highway that is the ultimate consideration in whether or not an accident will occur, and
2. The most-important single effect on driver behavior is the safety potential of the road itself.

By using these two propositions as a starting point, researchers at Potters Industries asked whether a traffic engineering approach to the road environment would be effective in combating the safety problems of the alcohol-impaired driver.

RESEARCH REVIEW

A review of research on alcohol impairment indicates that, although the unimpaired driver generally maintains good visual communication with the roadway, the presence of even small quantities of alcohol tends to block this visual linkage. Since at least 90 percent of all guidance information received by drivers comes from visual sources (3), this blockage creates a critical problem.

Previous research indicates that alcohol affects drivers through various mechanisms, all of which interreact with one another. For example, the alcohol-impaired driver has relatively low sensitivity to contrast, so that all objects seen along the highway tend to merge into the same shading, and the driver's ability to distinguish one object from another is reduced. In addition, alcohol impairment reduces peripheral vision, so that the driver tends to receive visual information only from the center of the roadway.

Another result of alcohol impairment is a tunnel-vision effect, in that the driver sees the road as if he or she were looking at it from inside a tunnel. Simultaneously, the impaired driver's visibility distance is shortened, so that visual search is concentrated on objects close to the front of the vehicle. Hence, the driver is not able to anticipate situations as well as when in an unimpaired condition.

These problems are compounded because alcohol reduces the ability to process information. In addition, the impaired driver has a relatively inflexible searching strategy when viewing objects in the roadway. He or she concentrates his or her visual search strategy on a few items for relatively long periods of time; in an unimpaired searching strategy he or she acquires information from many objects, each viewed for relatively short time periods. The alcohol-impaired driver also suffers some loss of dynamic visual acuity, so that objects in the periphery that are in motion relative to the vehicle, such as signs and signals, appear blurred. Finally, the alcohol-impaired driver is relatively indifferent to deviations in the driving path. All drivers tend to weave to some extent in the driving lane, but the weaving on the part of an impaired driver is much more pronounced, as a result of taking corrective action rather late, and such corrections tend to be over-corrections.

All of these effects of alcohol impairment directly impact the visual link between the road and the driver. Potter's researchers hypothesized that improved delineation, particularly at the edge of the roadway, would improve the visual linkage between the roadway and the alcohol-impaired driver and reduce the probability of road accidents. Controlled tests conducted in both the United States and Europe have shown that continuous 10-cm wide (4 in) reflectorized road markings at the road edge are a highly cost-effective means of reducing road accidents. Since these markings are particularly effective at night, when the highest incidence of alcohol impairment occurs, it is likely that they are already acting as an alcohol countermeasure; the presence of a stronger pattern at the road edge was considered likely to have an incremental benefit for the alcohol-impaired driver.

STUDY DESIGN

In order to safely conduct a realistic controlled study, it was decided not to conduct either a before-and-after accident study or a simulator study. Instead, surrogate measures of driver performance on two-lane rural roads were studied. Vehicle position reflects driver performance, and prior research has shown positional data to be usable in place of before-and-after accident data to predict accident probability.

Research conducted at Pennsylvania State University in 1969, by the Federal Highway Administration (FHWA) in 1977, and by Illinois in 1978, showed accident rate to be a function of the driver positional variability, vehicle positioning in the center of the lane, and driver-to-driver grouping (4, pp. 276-283; 5, pp. 85-99; 6). The Potters Industries research analyzed these performance measurements as well as measures of driver path range and average vehicle speed. In a refinement of the techniques used in the prior research, this study evaluated vehicle position photographically every 100 ft in each of the 14 0.8-km (0.5-mile) test sections. The prior research had measured vehicle position electronically at only two locations in each test section.

The study evaluated four edgeline conditions--no edgelines and edgelines 10, 15, and 20 cm (4, 6, and 8 in) in width. Three dosage levels of alcohol were applied--a placebo level of 0.00 blood alcohol concentration (BAC) and 0.05 and 0.08 BAC. Sixteen test subjects were selected from among male students aged 21-25, who were representative of the highest-risk group in the driving population.

During the conduct of the test between midnight and 3:00 a.m., the test course was closed to traffic by the police, who were positioned out of sight of the test subjects. These subjects drove in dual-controlled test cars and were accompanied by licensed driving instructors. Each subject drove twice, and his trials were separated by one week. One trial was in a control condition with the subject only having placebo (0.00 BAC) drinks; the other trial was conducted after the subject consumed either another placebo or a controlled alcohol dosage (0.05 or 0.08 BAC).

DATA ANALYSIS

More than 9200 measurements of vehicle position were collected from the 14 test sections, and the results of the data analysis illustrate the vital role that reflectorized edgelines play in visual communication between the roadway and the driver. These beneficial effects are apparent in the following analysis of the six measures of driving performance for the 10 curved test sections. Since examination of the data for the 4 tangent test sections yielded no strong conclusions, they have been omitted from the subsequent discussion.

Analysis of the driving range was conducted by segmenting the range into its central 70 percent increment as well as separate consideration of its left and right 10 and 5 percent extremes. The effect of alcohol on the drivers, regardless of edgeline condition, was to increase the range at both the left and right extremes (Figure 1). When edgeline width was considered, it was apparent that the effect of increasing edgeline width was to move the drivers away from the edgeline toward the centerline. This occurred for both the placebo-dosed drivers (0.00 BAC) and drivers dosed to the 0.05 and 0.08 BAC levels. However, this shift toward the centerline did not increase the number of centerline incursions. Rather, the range was compressed against the centerline, so that more driving was in the lane and centrality of positioning was greater.

When analyzing the statistical significance of these data (Figure 2), it can be seen that, when the edgeline width increases from 10 cm (4 in) to 20 cm (8 in), there are no statistically significant positional changes at the left extreme of the range. Conversely, only at this increased width of 20 cm does the right extreme 5 percent of the range show a statistically significant movement away from the edgeline and toward the centerline. This indicates that, for the best improvement in driving performance, the minimum incremental width desired is a further 10 cm rather than merely a 5 cm (2 in) increment.

The results confirm that alcohol is a safety hazard, and that a standard 10 cm-wide edgeline, when compared with no edgelines, improves road safety. However, the analysis also shows that wider edgelines serve to decrease the driving range even further than does a standard edgeline, and this benefit does not cause incursions into the opposite lane.

When considering positional variability, or the amount of vehicle weaving along the roadway, it is apparent that the affect of alcohol is to increase the amount of weaving (Figure 3). This, again, confirms prior research, and similarly, the benefits of standard edgelines compared with no edgelines are confirmed in that a 10-cm wide line reduces variability for both unimpaired and alcohol-impaired drivers. The presence of a wider edgeline serves to decrease variability even further. The maximum reduction occurs in the presence of a 20-cm edgeline. This incremental reduction in variability is only found consistently in the presence of a 20-cm wide edgeline; with a 15-cm (6-in) wide edgeline there are some instances where no reduction in variability occurs.

Earlier studies indicate that road safety is promoted when drivers centralize their position in the driving lane. Analysis of vehicle mean position indicates that alcohol acts as a decentralizer by moving drivers toward the edgeline. However, for both the placebo and dosed drivers, the presence of standard and wider edgelines was found to move the driver away from the edgeline and toward the centerline (Figure 4). However, this movement reaches its maximum when a 15-cm wide edgeline is present. As the edgeline width increases to 20-cm, drivers shift back toward the edgeline, although their final positions were found to be closer to the centerline than they had been in the presence of 10-cm edgelines. The Potters researchers believe that this movement results from drivers being actively aware of the presence of a 20-cm wide line at the left side of a two-lane roadway. As drivers approach the centerline, they are probably responding to the presence of this left side and hence shift back in their lane toward a central position. This effect was apparent for all groups of drivers and indicates the superior benefits derived from a 20-cm wide edgeline for both the alcohol-impaired and the unimpaired drivers, as compared with the lower-incremental benefits of a 15-cm wide edgeline.

Research conducted at Pennsylvania State University indicates that, when drivers perform like one another, the road involved has a high safety potential because drivers are perceiving the road similarly and hence respond similarly. In this study, similarity or grouping of performance was analyzed in terms of individual mean positions. When evaluating these groupings, two effects are apparent (Figure 5). First, the location of the group shifts toward the centerline as edgeline width increases, with the same maximum shift in the presence of a 15-cm wide edgeline as noted when analyzing individual driver mean positions. Second, the size of the group is sharply reduced in the presence of wider lines for both the alcohol-impaired and the unimpaired drivers. In contrast, when standard-width edgelines or no edgelines are present, the size of the group either does not change or, in fact, shows some increase in size due to alcohol impairment. The reduced group size again demonstrates the benefits of wider edgelines.

Finally, the average speed of drivers in the test sections was analyzed by subject grouping. No correlations of speed with either average position or driver variability were found; the only significant conclusion was that, with edgelines of any width, fewer drivers exceeded the posted speed limit than when no edgelines were present.

Figure 1. Driver path range: effect of edgelines.

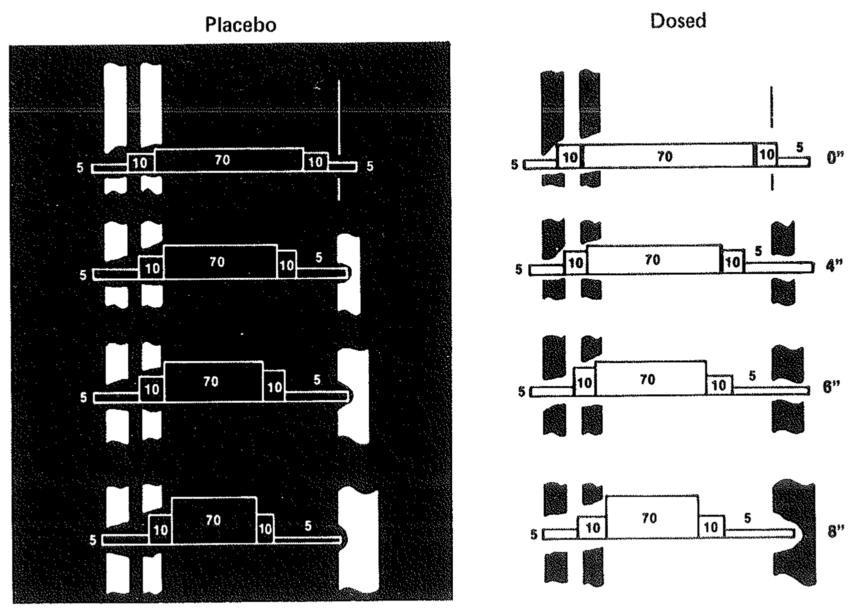
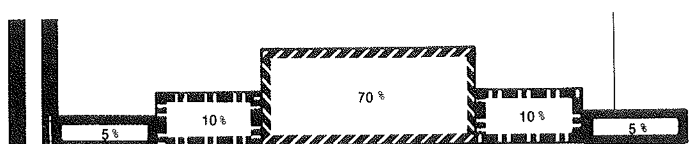


Figure 2. Driver path range: analysis of positional changes at the 80 percent confidence level.



Change In Edgeline Width	Left Extreme 100%	90%	70%	Right Extreme 70%	90%	100%
0" → 4"	N.S.	-1.4	N.S.	-2.0*	-3.6*	N.S.
4" → 6"	-0.9	0.7	0.6	-1.1*	-1.1*	N.S.
4" → 8"	N.S.	N.S.	N.S.	-0.9*	-1.0*	-2.0*

N.S. Not Significant at 80% Level
* Significant Change at 90% Level or Above

Figure 3. Driver variation.

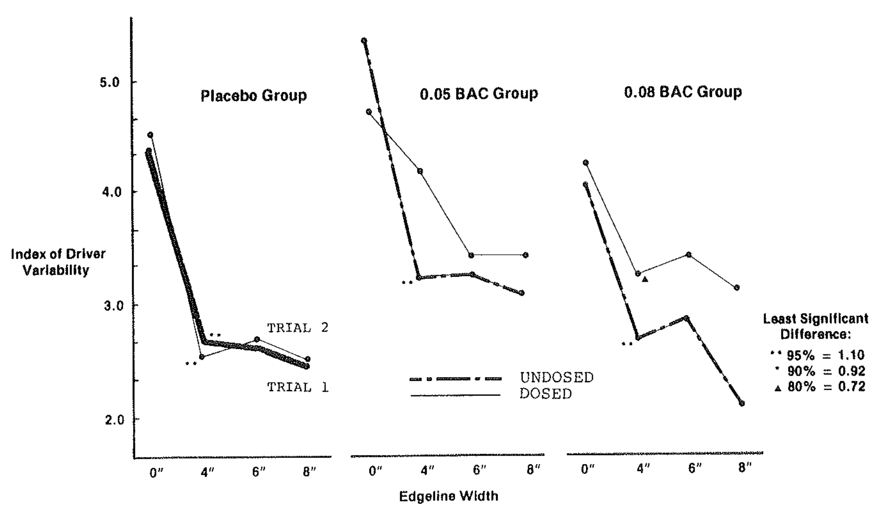


Figure 4. Driver mean position.

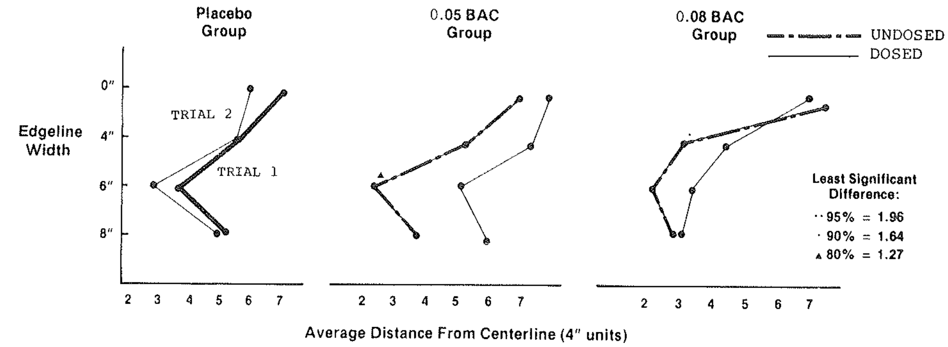
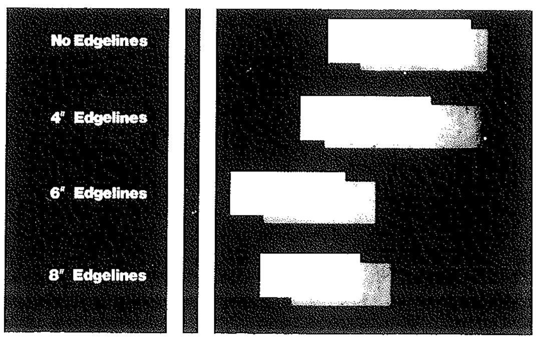


Figure 5. Driver to driver grouping.



Following the conclusion of the Potters positional and speed analysis, FHWA performed their own evaluation of the base data. A measure of good driving was created and analyzed in terms of edgeline width and degree of alcohol impairment. Two conclusions were drawn:

1. When drivers were undosed, the presence of edgelines of any width resulted in more good driving then occurred without edgelines; and
2. When drivers were dosed to the 0.05 or 0.08 BAC level, there was more good driving in the presence of wide edgelines than in the presence of either standard or no edgelines.

CONCLUSION

If further major progress is to be made in reducing the annual toll of fatal highway accidents, then new efforts must be made to eliminate that half of all fatal accidents that involve alcohol. This research examined an engineering approach to what has traditionally been considered a law enforcement, health, and public education problem.

Alcohol impairment is known to have an adverse effect on road safety. The data analysis confirmed that, for all performance measures, there was a detrimental effect on driver performance when alcohol was present. A standard 10-cm (4-in) wide edgeline, when compared with no edgeline, was found to provide significant safety benefits, and this confirmed the results of earlier before-and-after accident studies on the effectiveness of edgelines. Wide edgelines, particularly those 20 cm (8 in) wide, were found to provide incremental benefits as compared with standard-width edgelines, and these benefits were provided for both the alcohol-impaired and the unimpaired driver (see table below).

Driver Performance	Alcohol Effect	Effect of 10-cm Edgeline	Effect of Wide Edgeline
Position			
Range	Increase	Decrease	Decrease further
Variability	Increase	Decrease	Decrease further
Average position	Shift toward edgeline	Shift toward centerline	Further shift toward centerline
Grouping			
Dispersion	Mixed	Mixed	Tighter grouping
Location	Move right	Move left	Move further left
FHWA data	Reduces good driving	Maintains good driving	Maintains more good driving
Overall	Adverse	Beneficial	More beneficial

Alcohol impairment may well relate to other forms of impairment, such as fatigue, the use of drugs, and the reduced visual ability common among older drivers. Hence, the beneficial effects of wider edgelines found for the alcohol-impaired driver may well extend to drivers who have other types of impairment, since the improved driver performance of our test subjects in the presence of wide edgelines indicates that strengthening the visual signal at the road edge may compensate to some degree for impairment and, therefore, reduce the risk of accidents. These results not only corroborate prior research, but also provide new insight into the safety benefits of roadway delineation.

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Causal Factors in Railroad-Highway Grade Crossing Accidents

WILLIAM D. BERG, KARL KNOBLAUCH, AND WAYNE HUCKE

This study examines the contributing factors of rail-highway grade crossing accidents at crossings that have flashing light or crossbuck warning devices. A conceptual model of driver behavior was adapted to the rail-highway grade crossing situation so that a vehicle-train accident could be characterized in terms of the event sequence that led to the collision and the prevailing conditions that were believed to have contributed significantly to the occurrence of the accident. A total of 79 vehicle-train accidents that occurred in North Carolina and Wisconsin were reconstructed and analyzed for patterns of driver error and contributing factors. The findings revealed that, at crossings that have flashers, the credibility of the warning device is a more important problem than its conspicuity. Lack of credibility occurs because of unnecessarily long warning times. At crossings that have crossbucks, driver failure to recognize the presence or approach of a train was the most-common problem. The principal contributing factors were low driver expectancy of a hazard and inadequate quadrant-sight distance. Potential safety countermeasures were identified based on the contributing factor patterns.

Despite extensive research, differences in opinion still remain regarding the major causes of vehicular accidents at rail-highway grade crossings. Factors frequently considered as major contributors to crossing accidents are (a) inadequate signing and signals, (b) lack of credibility and conspicuity of warning devices, (c) driver inattention or risk-taking, and (d) alcohol

The purpose of this study (1) was to identify patterns of contributing factors for vehicle-train accidents by using a case-study approach. The term contributing factor was used in lieu of causal factor to denote a set of prevailing conditions that, when present, can lead to or be associated with a type of accident. A causal factor would denote that the factor was the cause of the accident and once it was present an accident must occur or, conversely, in its absence an accident would not occur. The scope of the research was limited to crossings that have crossbuck or flashing-light warning devices that had recently experienced a vehicle-train accident. Accidents that involved alcohol or a stalled vehicle were excluded. The presence of alcohol was considered to dominate any other contributing factor. Stalled-vehicle accidents were assumed to be due to a vehicle breakdown rather than to a driver-related error.

RESEARCH APPROACH

A driver behavior model was developed for the rail-highway grade crossing situation so that a vehicle-train accident could be characterized in terms of the event sequence that led to the collision and the prevailing conditions that were believed to have contributed significantly to the occurrence of the accident. The assignment of contributing factors for any given accident required that the operational steps in the driving guidance and control process be specified fully. Fundamentally, an accident occurs because a driver is unable to select an appropriate speed and path through a roadway segment or is unable to successfully carry out that decision. Driver error is not the essential consideration; rather, the prevailing conditions interacted to create the opportunity for driver error. These prevailing conditions can encompass the full range of driver, vehicle, and roadway characteristics.

Driver Behavior Model

A useful model for conceptualizing these behavior relations is one formulated by Michaels (2) and shown in Figure 1. The model depicts the operational steps in the driving guidance and control process in the context of a driver-vehicle-roadway system. The blocks labeled sensory detection, perception, analytic operations, decisionmaking, and control response constitute the basic chain of the driving guidance and control process. A breakdown at any one of these tasks can lead to an accident.

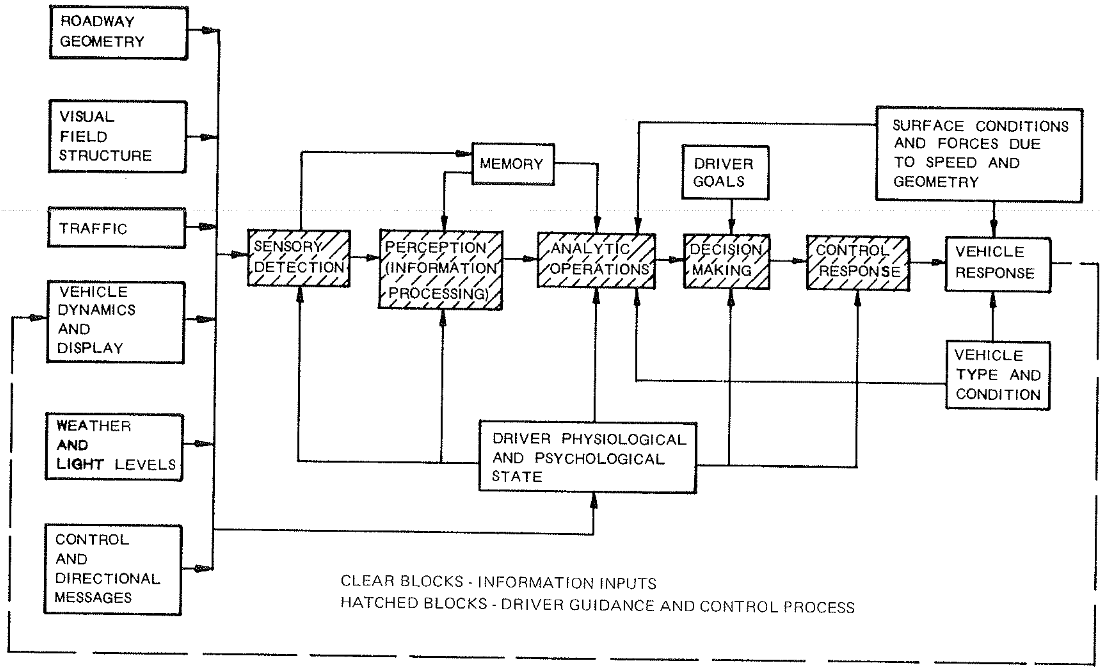
The performance of these tasks is shown to be a function of a variety of information inputs from the driver-vehicle-roadway system. In the context of the rail-highway grade crossing, roadway geometry includes the various design features of the street or highway as well as the crossing itself. Visual field structure refers to the objects, lines, edges, road textures, and contrasts within the driver's visual field. Traffic information includes the velocities and positions of other vehicles, including approaching trains. Information about vehicle response to adjustments in speed and path are transmitted to the driver by means of physical sensations or visual reading of dashboard instruments. Weather and light conditions affect the driving process by altering the available tire-roadway friction as well as the amount of information that can be seen and used for vehicle control.

Traffic control devices, including warning devices at the rail-highway crossing, inform or misinform the driver, depending in part on their conspicuity and credibility. The driver's prior knowledge influences expectancy regarding various rail-highway crossing situations and, therefore, the way in which he or she responds to the hazard presented by the crossing. Vehicle type and condition also influence the response of drivers to hazardous situations. Finally, the driver's own physiological and psychological state will modify the entire guidance and control process.

Possible driver-vehicle-roadway interactions are numerous and complex. If reasonable countermeasures to accidents at grade crossings are to be developed, then the principal interaction patterns that are active in the case of vehicle-train accidents must be identified, categorized, and interpreted in the context of a systematic model of driver behavior. For the purposes of this study, the basic tasks in the driving guidance and control process were aggregated into three elements: recognition, decision, and action.

We hypothesized that the occurrence of a vehicle-train accident was the result of a recognition, decision, or action error. A recognition error was defined as a breakdown in the detection or perception of information necessary to (a) recognize the presence or approach of a train and (b) identify the available actions that would avoid a collision. A decision error was defined as a breakdown in either the analysis of that information or the selection of an appropriate collision-avoidance maneuver. For this type of error, we assumed that the necessary information to perform these tasks has been detected and perceived in sufficient time to make a decision

Figure 1. Operational steps in driver guidance and control.



and complete the maneuver successfully. An action error was defined as the failure to successfully execute what would have been an appropriate collision-avoidance maneuver.

Information-Handling Zones

The evaluation of the possible presence of recognition, decision, or action errors associated with rail-highway crossing accidents required that these basic tasks be considered within the context of a specific set of time-space relations for a vehicle-train encounter. The principles of information-handling zones as defined by positive-guidance concepts (3) were used for this purpose.

The approach to a grade crossing where an accident had occurred was divided into three zones, as illustrated in Figure 2. An area that extended 15 ft on either side of the centerline of the tracks (centerline of outside sets of tracks in the case of a multiple-track crossing) was defined as the hazard zone, or the area in which impact between a vehicle and a train could occur. An area immediately upstream of the hazard zone was defined as the nonrecovery zone. The length of this zone was equivalent to the minimum stopping-sight distance based on the prevailing surface conditions of the roadway and the reconstructed initial approach speed of the vehicle prior to the accident. Located immediately upstream of the nonrecovery zone was the approach zone. Its length was equivalent to the difference between the decision-sight distance (3) for the posted or assumed speed limit and the computed length of the nonrecovery zone.

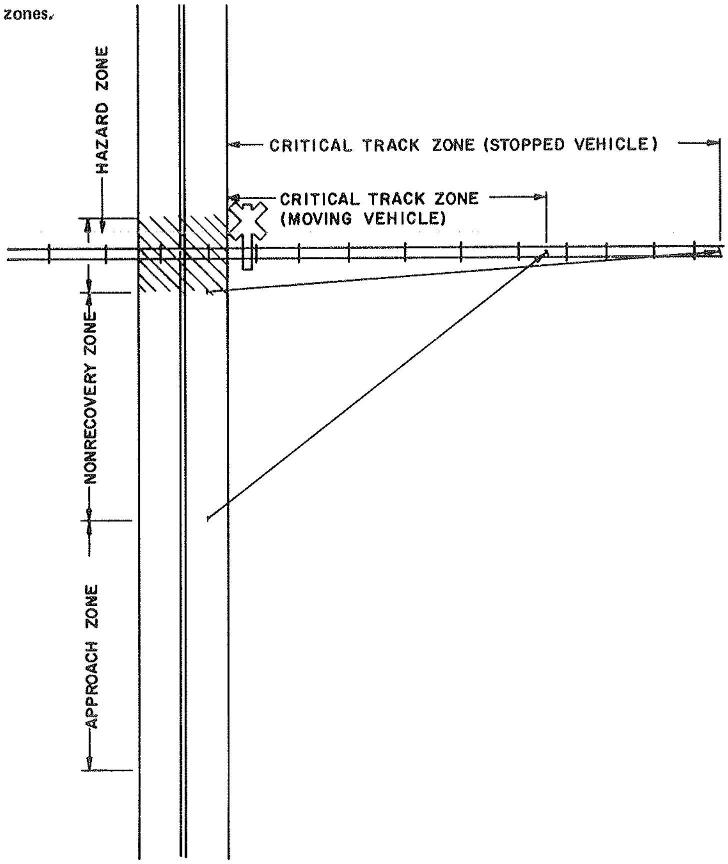
The rail approach on which the involved train was moving was also divided into zones. Two critical zones were defined based on the sight-distance requirements of the involved motorist. The critical track zone for a moving vehicle represented the distance that the involved train would travel during the time required for the subject motor vehicle, approaching at the reconstructed initial speed, to traverse the nonrecovery and hazard zones. The critical moving vehicle track zone, therefore, constituted the minimum desirable quadrant-sight distance under the conditions that prevailed for a given vehicle-train accident. In the context of the previously defined recognition, decision, and action tasks, safe driving performance would require that the driver recognize an activated signal (if present) or the approaching train before he or she entered the nonrecovery zone. If the train had entered the critical track zone by the time the driver reached the beginning of the nonrecovery zone, the appropriate decision would be to stop. If the train had yet to reach the critical zone, the driver could maintain vehicle speed and safely traverse the crossing ahead of the train.

The critical track zone for a vehicle stopped immediately in advance of the hazard zone represented the distance that the involved train would travel during the time required for the motorist to decide to proceed and then to accelerate and traverse the hazard zone. The critical stopped vehicle track zone, therefore, constituted the minimum desirable stop-line sight distance (4) for the conditions associated with a given vehicle-train accident. Safe driving performance would require that a driver who had stopped at the beginning of the hazard zone must recognize whether or not the approaching train was within the critical track zone. If the train is in the critical zone, the appropriate decision would be to wait until the train has passed. If the train has yet to enter the critical zone, the driver could accelerate and traverse the crossing ahead of the train.

Pre-Crash-Event Sequences

The basic recognition, decision, and action steps of the driving guidance and control process were integrated within the information-handling zone framework to produce a set of logic flowcharts that characterize the critical sequence of events that precede a vehicle-train accident. Each unique event sequence was to be examined for predominant patterns of contributing driver-vehicle-roadway factors. These joint patterns of event sequences and con-

Figure 2. Driver information-handling zones.



tributing factors would then serve as the foundation for characterizing the behavorial causes of various types of vehicle-train accidents, the frequency with which these patterns appeared, and potential countermeasures that might be considered.

Figure 3 illustrates the logic flowchart for event sequences and categories of driver error at crossings that have crossbuck warning devices. The chart shows an event sequence that goes from top to bottom. At each recognition, decision, or action point, the alternative paths are identified. The chart, therefore, appears as a tree whose branches terminate with collision between the vehicle and train. Because each path or branch is unique, the driver error that resulted in the accident is identified both by type (recognition, decision, or action), and by a number that references the specific event sequence.

For example, three possible decision errors were defined: D1, D2, and D3. In each case, the driver is believed to have recognized the train from the approach zone. For the D1 and D2 errors, the driver recognizes the train but decides to maintain his or her initial speed and enters the nonrecovery zone. Once within the nonrecovery zone, the driver either decides to attempt to traverse the crossing ahead of the approach train (error D1) or decides to make an emergency stop by placing the vehicle into a skid (error D2). In the case of a D3 error, the driver stops in advance of the hazard zone but decides to traverse the crossing after the approaching train has entered the critical track zone. In each of the above situations, a decision error has been made. The prevailing driver, vehicle, and roadway conditions must then be examined to determine if there is a plausible explanation for the driver's behavior.

In addition to the three types of decision error, Figure 3 illustrates the event sequences for four types of recognition errors and two types of action errors. Figure 4 depicts the similar even sequences and driver errors associated with crossings that have flashing-light warning devices.

DATA COLLECTION

Case-study accidents were selected from Wisconsin and North Carolina because of the completeness and accessibility of police accident reports for 1978 and 1979 vehicle-train accidents. Crossings that had experienced accidents were sorted first by type of warning device (flashing light or crossbuck) and then by county. In Wisconsin, accident sites were only identified for the six contiguous southeastern counties of Milwaukee, Waukesha, Dane, Jefferson, Rock, and Dodge. This selection was made to limit travel time between crossing locations during the field investigations.

The Federal Railroad Administration (FRA) was then contacted to procure the U.S. Department of Transportation-Association of American Railroads crossing inventory information printouts and the FRA rail-highway grade crossing accident-incident reports for each of the accident sites. These reports were subsequently merged with the police accident reports provided by each state. Based on time and resource constraints, approximately 80 accidents were to be selected for reconstruction and causation analysis.

In Wisconsin, a random sample of 22 flashing-light crossings and 14 crossbuck crossings was selected. The 22 flashing-light crossings had 24 accidents during 1978 and 1979. The 14 crossbuck crossings experienced 16 accidents in 1978 and 1979.

In North Carolina, 19 flashing-light crossings

Figure 3. Logic flowchart for driver error at interactions that have crossbuck warning devices.

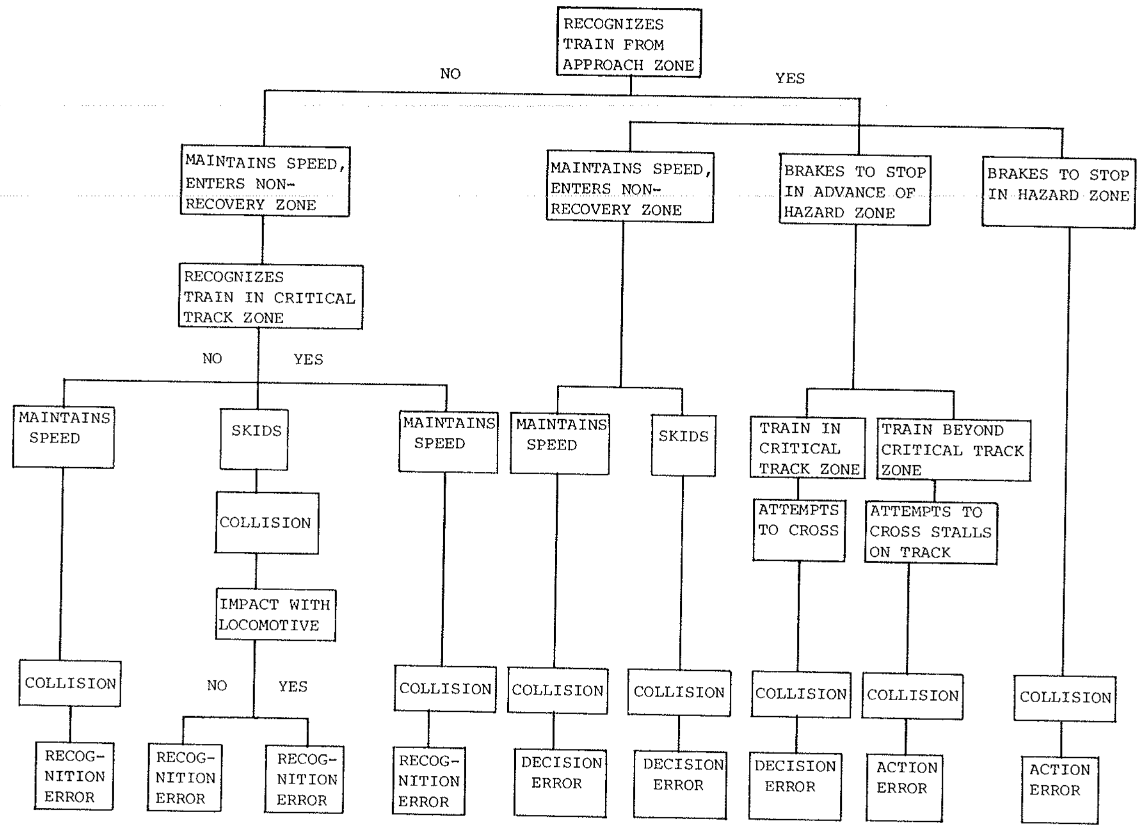
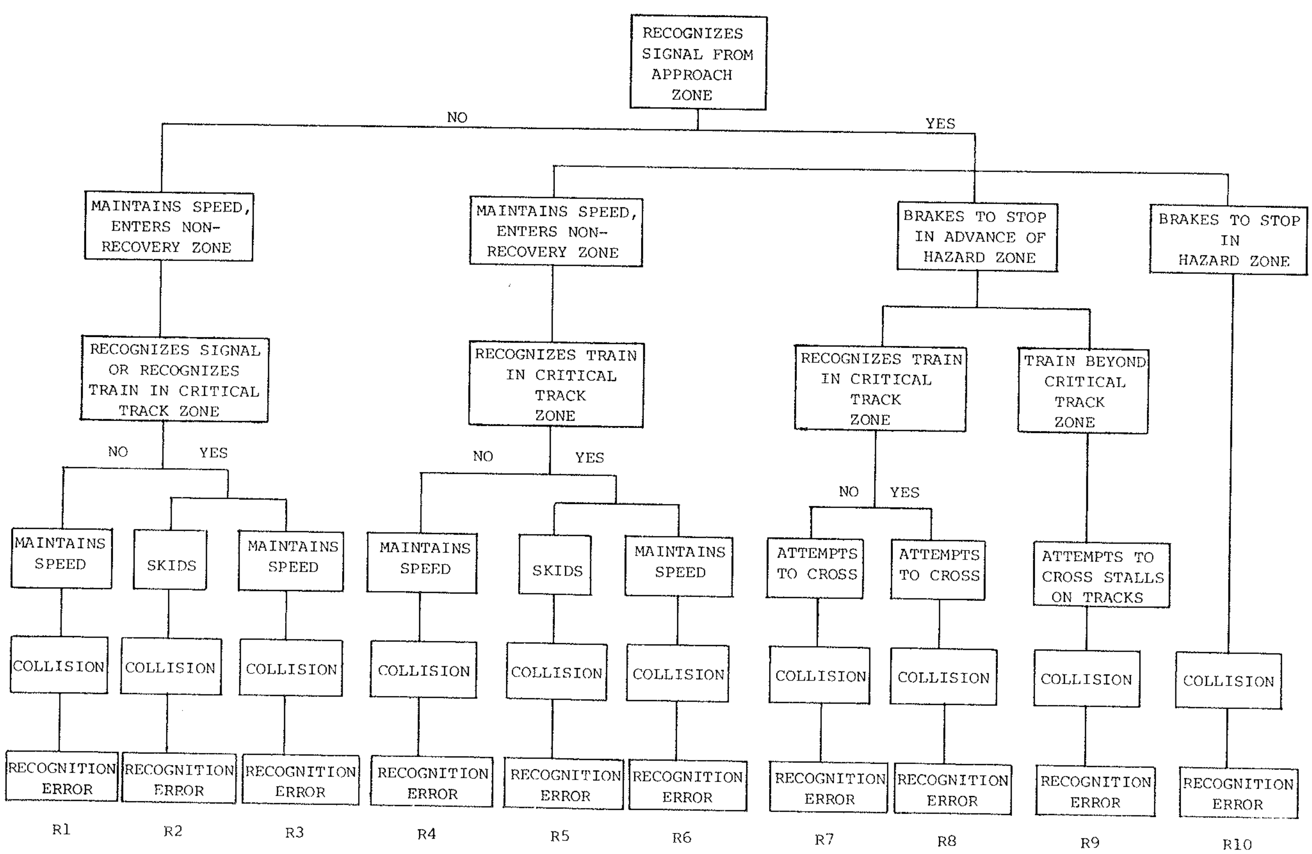


Figure 4. Logic flowchart for driver error at intersections that have flashing-light warning devices.



were chosen on a random basis. A sample of 20 crossbuck sites was then chosen so that these crossings would cluster around the selected flashing-light crossings to minimize travel time during the field studies. Each crossing experienced only one accident during the two-year study period.

Each accident site was then visited to collect data on sight distance, roadway design features, and roadside conditions that might influence driver behavior. In addition, each accident was reconstructed in the context of the event sequence charts shown in Figures 3 and 4. Procedurally, this involved use of skidmark data, reported vehicle and train approach speeds, and the accident report narrative to establish the action taken by the driver and the corresponding positions of the vehicle and train at the critical decision and action points. Notations were also made regarding those factors that might have influenced driver behavior and, therefore, contributed to the cause of the accident.

CONTRIBUTING FACTOR PATTERNS AND COUNTERMEASURES

Each accident was assigned to one of the pre-crash-event sequences shown in Figures 3 and 4. In some cases, an alternative event sequence was defined due to incomplete information about what the driver actually recognized and what decision was made. By using information from the site investigations and accident-reconstruction analyses, the next step was to list the principal factors that were believed to have contributed to the occurrence of each accident. Based on the relative frequency with which various factors were cited, patterns of predominant contributing factors and potential countermeasures were then identified. The results of this process are summarized in Tables 1 and 2 and discussed below. Action errors do not appear because none of the investigated accidents was attributable to this type of error.

Recognition Errors at Crossings That Have Flashers

Of the 43 vehicle-train accidents investigated, 33-44 percent involved some form of driver recognition error, and 53-71 percent were attributed to some form of decision error. The range in frequency is due to uncertainty in assigning type of driver error for some of the accidents.

The information presented in Table 1 reveals that the most-frequent recognition error involves a driver's failure to detect the presence of either the signal or the train. Of the accidents that involved recognition errors, 79-84 percent of the drivers did not detect the signal when they were in the approach zone and had the opportunity to bring their vehicles to a safe stop without placing the vehicle into a skid.

The principal contributing factors for those accidents that involved recognition errors reveal several patterns that have similar potential countermeasures. External distractions that can produce an information overload or divert a motorist's attention from the driving task was a recurring factor. Typical distractions included visual clutter, heavy traffic, adjacent intersections, multiple lanes, rough crossings, and slippery pavement (i.e., wet, snow- or ice-covered). This factor often appeared in combination with the presence of an elderly driver who may have reduced information-processing abilities (error type R1) and with situations in which the signal is obscured from view due to inadequate approach-sight distance or adverse weather conditions (error types R1 and R2). Countermeasures for these types of accidents include increasing signal and possibly train conspicuity. Signal conspicuity might be increased simply by installing additional signals, such as on cantilevers, or by increasing their target value through the use of 12-in roundels or supplementary strobe lights. Although no information was available to determine whether train conspicuity was a problem, the use of roof-mounted strobe lights and high-target-value locomotive paint schemes constitute potentially effective countermeasures.

Recognition error type R4 represents a unique problem where inadequate sight distance is available to drivers of large trucks that have stopped at the crossing. The sight obstruction is created by the combination of an acute crossing angle and a large truck that has restricted visibility to the side and rear. Possible countermeasures to this problem include the use of gates at crossings that have a high percentage of trucks and an acute crossing angle or driver-education materials specifically oriented to truck drivers that would explain precautions that should be taken at this type of grade crossing.

Design Errors at Crossings That Have Flashers

The most-frequent (53-71 percent) type of driver error noted at crossings equipped with flashers was a decision error. In 91-93 percent of these accidents, the driver had recognized the activated signal from the approach zone and either failed to stop or delayed the decision to stop until the vehicle was in the nonrecovery zone. The predominant pattern of contributing factors to these accidents involved extended signal warning time (in excess of 30 s). This factor was usually accompanied by one or more of the following: competing inputs such as multiple tracks and heavy vehicular traffic volume, limited quadrant-sight distance, or inexperienced or elderly drivers.

Warning times in excess of 30 s represented an unnecessarily long advance warning of the approach of a train. Thus, although the activated signal clearly indicated that a train was in the vicinity of the crossing, the train could still be sufficiently far away that it did not constitute an immediate hazard to motorists. The frequently observed pattern of motorists crossing railroad tracks while flashers are operating is believed to be due in large part to extended warning times. Based on the length of the track circuit as measured during the field studies, warning times provided for the accident-involved trains were found to average about 70 s and ranged up to almost 9 min.

The cause of the extended warning times was due primarily to the presence of a low-speed train within a track circuit that had been designed for higher-speed operations. This incompatibility between train speed and track circuit could occur in two ways. First, there might be a wide range in train speeds over the crossing, as could occur near yard areas where switching movements are common. Alternatively, the track circuit may originally have been designed on the basis of high-speed passenger trains that no longer operate over the crossing. Based on the inventory data available for the sample accident sites, the existence of extended warning times was probably a common occurrence at these crossings. This would have an adverse effect on the credibility of the warning device, especially for those motor vehicle operators who tend to drive aggressively and avoid delays or lost time. The presence of heavy traffic flow also creates pressure to maintain speed and avoid stopping.

The frequent presence of limited quadrant-sight distance often effectively delayed the point at which the driver was actually able to observe the train. This tended to compound the extended warning

Table 1. Driver errors, associated contributing factors, and possible countermeasures at crossings that have flashers.

Type of Driver Error[a]	Pre-Crash-Event Sequence	Frequency (%)	Principal Contributing Factors	Possible Countermeasures
R1	Driver does not detect signal or train	12-16	Elderly driver, external distractions, limited quadrant-sight distance	Increase signal and train conspicuity
		7-9	Visibility of signal obscured	Same as R1
R2	Driver recognizes signal or train from nonrecovery zone, attempts to stop	5-7	Visibility of signal obscured due to adverse weather, slippery pavement	Same as R1
R3	Driver recognizes signal or train from nonrecovery zone, does not stop	2-5	Internal distractions; external distractions—multilane highway, heavy traffic, slippery pavement	Same as R1
R4	Driver stops, does not detect train, attempts to cross	7	Limited stop-line sight distance, large vehicle, acute crossing angle, heavy traffic	Install gates, driver education
D1	Driver recognizes signal from approach zone, does not stop, does not detect train	9-14	Extended warning time; competing inputs—multiple tracks, slippery pavement, limited quadrant-sight distance	Relocate beginning of track circuit, provide constant warning time detection, install gates, driver education
		5-7	Inexperienced or elderly driver; competing inputs—heavy traffic, adjacent intersection, multiple tracks; limited quadrant-sight distance	Install gates, driver education
D2	Driver recognizes signal from approach zone, does not stop, recognizes train from nonrecovery zone, attempts to stop	16-19	Extended warning time, limited quadrant-sight distance, truck driver; competing inputs—heavy traffic	Same as D1
D3	Driver recognizes signal from approach zone, does not stop, recognizes train from nonrecovery zone, does not stop	9-14	Extended warning time; competing inputs—low train speed, multiple tracks	Same as D1
		9-12	Extended warning time, inexperienced or elderly driver, limited quadrant-sight distance, competing inputs	Same as D1
D4	Driver recognizes signal from approach zone, brakes to stop, recognizes train, attempts to cross	5	Extended warning time, low train speed, inexperienced driver, limited visibility	Install gates, driver education

[a]See Figure 4.

Table 2. Driver errors, associated contributing factors, and possible countermeasures at crossings that have crossbucks.

Type of Driver Error[a]	Pre-Crash-Event Sequence	Frequency (%)	Principal Contributing Factors	Possible Countermeasures
R1	Driver does not detect train	31-36	Limited quadrant-sight distance, acute crossing angle, low train speed, low expectancy	Remove sight obstructions, additional motorist information, automatic warning device
R2	Train on crossing, driver recognizes train from nonrecovery zone, attempts to stop	19	Limited visibility—darkness, restricted sight distance; inexperienced or elderly driver, slippery pavement, internal distractions	Reflectorization of rolling stock, additional motorist information, crossing illumination
R3	Driver recognizes train from nonrecovery zone, attempts to stop	19	Limited quadrant-sight distance, low expectancy	Same as R1
R4	Driver recognizes train from nonrecovery zone, does not stop	8-11	Limited sight distance, acute crossing angle, darkness	Same as R1
D1	Driver recognizes train from approach zone, does not stop	6-8	Inexperienced or truck driver, slippery pavement	Driver education
D2	Driver recognizes train from approach zone, enters nonrecovery zone, attempts to stop	8	High approach speed, acute crossing angle, low train speed	Driver education
D3	Driver recognizes train from approach zone, brakes to stop, attempts to cross	3	Heavy traffic, multiple lanes, low train speed, limited visibility	Driver education

[a]See Figure 3.

Figure 5. Supplemental advance-warning signs authorized by Minnesota.

time problem because, even though the activated signal indicated that a train was in the vicinity of the crossing, an unnecessarily long warning interval elapsed before the hazard became visible.

Similar countermeasures exist for each of the identified decision errors. The credibility problem created by the extended warning times can be minimized either by adjusting the length of the track circuit to provide an approximately 25-s warning time or by installing constant warning time detection equipment where the range of train operating speeds is large. In some cases, it may be desirable to install gates to remove the driver's option of proceeding across the tracks ahead of the train. Finally, some benefits might be derived over the long run through driver education activities that would emphasize the function and operation of grade-crossing signals and train-detection systems.

Recognition Errors at Crossings That Have Crossbucks

Of the 36 vehicle-train accidents investigated that occurred at crossings that have only standard crossbuck, 77-85 percent involved errors of driver recognition. Of these, 22-25 percent involved late recognition of a train that was already on the crossing.

The predominant contributing factors associated with the recognition of an approaching train was limited quadrant-sight distance due to (a) vegetation, terrain features, or person-made objects or (b) low driver expectancy as reflected by a very low train volume. Low hazard expectancy discourages drivers from actively searching for a train. If a visual search is conducted, the presence of sight obstructions prevents the train from being detected in sufficient time to bring the vehicle to a safe stop.

The principal contributing factor for those accidents that involve a train already on the crossing was limited visibility due to darkness or roadway approaches whose alignment restricted visibility of the crossing from the approach zone. This situation was often compounded by an inexperienced driver, a driver who had reduced visual acuity due to age, or possible internal distractions due to the presence of passengers.

Potential countermeasures for accidents that involve recognition errors associated with an approaching train range from removal of the sight obstructions to the installation of automatic-warning devices. The former is often impractical, and the latter may not be cost effective. An alternative countermeasure would be to provide the motorist with some type of additional information regarding the nature of the hazard and an appropriate approach speed to the crossing. For example, special advance-warning signs (5), as shown in Figure 5, could be used to indicate the presence of limited sight distance, an acute crossing angle, or high-speed trains. These could be supplemented with advisory speed plates that indicate the maximum speed at which adequate quadrant visibility would be available. In some situations, the use of a STOP sign might be warranted (6).

Countermeasures for accidents that involve failure to recognize a train already on the crossing relate principally to improving either the conspicuity of the train or the visibility of the crossing. Reflectorization of railroad rolling stock and illumination of crossings have been studied previously (7,8), and both would appear to offer potentially effective countermeasures for those accidents that occur at night. For those accidents that occur during daylight, the recognition problem was generally created by inadequate visibility of the crossing from the approach zone. Because improvement of the alignment of the roadway is often difficult, special advance-warning signs with advisory speed plates might reduce the likelihood of this type of accident.

Decision Errors at Crossings That Have Crossbucks

Decision errors comprised only 17-19 percent of the accidents investigated that occurred at crossings that have crossbucks. The principal contributing factor patterns were (a) an inexperienced driver or a truck driver traveling on a pavement that is slippery and (b) crossings that have high-volume or high-speed vehicular traffic in combination with low train speeds. Both situations can lead to indecisiveness or risk-taking on the part of the driver due to the desire to avoid unnecessary delay.

Given the typically low train volumes found at this type of crossing and that the train was recognized from the approach zone, the principal countermeasures to this type of accident would appear to be driver-education materials oriented toward better understanding of the hazards posed by grade crossings and the appropriate actions that should be taken once a train has been sighted.

CONCLUSIONS

The results of this study reveal a number of contributing factor patterns associated with driver recognition and decision errors at rail-highway grade crossings. Because of the relatively small sample of vehicle-train accidents that were reconstructed, it is difficult to generalize on the frequency with which any given combination of driver error and principal contributing factors would occur on a nationwide basis. Nevertheless, the results are useful in explaining a behavioral basis for accidents at grade crossings and in identifying potential countermeasures.

Regarding vehicle-train accidents at crossings equipped with flashing-light signals, the study results suggest that the credibility of warning devices is a more-important problem than conspicuity. This is based on the approximately two-to-one ratio in driver decision-to-recognition errors. Lack of credibility occurred principally due to unnecessarily long warning times prior to the actual arrival of the train. Potential countermeasures to the credibility problem include reduction in the length of track circuits to ensure compatibility with existing train operations and installation of constant-warning-time devices where the range in train speeds is large.

For grade crossings that have crossbucks as the single warning device, the study findings revealed that approximately 80 percent of the investigated accidents at these crossings involve driver-recognition errors. The principal contributing factors were lack of quadrant-sight distance and low driver expectancy of the presence of a train. These factors create a behavior pattern in which drivers tend not to look for a train. When an active visual search is made, the train is often hidden from view at the last point where the driver must make the decision to stop. Because removal of sight obstructions and installation of automatic-warning devices are often found to be impractical or not cost effective, a reasonable countermeasure would be to provide the motorist with more-complete information about the nature of the hazard and an appropriate safe approach speed. This could be accomplished with the use of special-message advance-warning signs coupled with speed advisory plates.

Finally, the results of the study indicate that driver-education activities (such as Operation-Lifesaver) should offer an important contribution to the safety problem at grade crossings by making motorists more aware of hazards at grade crossings and how to respond to them. This includes an understanding of the function and operation of the various types of warning devices.

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Pedestrian Cross Flows in Corridors

C. J. KHISTY

An investigation into the nature of pedestrian cross flows in corridors at right angles to one another is described. This study was undertaken by using time-lapse photography to determine the effect of a minor pedestrian flow crossing a major pedestrian flow. Such cross flows of pedestrians are common in major activity centers and in special event transportation systems, such as universities, bus stations, art galleries, museums, and places of entertainment. The results of this study were compared with those obtained from theoretical gap and collision analysis. The comparisons were found to match closely. A design criterion for facilities where cross flows of pedestrians occur is developed based on the data gathered from the films and the theoretical analysis.

This paper describes a study undertaken at Washington State University to examine the characteristics of pedestrian cross flows in corridors, passageways, and hallways and to determine the effect of one pedestrian flow crossing another. Statistical analysis was used to explain these characteristics and to establish a design criterion for facilities where such cross flows of pedestrians occur. Flow characteristics of pedestrians in single channels have been studied and documented by several researchers (1-4). However, investigations into the nature and characteristics of pedestrian cross flows is very limited (5).

Pedestrian crossing movements in this study were observed by using time-lapse photography. Speed-density-flow relations were established from data derived from films. Pedestrian conflicts at cross flows were also observed and analyzed from films. Subsequently, a theoretical gap analysis was used to verify the experimental work. A design criterion based on this investigation is suggested for pedestrian facilities.

PEDESTRIAN CROSS FLOWS

Cross flows of pedestrians are ubiquitous. Corridors, passages, and hallways in schools, booking offices, cinema theaters, art galleries, museums, and places of entertainment are instances where such cross flows are commonly observed. Where pedestrian densities are low, cross flows of pedestrians seldom create problems; but when the pedestrian densities in one or both streams are heavy, the probability of conflicts is high.

Corridors, for instance, dominate the space configuration in buildings. When two corridors cross one another, their users have to use a common area, similar to an uncontrolled highway intersection. Corridors in school and college buildings serve to circulate their users when class schedules require movement of students and faculty on an hourly basis. Conflicts of two pedestrian streams at the junction of two corridors are all too common in such situations. When corridor widths are narrow and the pedestrian concentrations are high in both streams, pedestrian walking speeds, particularly in the minor flow, come to a standstill and queues build up. In the major flow there is evidence of extremely restricted walking speeds, shuffling, and frequent conflicts. One of the reasons for this condition is that corridor widths are usually determined by building codes rather than with respect to pedestrian traffic demand. Even those guidelines and design criteria currently used for corridor design do not take cognizance of cross-flow con-

Figure 1. Location map.

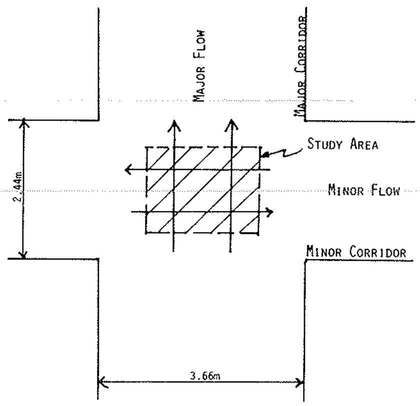


Figure 2. Density and speed relation of major flow.

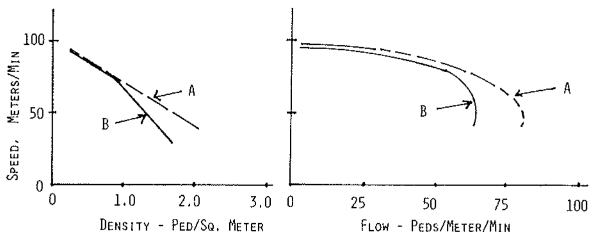
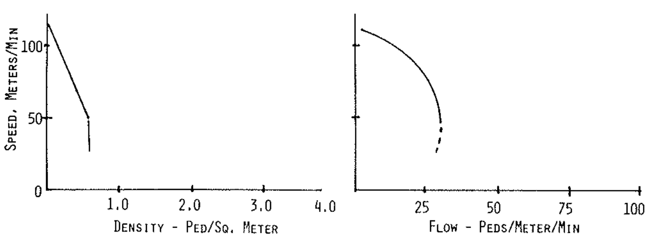


Figure 3. Density and speed relation of minor flow.

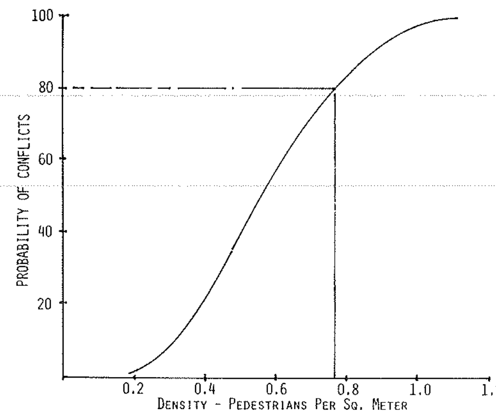


flicts between pedestrians. Generally, two corridors cross one another, where one of the corridors serves the main stream and the other the minor stream. As densities in the main stream rise higher and higher, there is a corresponding increase in conflicts. These conflicts are obviously a function of walking speeds and pedestrian spacing in the main stream. For the purpose of this investigation a conflict is defined as any stopping and shuffling or breaking of the normal walking pace due to a close confrontation with another pedestrian. Such confrontation naturally requires immediate adjustment in speed and direction to avoid collisions (1).

LOCATION AND TECHNIQUE

Although the campus buildings of Washington State University abound with locations where moderate-to-heavy cross flows of pedestrians occur, a location that was suitable to photograph was not easy to find. Basically, a fairly simple location was desired, where two pedestrian flows cross preferably at right angles and where natural boundaries to each flow prevent people from spreading out all over. We desired that each individual flow be predominantly in one direction (in contrast to being two-way). A location that matches these preferences was found

Figure 4. Cross-flow traffic conflicts.



where the pedestrian flow in the corridor crossing had a large variation, and this afforded an opportunity to make observations in a variety of flow conditions. The main corridor is 3.66 m wide and the minor corridor is 2.44 m. The area considered for analysis was 2x1.5 m, and this was clearly marked on the floor of the two intersecting corridors. The number of pedestrians crossing this demarcated area and their speeds was calculated based on this area. This avoided the problem of contending with the edge effect (see Figure 1).

For 3-5 min every hour there is a heavy surge of pedestrians (95 percent students) in the major flow, predominantly in the same direction. A similar flow of somewhat weaker proportion attempts to cross the main flow at about the same time. Time-lapse photography was used in filming this location at a speed of 18 frames/s. Although this speed was not necessary for this study, the equipment available necessitated use of this speed. Filming was done on two different days: Thursday, November 6, and Tuesday, December 2, 1980, during the 12:00 noon and 1:00 p.m breaks. Data were gathered on two rolls of film--super 8, each 15 m long. The analysis of the film was done by using a hand-operated editing machine. All timing measurements were initially made in frames and subsequently converted to real time.

Data

The films yielded two sets of data: first, the flow (q)-density (k)-speed (v) information for the major and minor streams of pedestrians who cross the study area, and second, the number of conflicts observed between pedestrians at different densities.

Flow-Density-Speed (q-k-v) Relations

The q-k-v relations were established for two conditions. In the first case the major flow was observed just before it intersected the minor flow. This is shown as curve A in Figure 2. In the second case, the major flow was observed with the minor flow and the corresponding curve is marked B. Figure 3 shows the q-k-v characteristics for the minor flow crossing the major flow.

Conflict Study

The number of collisions or near collisions were counted when the major and minor streams were crossing at various densities. Figure 4 shows the relation between pedestrian densities and the percentage of collisions that occurred in the study area.

Analysis and Discussion

An examination of curves A and B in Figure 2 indicates no significant change in the major flow pattern because of the minor flow up to a pedestrian density of about 0.8-1.0 pedestrians/m². Beyond this density the difference between curves A and B for flow and speed increases progressively. The curves shown in Figure 3 for the minor flow appear to confirm that the minor flow suffers when the density reaches 0.7-0.8 pedestrians/m². Notice that speeds for the minor flow are slightly higher than speeds for the major flow when the densities are low. This is probably because pedestrians in the minor flow have to be more aggressive in crossing the major flow.

An important observation made from the films was that the speed and density of the major flow were more or less independent of the minor flow. In fact, the minor flow was heavily dependent on the characteristics of the major flow. Evident, for example, is that pedestrians in the minor flow waited outside the major flow until a large enough gap appeared, and then accelerated their pace to get across. The streams hardly mixed, except for a very small number of pedestrians who turned right or left. Also, the major flow appeared to be homogeneous and continuous as opposed to the minor flow, which was in some cases discontinuous or stationary.

The films also brought out that, when the density in the major flow reached levels beyond about 1.0 pedestrians/m², the minor flow reduced drastically and queues built up. In fact, one found high densities building up in both the major and minor streams simultaneously.

Conflict Study

The analysis of pedestrian conflicts shown in Figure 4 connects the density of pedestrians in the study area with the percentage of conflicting pedestrians. In a sense the curve supports that, at density of about 0.8 pedestrians/m², the conflicts are 80 percent. A close examination of the films revealed that restricted passage of the minor flow through the main flow could be accomplished at densities even higher than 0.8 pedestrians/m² but pedestrians, in such cases, invariably turned themselves sideways to expose the minimum profile position in order to pass.

Gap Analysis

In order to verify the conclusions drawn from the films, an analysis of cross flows by using gap analysis was used. If two pedestrian streams, one major and one minor, cross one another, what is the flow in the main stream that will make it uncomfortable for pedestrians in the minor stream to cross?

The analysis assumed that pedestrians crossing the minor stream constitute a Poisson process, and that the width of the main stream is 1 m. Also assumed was that the speed of pedestrians that cross the main stream is 1.22 m/s. The number of t-second intervals in an hour is 3600/t, whereas in an interval of t seconds, the probability of no pedestrians passing through the area is e^{-vt}, where v is the pedestrian speed. Based on these assumptions, the number of pedestrians who can cross the main stream was tabulated and the results are given in the following table:

Capacity of Pedestrian Cross Flows [(pedestrians/min)/m]

Major Flow Width, A	Minor Flow Width, B	Total Capacity (pedestrian/min)
30	30	60
40	28	68
50	25	75
60	22	82
70	19	89
80	17	97

Recognize that these results indicate, in effect, some capacity values of pedestrians in a minor stream to cross a major pedestrian stream based on queuing theory (7). The major and minor cross flows shown in the table match the experimental results fairly closely. When the values of the permissible flows in the major and minor streams shown in the table are added, it appears obvious that this figure could be used for practical design purposes.

Design Criterion

When this study was undertaken, one of the important objectives was to gather sufficient data about the character and nature of two pedestrian cross flows to suggest an acceptable design criterion. Design and performance criteria are the preferred means of control where they can be administered capably.

Based on the discussions and also on the general objectives of providing continuity, convenience, and comfort (1), the following values are suggested:

1. Minimum speed of 60 m/min,
2. Maximum flow of 75 pedestrians/min/m for the total of the two flow rates, and
3. Maximum density of 0.8 pedestrians/m².

This suggestion compares well with that recommended by Weston and Marshall (5).

SUMMARY AND CONCLUSION

The investigation described in this paper determines the characteristics of pedestrian cross flows in corridors. The data and analysis from this study were compared with results derived from a theoretical gap analysis. A design criterion suggested is to limit the maximum density in such cross flows to 0.8 pedestrians/m².

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Portable Intersection to Accelerate Travel Training of Mentally Handicapped Children

LOUIS J. PIGNATARO AND JOSE ULERIO

This paper describes the design and construction of a portable intersection system to facilitate the travel training of severely and moderately mentally handicapped children. It also describes the procedures used to evaluate the effectiveness of the portable intersection as a teaching tool. The test results showed a statistically significant improvement in test scores with the intersection trainer, both relative to pretraining scores and posttraining scores of a control group trained in the conventional way.

Most of us cross an intersection several times a day without thinking about it, but for thousands of mentally handicapped children this is a hazardous and sometimes impossible task to accomplish. Many handicapped children have learned to board a bus or train and reach their destination but cannot travel independently because of an inability to cross streets safely with consistency. Flashing DON'T WALK signs, traffic lights, street noise, and moving vehicles often confuse mentally handicapped children and create a potentially hazardous condition not only for the child but also for the motorist.

The New York City Board of Education's travel training program teaches handicapped students to travel independently on the public transportation system. The program is designed to provide individualized instruction in transportation skills to handicapped students. Included in this instructional program are the following skills:

1. Safe crossing of streets,
2. Identification and boarding of the correct bus or subway,
3. Exiting of bus or subway at the correct stop,
4. Obtaining assistance when necessary (i.e., lost situations), and
5. Appropriate behavior.

Travel Training is a citywide program that offers instructional services to handicapped students who attend special classes in schools throughout the city. Handicapped students who receive these services are taken out of their classrooms by specially trained travel trainers and return to their classrooms after completing the instructional program. Frequently these students have had little or no experience or specific instruction in travel or travel-related skills prior to entering the program. Therefore, the Travel Training staff uses considerable instructional time in teaching basic prerequisite travel skills, such as street crossing. Over a 10-year period, 85 percent of the handicapped students who participated in the Travel Training program have successfully achieved independent travel. As a result of the program's success, the New York State Education Department validated the program in 1976 and granted funds for the program to assist school districts throughout the state to replicate the program. The inability to cross streets safely with consistency is a major factor in the failure of those students who are not successful in travel training. The specialized nature of the program does not provide sufficient time for any of these students to acquire these basic skills. The present method of teaching street-crossing skills in the classrooms allows for little exposure to an actual intersection for safety reasons, especially when the students are young children or severely handicapped.

A method of exposing the child to a real intersection had to be developed if a child was to behave in a rational manner when approaching an intersection. He or she would have to know that when the signal changed to green he or she had the chance to cross the street. He or she would have to know that when he or she approached an intersection that did not contain a traffic signal, he or she would have to look in both directions before crossing when no cars were coming. The best way to accomplish this type of training was to build an intersection that could be placed inside a classroom where the training could be done by the teacher or an instructor. The intersection had to be relatively easy to assemble and disassemble, compact, and, most importantly, portable.

The Transportation Training and Research Center at the Polytechnic Institute of New York undertook the effort of designing and constructing a portable intersection to facilitate the travel training of severely and moderately mentally handicapped students.

The project's aim was to design, build, and test a small portable intersection that could be assembled and taken apart with relative ease and that could be stored within a relatively small area.

The project was divided into three phases:

Phase 1: Development of preliminary construction plans and construction of prototype model,
Phase 2: Construction of portable intersection, and
Phase 3: Testing and evaluation of the portable intersection.

The intersection system is made up of fiberglass modules that simulate sidewalks, traffic signals, and pedestrian crosswalks; traffic signs; a miniature car and bicycle; barricades to simulate construction areas; a tape recording of traffic noise at an intersection; and a video recording for training instructors and students on how to set up the intersection and training procedures. A prototype model was designed and built (see Figure 1) to aid in the development of the assembly procedures and for use by the training instructors.

The program for the evaluation of the methods and procedures used to assess the effectiveness of the portable intersection as an instructional tool was developed by the staff of the New York City Board of Education's Travel Training for the Handicapped Program. The lesson plan, the construction drawing, and other materials are available to interested parties.

Figure 1. Prototype model used in development of assembly procedures and also used by training instructors.

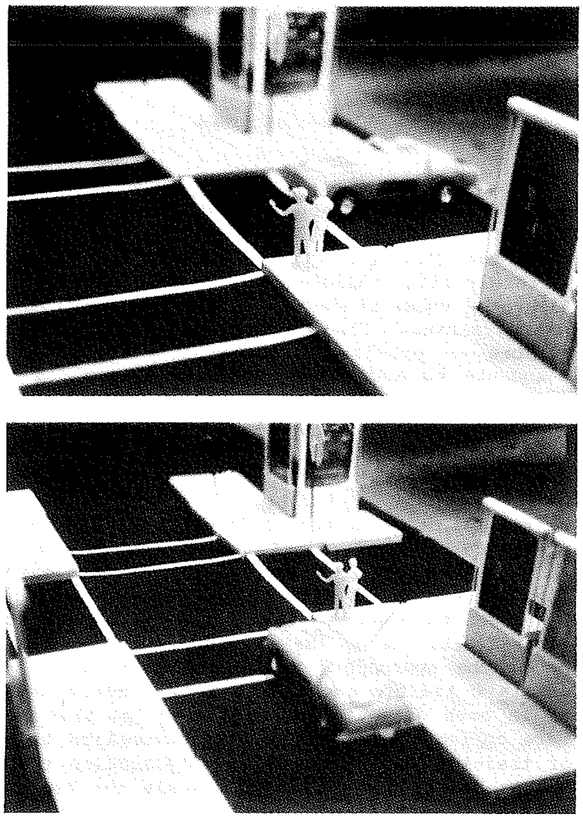
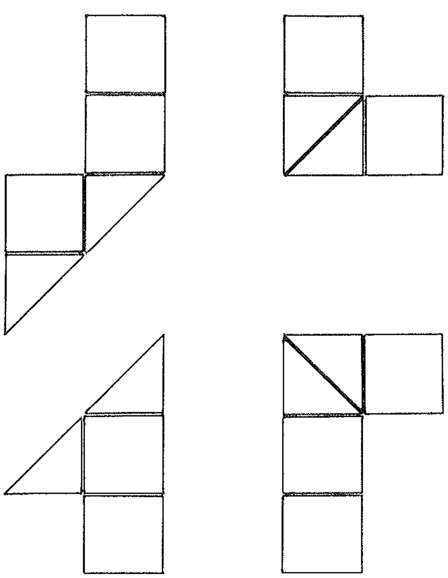


Figure 2. Possible ways to arrange modules to simulate different intersection configurations.



CONSTRUCTION OF PORTABLE INTERSECTION

The portable intersection consists of the following components, which attempt to simulate real-life conditions.

The sidewalks are simulated with twelve 4.5-ft square modules and eight 4.5-ft right triangular modules made of reinforced fiberglass. The fiberglass modules can be put together to resemble a number of different crossing zones and street corners. Figure 2 shows possible ways of arranging the modules to create different configurations at an intersection. Fiberglass was chosen because of its high strength, durability, and weather resistance. The portable intersection is assembled by a crew of mentally handicapped students under the supervision of a school instructor. It can be easily assembled in approximately 25-30 min in any multipurpose room or gymnasium that occupies an area of approximately 50x50 ft. The assembly of the intersection by the students is used as a workshop class in which the students learn to put the modules together.

Two miniature traffic signals and pedestrian crossing signals are provided to simulate real traffic signals that the student might encounter on a trip to or from school. The signals can operate automatically on a 30-s cycle length and also can be operated manually.

A miniature car and bicycle are provided to simulate moving traffic within the intersection and to educate the students about the dangers of a moving vehicle.

Various traffic signs that the students might encounter on a trip to school are provided so that they may be trained to respond to the different traffic situations that correspond to the different signs. Figure 3 shows the various signs used in the trainer.

Two A-frame barricades are provided to simulate construction areas. The students are instructed about the dangers involved in and around construction areas that might alter their behavior at an intersection.

Temporary marking tape is provided to create the pedestrian crosswalk where the students are taught to cross. Backdrops, 3.5x3.5 ft, made of plywood covered with simulated brick-vinyl adhesive paper were provided to simulate walls of buildings at the intersecting sidewalks.

A portable tape recorder and a recording of traffic noise at an intersection is provided to familiarize the students with the noise at an intersection.

A video-cassette recording was made for the training of instructors and students on how to assemble the modules. It also contains information that can be used for the training of the students in crossing streets. A prototype (scale) model was designed and built to aid in the development of placement of traffic control devices, assembling and disassembling procedures, intersection configurations, and a lesson plan.

Figure 4 shows the assembled intersection with some of its components as it appeared in a multipurpose room at the Polytechnic Institute of New York.

EVALUATION DESIGN

The portable intersection was field-tested at a school for severely and moderately mentally retarded

Figure 3. Various traffic signs that can be used to represent those encountered on city streets.

Figure 4. Assembled intersection with its various components.

adolescents, the Adult Skills Training Center in Brooklyn.

The evaluation phase was a comparison of a pretest and posttest of selected students based on their level of street crossing skills before and after a given instructional period. Students were divided into two groups--a control group and an experimental group. The experimental group received instruction in street crossing by using the portable intersection; the control group continued to receive instruction in street crossing with the method currently used by the Board of Education, which includes color recognition, word recognition, and class trips.

The New York City Board of Education's travel training program developed a test to evaluate street-crossing skills. The street crossing and related skills test assesses the following:

1. Student's knowledge of skills related to street crossing through picture identification (i.e., colors and words) and

2. Student's ability to cross streets under specific conditions (i.e., no traffic signals, traffic signals, and obstructions in street).

Forty-four severely to moderately mentally retarded students were tested on the street crossing and related skills test. Each student was tested individually. The testing procedure was carried out in the following manner.

Each student was brought into a room and asked to participate in a special project. The student was presented with a set of three pictures and asked to select the picture that identified a particular situation. After the selection was made, the three pictures were removed and another set of three pictures was presented. A total of six sets was used to assess each student's knowledge of colors, WALK-DON'T WALK signs, one-way streets, two-way streets, police, and use of a crosswalk.

Each student was taken outside the building to the streets adjacent to the school. Students were told to cross city streets that represented four street situations: no traffic signals, red signal at corner, green signal at corner, and double-parked car obstructing the crosswalk. Because there are no traffic signals in the immediate vicinity of the Adult Skills Training Center, the signals from the portable intersection were set up at two street corners, and a car was double-parked at the crosswalk of the third corner. A Travel Training staff person stood near each of the four corners to ensure the students' safety.

Each student was sent around the square individually and given the direction, "Cross the street when it is safe" as he or she approached the street. The first street crossing had no traffic signal; the second street's crosswalk was obstructed by a double-parked car; the third crosswalk had a red signal indication; and the fourth crosswalk had a green signal indication. The use of the portable intersection permitted control of the traffic signals to conform to the test situation.

Characteristics of Sample Population

The 44 students were all between the ages of 19-20. Their intelligence quotients (IQs) were below 50. The students were divided equally into an experimental and control group. The groups were matched as closely as possible in the following areas: IQ, language, sex, living situation, and pretest score on the street-crossing test.

In the control group the range of IQ scores was from 24 to 46, with a mean of 36.4. Of the 22 students, 21 spoke English and 1 spoke Spanish; 2 lived in a state developmental center, 3 lived in group homes for the mentally retarded, and 17 lived at home; 9 were female and 13 were male; the range of the pretest scores on the street-crossing test was from 2.3 to 19.7, with a mean score of 7.89. The maximum attainable score on the pretest was 23. These characteristics are shown in Table 1.

In the experimental group, the range of IQ scores was from 29 to 47 with a mean IQ of 36.6. Of the 22 students, 19 spoke English, 1 used sign language, 1 spoke Turkish, and 1 spoke Spanish; 4 lived in a state developmental center, 4 lived in group homes for the mentally retarded, and 14 lived at home; 10 were female and 12 were male. The range of the pretest scores on the street-crossing test was from 1.7 to 20.0, with a mean score of 8.59. These characteristics are shown in Table 2.

Instructional Program

After the pretesting was completed and the students

Table 1. Characteristics of students in control group.

Sex	IQ	Living Situation	Language	Score Pretest	Score Posttest
F	40	Home	Spanish	10.2	8.7
F	30	Home	English	4.0	4.8
F	Unknown	Home	English	4.3	8.7
F	32	Group home	English	2.3	0.7
F	36	Home	English	19.7	20.0
F	Unknown	Group home	English	6.0	5.5
F	46	Home	English	9.0	12.0
F	28	Home	English	7.0	12.0
F	Unknown	Home	English	8.2	3.2
M	40	Home	English	2.5	1.8
M	Unknown	Home	English	6.0	3.5
M	24	Developmental center	English	5.5	6.5
M	Unknown	Home	English	17.0	22.0
M	Unknown	Home	English	6.0	5.0
M	Unknown	Home	English	11.0	10.5
M	Unknown	Home	English	10.0	8.0
M	36	Home	English	4.3	2.7
M	40	Home	English	4.7	2.3
M	Unknown	Home	English	9.3	2.8
M	40	Developmental center	English	6.3	18.7
M	Unknown	Home	English	9.2	11.8
M	45	Group home	English	11.0	5.2

Table 2. Characteristics of students in experimental group.

Sex	IQ	Living Situation	Language	Score Pretest	Score Posttest
F	30	Group home	English	8.0	7.0
F	Unknown	Home	English	9.0	14.0
F	28	Home	English	11.5	23.0
F	33	Home	English	3.0	14.0
F	40	Home	English	1.7	4.3
F	Unknown	Home	English	5.0	9.0
F	47	Group home	English	16.0	23.0
F	Unknown	Home	English	15.0	23.0
F	Unknown	Home	English	5.0	14.0
F	39	Home	English	15.0	22.0
M	42	Developmental center	English	10.0	8.0
M	30	Group home	English	4.5	16.0
M	40	Developmental center	Sign language	2.0	9.0
M	Unknown	Home	English	7.0	8.5
M	47	Developmental center	English	10.0	11.0
M	Unknown	Home	English	12.0	17.5
M	40	Home	Spanish	20.0	20.0
M	30	Home	Turkish	5.7	3.3
M	40	Home	English	13.3	20.7
M	34	Developmental center	English	6.0	2.0
M	29	Group home	English	3.7	7.0
M	Unknown	Home	English	5.5	10.0

were divided into the control and experimental groups, instruction on the portable intersection was initiated for the experimental group. A travel training paraprofessional developed the instructional model, which was started on December 3, 1980, and completed on February 6, 1981. Due to school holidays and personnel absences, instruction lasted for a six-week period.

The portable intersection was assembled each morning by a crew of mentally retarded students. The only space available was the combination lunchroom and gymnasium. The space was shared with the physical education instructor who conducted small group activities in half of the room while the portable intersection activities were going on in the other half. The hours available for instruction were 9:30 to 11:00 a.m. because the space was needed for lunch services at 11:20 a.m.

The travel trainer brought a group of four students to the room at a time and conducted individual and small group training activities. The length of the instructional period varied according to the student's needs and abilities, ranging from 1 h to 10.25 h of instruction over the six-week period. Eight students received less than 3 h of instruction. Of these, four achieved skill mastery on the intersection, three presented behavioral problems or refused to participate, and the fourth was absent frequently. The remaining 14 students received a minimum of 3 h of instruction each, spread out over the instructional period. These students received approximately 0.5 h of instruction one to three days per week over the six-week period. Specific street-crossing skills were taught, including stopping at corners, using the crosswalk, responding appropriately to traffic signals, and looking for and responding to cars. The 22 students in the control group continued to receive instruction in traffic safety and street crossing from their classroom teachers in the usual manner.

At the completion of the six-week instructional period, the 44 students were again tested outside by using the street-crossing and related skills test. The format and procedures for the posttest were identical to those for the pretest.

Statistical Procedure

The t-test was selected to compare the pretest and posttest scores for the control and the experimental groups.

Pretest Scores	
Control Group	Experimental Group
19.7	20.0
17.0	16.0
11.0	15.0
11.0	15.0
10.2	13.3
10.0	12.0
9.3	11.5
9.2	10.0
9.0	10.0
8.2	9.0
7.0	8.0
6.3	7.0
6.0	6.0
6.0	5.5
6.0	5.7
5.5	5.0
4.7	5.0
4.3	4.5
4.3	3.7
4.0	3.0
2.5	2.0
2.3	1.7

In the above table, because there is a computed difference between the means of the pretest scores for the control group (7.89) and for the experimental group (8.59), the t for testing the difference between uncorrelated means in two samples of equal size was used. The computed value of t is 0.496, and there were 21 degrees of freedom. At the $\alpha = 0.01$ level of confidence, the tabulated t = 2.831 for a two-sided test, which is appropriate. Therefore, the t-score of 0.496 shows that there is no detectable difference between the pretest scores of the control and experimental groups. The standard deviation is 4.30 for the control group and 5.02 for the experimental group.

To determine the significance of the difference between the pretest and posttest scores for the

Table 3. Comparison between pretest and posttest scores.

Control Group			Experimental Group		
Before (X)	After (Y)	Y-X[a]	Before (X)	After (Y)	Y-X[b]
19.7	20.0	0.3	20.0	20.0	0.0
17.0	22.0	5.0	16.0	23.0	7.0
11.0	10.5	-0.5	15.0	23.0	8.0
11.0	5.0	-6.0	15.0	22.0	7.0
10.2	8.7	-1.5	13.3	20.7	7.4
10.0	8.0	-2.0	12.0	17.5	5.5
9.3	2.8	-6.5	11.5	23.0	11.5
9.2	11.8	2.6	10.0	8.0	-2.0
9.0	12.0	3.0	10.0	11.0	1.0
8.2	13.2	-5.0	9.0	14.0	5.0
7.0	12.0	5.0	8.0	7.0	-1.0
6.3	18.7	12.4	7.0	8.5	1.5
6.0	5.0	-1.0	6.0	2.0	-4.0
6.0	3.5	-2.5	5.5	10.0	4.5
6.0	5.5	-0.5	5.7	3.3	-2.4
5.5	6.5	1.0	5.0	14.0	9.0
4.7	2.2	-2.5	5.0	9.0	4.0
4.3	2.7	-1.6	4.5	16.0	11.5
4.3	8.7	4.4	3.7	7.0	3.3
4.0	4.8	-0.8	3.0	14.0	11.0
2.5	1.8	-0.7	2.0	9.0	7.0
2.3	0.7	-1.6	1.0	4.3	2.7

Note: For the control group, the standard deviation of X is 4.30, of Y is 6.05, and of Y-X is 4.19. For the experimental group, the standard deviation of X is 5.02, of Y is 6.81, and of Y-X is 4.58.
[a]For paired t-test, t = 0.132.
[b]For paired t-test, t = 4.53.

control and experimental groups, the t-formula for testing the difference between correlated pairs of means was used by using a one-sided test because one would expect a positive improvement in each case (see Table 3). Further, a level of confidence of α = 0.01 was used in each case, to ensure greater power to the result if the hypothesis of no difference were rejected. (The procedure was taking only 1 in 100 chance of rejecting a true hypothesis of no difference.) The results may be summarized as follows.

Control Group

The mean for the control group's pretest was 7.89 and for the posttest was 8.00. The difference in mean scores was 0.12. At the α = 0.01 level of confidence, the computed t = 0.132, and there were 21 degrees of freedom. The tabulated t = 2.52; therefore, the t-score of 0.132 reveals no statistically significant difference between the pretest and posttest scores for the control group.

Experimental Group

The mean for the experimental group's pretest was 8.59 and for the posttest was 13.01. The difference in mean scores was 4.43. The computed t = 4.53, and there were 21 degrees of freedom. At the α = 0.01 level of confidence, t = 2.52; therefore, the t-score of 4.53 causes one to reject the hypothesis that there is no difference in the pretest and posttest means.

Comparison

Thus, there is a clear indication that the portable intersection trainer was an effective instructional mechanism for the target population. Moreover, for the particular evaluation procedure used, it was shown to be more reliable than the existing mechanism, for no improvement in the aggregate control group was detected. (Inspection of the data in Table 3 does show individual successes with each mechanism, but a more-consistent pattern occurred with the portable intersection).

SUMMARY

Safe street crossing is a complex task. Achievement of this skill provides benefits to children in other areas of their lives, such as the following:

1. Making decisions for themselves; in order to cross a street safely, a child must decide when to wait and when it is safe to cross the street. This is often the first major decision children learn to make for themselves.
2. Assuming responsibility for themselves; in deciding when to cross a street, a child assumes responsibility for getting across the street safely.
3. Achieving a sense of independence; in learning to cross a street safely, children take a step toward being able to move around the community on their own.

The portable intersection can be an effective tool in helping mentally handicapped children adapt to community life. By learning basic safety and street-crossing skills and by increasing their awareness of traffic and pedestrian rules, the mobility of the children can be increased. Class trips should be easier for classroom teachers and shopping trips should be easier for parents.

The results of the experiment of using a portable intersection in a simulated environment to teach street-crossing skills to severely and moderately mentally retarded adolescents show that the skills acquired were transferable to actual conditions on city streets. The difficulty in using the portable intersection is its size, bulk, and the number of people required to assemble it. However, the need to stand up to intensive use mandates some sturdiness. At the Adult Skills Training Center a crew of five mentally retarded young men learned to assemble the intersection in 0.5 h and to disassemble it in 20 min. The learning experience of assembling and disassembling the system is valuable in itself for students old enough and strong enough to do the task.

The amount of time and number of people required to assemble the unit would prohibit its use in some circumstances, such as a single-unit classroom of handicapped young children and one teacher. The portable intersection could best be used as an educational tool by moving it to schools that have a large population of students and with sufficient support staff. Because of its size and weight, we recommend that the intersection be semipermanently set up. Instead of setting it up daily, setting it up on a weekly basis might be better. This would depend, of course, on the amount of room available to a particular school.

The travel-training observers and instructor thought that the students in the experimental group benefitted greatly from the experience. The portable intersection has been shown to do what it was developed to do; that is, to serve as an educational tool to accelerate the teaching of mentally handicapped students to safely cross streets.

ACKNOWLEDGMENT

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Effect of Pedestrian Signals and Signal Timing on Pedestrian Accidents

CHARLES V. ZEGEER, KENNETH S. OPIELA, AND MICHAEL J. CYNECKI

The purpose of this study was to determine whether pedestrian accidents are significantly affected by the presence of pedestrian signal indications and by different strategies for signal timing. Data related to pedestrian accidents, intersection geometrics, traffic and pedestrian volumes, roadway environment, and signal operation were collected for 1297 traffic-signalized intersections in 15 cities throughout the United States. The data were analyzed by using various statistical tests, which included branching analysis, correlation analysis, chi-square analysis, and the analysis of variance and covariance. The results showed no significant difference in pedestrian accidents between intersections that had standard-timed (concurrent walk) pedestrian signals compared with intersections that had no pedestrian signal indications. In addition, exclusive-timed locations were found to be associated with lower pedestrian accident experience for intersections with moderate-to-high pedestrian volumes when compared with both standard-timed intersections and intersections that had no pedestrian signals. In some cases pedestrian accidents were also found to be significantly affected by other variables, including street operation (one-way and two-way streets), presence of local bus operations, and area type.

Recent pedestrian safety research has uncovered numerous problems regarding current pedestrian signalization practices. The lack of uniformity in strategies and devices for pedestrian signal timing has been thought to contribute to the ineffectiveness of the signals in achieving improved pedestrian safety. Further, pedestrians have expressed considerable confusion and misunderstanding regarding the meaning of the flashing DON'T WALK indication (or flashing hand) for the clearance interval and the flashing WALK indication (or flashing man) to warn pedestrians of turning vehicles. Such confusion over the meaning of pedestrian traffic-control devices may also contribute to pedestrian safety problems.

Although many problems have been attributed to the current uses of pedestrian signals, a literature review failed to find conclusive studies that adequately quantified the effect of pedestrian signals on pedestrian accidents. The effect of pedestrian signals on safety must be understood in order to determine whether the continued use of pedestrian signals is justified. The results of this analysis can help to determine whether changes are needed in the design and deployment of pedestrian signals.

The impact of the various pedestrian signal-timing schemes on operational strategies also need to be evaluated. Schemes for pedestrian signal timing include the following (1):

1. Concurrent (standard)--allows pedestrians to walk concurrently with the movement of traffic;
2. Early release--allows pedestrians to leave the curb before vehicles are permitted to turn;
3. Late release--holds pedestrians (with respect to vehicles) until a certain portion of the phase has been given to turning vehicles;
4. Exclusive--traffic is held on all approaches to allow pedestrians to cross any street; scramble (or Barnes dance) timing is a form of exclusive timing that also allows for diagonal crossings; and
5. Other--variations of the above where pedestrians are given different indications on parallel crosswalks to protect them during special traffic phases (i.e., special left-turn phases, or split phasing).

The purpose of this study was to determine whether pedestrian accidents at signalized intersections are affected by different uses of pedestrian signals and signal-timing schemes. We hoped that the results of this analysis would (a) help to identify the types of intersections or situations where pedestrian signals are most (or least) desirable from a safety standpoint and (b) aid in determining whether changes are needed in the design of pedestrian signals to improve their effectiveness. Such information should be of considerable value to the traffic engineering community, which is responsible for the installation and timing of pedestrian signals.

BACKGROUND

Although in recent years considerable research has been conducted regarding pedestrian safety, little has been published specifically on the issue of pedestrian signals and safety. In terms of the effect of pedestrian signals on accidents, Fleig and Duffy found no significant reduction in the proportion of unsafe acts or pedestrian accidents after the installation of scramble-timed pedestrian signals at 11 locations (2). Their accident data were limited to 27 accidents in the before period and 25 accidents in the after period, with each of these periods only one year in duration. The authors of the study concluded that pedestrian signals are not effective in reducing pedestrian accidents, but the limited data used raise questions about the statistical validity of this conclusion.

Several studies have been conducted concerning the effect of pedestrian signals on pedestrian compliance and behavior, which are sometimes considered to be indirect measures of pedestrian safety. A study by Abrams and Smith in 1977 concluded that higher pedestrian compliance rates are associated with late-release techniques and that early-release timing may provide an additional measure of safety, but the benefits were not determined precisely (3). Scramble timing was found to be associated with higher violation rates than were other timing schemes (3). Mortimer conducted a study in 1973 to test compliance rates at pedestrian crossings with and without pedestrian signals (4). He found better signal compliance rates and fewer serious pedestrian-vehicle conflicts at intersections with pedestrian signals than at those without them.

Several other related studies have been conducted outside the United States regarding the effect of pedestrian signals on safety. A 1979 study in England by Inwood and Grayson found that push-button pedestrian signals (termed pelican crossings) are no more effective than black-and-white-striped crosswalks and flashing beacons (termed zebra crossings) in reducing pedestrian accidents (5). However, a study in Australia by Williams reported that accidents dropped by 60 percent at a group of locations that had pedestrian-actuated signals that were installed at former zebra crossings (6). The precise effect of each of these countermeasures was not determined. These studies were also inconclusive on the safety benefits of pedestrian signals.

Many studies conducted in the United States and abroad have used measures of effectiveness such as pedestrian compliance and behavior to evaluate the effect of pedestrian signals on pedestrian safety. However, a clear relation has not yet been established between pedestrian accidents and such surrogate measures. Although these past studies provide useful insights about pedestrian control at intersections, they do not provide sufficient information to establish the safety benefits of pedestrian signals. We therefore decided that a more-comprehensive analysis was warranted that would use several years of pedestrian accident data at a large number of urban intersections.

METHODOLOGY

The evaluation approach selected for this research involved the use of pedestrian accident experience instead of pedestrian behavior, compliance measures, or other accident surrogates to determine the effect of pedestrian signals and timing on pedestrian safety. The two types of accident analysis considered were (a) the analysis of pedestrian accident before and after the installation of a pedestrian signal and (b) a comparative analysis of accidents at locations with and without pedestrian signals. Before and after analyses can be used to determine cause-and-effect relations, preferably by using comparison sites and looking at accident trends over time in order to minimize the common threats to evaluation validity (i.e., regression-to-the-mean, changes in accident trends over time, compounding effects of other locational factors, and data instability). However, this analysis approach was rejected for this study due to (a) the small accident samples per site, (b) the difficulty in finding suitable sites (with several years of accident data before and after the installation of a pedestrian signal) and comparison sites, and (c) the problem of isolating the true effect of the pedestrian signals on pedestrian accidents from other locational features.

The comparative analysis approach involves the selection of a large sample of sites with and without pedestrian signals and the representation of various timing schemes. Intersections that have similar geometric or operational features are grouped together and accident data are compared for each group. This approach usually allows for the creation of a large data base without relying on sites where pedestrian signals have been added in recent years. The possible disadvantages with a comparative analysis are that no two intersections are exactly alike, so a large number of traffic, geometric, and operational data variables are needed for each site to help ensure reliability of results. A comparative analysis does not show cause-and-effect relation but does allow for determining relations among variables if the proper statistical tests are used. A comparative analysis approach was subsequently selected for this study.

Data Needs

Data needs were established based on the findings of the literature review, the objectives of the study, and the need to assess pedestrian accident experience and to characterize intersection locations to permit the isolation of influencing factors. The basic analysis approach was designed to compare the pedestrian accident experience between signalized locations with and without pedestrian signal indications. Since a variety of signal-timing schemes are used for pedestrian signals, it was deemed important to assess individually the effect of the various schemes on pedestrian accidents.

Independent variables were defined that would be appropriate for classifying each candidate intersection in terms of its design, operation, and environment. The prime requirement of such variables was that they represent different levels of opportunity for pedestrian accidents or should have some influence on the potential for an accident. Since pedestrian accidents are directly related to traffic and pedestrian volumes, these two variables were considered to be of major importance. Therefore, data on traffic and pedestrian volumes were collected for each intersection by leg (if available) within the period for which the accident data were available.

Additional independent variables used to describe the intersection characteristics were also identified. These variables included the following:

1. Design factors--number of lanes, intersection skewness, use and type of pedestrian signal, number of turn lanes or turn prohibitions, and street width;
2. Environmental factors--city, land use, area type, and functional classification; and
3. Operational factors--signal timing and phasing, provision for right-turn-on-red, bus operations, speed limits, one-way or two-way street operations, and parking.

The data analysis plan addressed the question of how many years of accident data would be necessary to provide sound statistical results. Although the use of pedestrian accident data was determined to be the most desirable method of measuring directly the effectiveness of pedestrian signalization options, the relative infrequency of pedestrian accidents at any location was recognized to create a problem in the statistical analysis of the data. Therefore, a conservative estimate indicated that about 1000 intersections were necessary, and 3 to 6 years of accident data per site, to ensure statistical reliability.

Copies of accident reports were obtained and reviewed before coding. All basic information about each accident, including who was at fault, the accident type, severity, contributing circumstances, and 20 other accident details, were entered into the data base. Accidents were included in the analysis only if they were within the influence of the intersection and thought to be related to a crossing maneuver at the signal. For example, highly unusual accidents (i.e., pedestrian falls from moving car, pedestrian is hit while standing on sidewalk, or police officer directing traffic) were not included. Computerized accident files were used in 2 of the 15 cities because the accident report forms were not readily available.

Site Selection

The selection of suitable sites for this study required that candidate cities first be chosen to satisfy the following criteria:

1. Cities should be willing to cooperate in the study and provide necessary data;
2. Pedestrian and traffic volume data should be available at a large number of locations from counts conducted within the past five years;
3. Other required locational data (i.e., signal timing sheets, land use maps, bus maps, and dates of when any major locational changes were made) should also be readily available;
4. Accident data should be of high quality, accidents should be referenced accurately to the proper location, and accident-reporting levels should be relatively consistent;
5. Candidate cities should cover a wide geographic range throughout the United States and represent a variety of types, density, traffic laws, and pedestrian attitudes; and
6. Cities should have an adequate sample of types of pedestrian signals and signal-timing schemes.

Of the more than 70 U.S. cities originally contacted for use in the study, 15 were selected after we determined that they substantially met the above criteria. The only city found that had more than 20 exclusive-timed intersections was Denver, Colorado. A few exclusive intersections were found in New Haven, Connecticut; Waltham, Massachusetts; Washington, D.C.; Kansas City, Missouri; West Hartford, Connecticut; Richmond, Virginia; and Tampa, Florida.

Problems were also encountered in identifying sites that had early- or late-release timing. Only a very few locations that use this scheme were found in discussions with city traffic engineers. Most engineers were of the opinion that, after flows of either automobiles or pedestrians were initiated, it is difficult to interrupt them on the same phase. Hence, very little use is made of this timing scheme within the cities contacted. The resulting categories of pedestrian signal timing included the following:

1. No pedestrian signal,
2. Concurrent timing,
3. Exclusive (including scramble) timing, and
4. Other timing (split phasing or early or late release).

Cities were selected from several geographic regions to eliminate unwanted biases in the accidents related to climate, driver attitudes, systemwide accident characteristics, areawide safety emphasis, lifestyles, and local highway design standards. Furthermore, an attempt was made to avoid cities that were considered to be highly unusual in terms of pedestrian activity, attitudes, and behavior. A total of 1297 intersections in 15 cities across the United States were selected for inclusion in the study, as indicated in Table 1.

Within each of the cities selected for data collection, candidate signalized intersections with and without pedestrian signals were selected for analysis. The selection was based on the availability of the required intersection data and the need for a relatively uniform sample of typical intersection situations. Therefore, all of the locations selected for inclusion in this study had the following features:

1. All intersections had four approach legs without unusual features; offset intersection approaches, multiple legged intersections, and traffic circles were not selected.
2. All intersections had traffic signals; some had pedestrian signal indications and others did not.
3. All intersections were in urban or suburban areas.

The locations were different to some degree in terms of

1. The use of pedestrian signal-timing schemes (concurrent, no pedestrian signal, or exclusive),
2. The range of pedestrian volume (about 50-50 000 pedestrians/day crossing all approaches) and traffic volumes (about 1600-78 000 entering vehicles/day),
3. Land uses (commercial, residential, or recreational), and
4. A variety of other roadway features (number of lanes, turn prohibitions, or presence or absence of right-turn-on-red).

Data Collection

The data collection effort usually involved one or more visits to the appropriate offices in the selected cities. Data were compiled from traffic, accident, and roadway data files; maps; and computer outputs. The unavailability of certain data in some of the cities necessitated field surveys and pedestrian volume counts at most of the intersections. After the data were coded, they were checked by a series of manual and computerized reviews of the data files to ensure integrity. The corrected data file was condensed and reformatted to facilitate analysis by using the Statistical Package for the Social Sciences (SPSS) and Statistical Analysis System (SAS) statistical packages.

DATA ANALYSIS

A comprehensive statistical analysis was undertaken on the data files to determine the effects of pedestrian signals on accidents. It was recognized at the outset that small samples of pedestrian accidents per site would exist, which would produce a skewed (nonnormal) distribution of accidents by site. This required the selection of not only a large number of sites and three to six years of accident data but also the selection of appropriate statistical tests. The analysis first included a review of accident characteristics to provide an understanding of the factors associated with pedestrian accidents. Next, correlation analysis was used to determine what traffic and roadway variables were most highly related to pedestrian accidents. A branching analysis was used to indicate variables that explained the most variation in pedestrian accidents and to identify the breakpoint levels that were important in subsequent statistical tests.

Based on the results of the correlation and branching analysis, chi-square and analysis of variance and covariance tests were applied. The chi-square test was used to compare the distributions of accidents for locations with and without pedestrian signals and for different timing schemes (i.e., concurrent versus exclusive timing) for various data groups. Finally, analysis of variance and covariance tests were used to isolate the effect of pedestrian signals and timing on accidents while controlling for other influencing variables.

Table 1. Summary of intersections from each city used in the study.

City	Locations: No Pedestrian Signal	Concurrent Timing	Exclusive Timing	Other Timing	Total	Accident Data-Collection Period	Total No. of Pedestrian Accidents
Albany, NY	9	0	0	0	9	1976-1980	17
Chicago, IL	112	112	0	12	236	1977-1980	635
Columbus, OH	1	46	0	3	47	1978-1980	54
Denver, CO	0	16	39	0	55	1978-1980	34
Detroit, MI	62	108	0	0	170	1978-1980	222
Grand Rapids, MI	7	9	0	0	16	1978-1980	10
Kansas City, MO	10	28	1	0	39	1978-1980	11
New Haven, CT	27	0	13	0	43	1977-1979	43
Richmond, VA	84	2	11	0	97	1978-1980	55
Seattle, WA	41	99	0	0	140	1974-1979	342
Tampa, FL	21	21	16	0	58	1977-1980	33
Toledo, OH	66	113	0	5	184	1976-1980	198
Waltham, MA	0	0	11	0	11	1977-1980	2
Washington, DC	68	104	10	5	187	1974-1979	425
West Hartford, CT	0	0	5	0	5	1976-1980	0
Total	508	658	109	22	1297		2081

Characteristics of Pedestrian Accidents

The accident data collected and analyzed consisted of 2081 accidents that occurred at the 1297 intersections shown in Table 1. The analysis period ranged from three to six years in each city based on the availability of historical accident data. Most of the accident data used in this study was associated with intersections in five large urban areas (Chicago, Illinois; Washington, D.C.; Toledo, Ohio; Detroit, Michigan; and Seattle, Washington) that represented more than 70 percent of the locations and 88 percent of the accidents in the sample.

The characteristics associated with the pedestrian accidents are summarized in Table 2, including details on accident severity, pedestrian age and sex, collision type, pedestrian action, and driver action. Only 29 (1.4 percent) of the 2081 accidents resulted in a pedestrian fatality; the vast majority (93 percent) of the accidents were injury accidents. In addition, 98 collisions (4.7 percent) involved no injury to the pedestrian.

Summaries of accidents by age and sex of pedestrians indicate that more than 40 percent of the accidents involved young and elderly persons and males are hit more often than are females. The designation of accident type was based on driver intent at the intersection (i.e., straight or turning right or left). The most common type of pedestrian accident involved a through vehicle (60.3 percent). Right-turning movements accounted for 14.8 percent of the accidents; left-turning vehicles were involved in 22.5 percent of the accidents.

The determination of pedestrian and driver action involved review of hard copy accident reports to determine whether the accident was caused by a hazardous pedestrian action (i.e., walking or running against the signal) or a hazardous driver action (i.e., run red light or failure to yield on a turn). The investigating officer's remarks and description of the accident and site condition were the basis for this determination. For those accidents where the pedestrian action could be determined, the pedestrian was crossing with the signal in 49.2 percent of the accidents. For the 1446 accidents where the driver action could be determined, 41.5 percent were judged to be driving safely at the time of the accident and were not judged at fault in the accident. This indicates that approximately one-half of the pedestrian accidents at intersections are caused by pedestrians in violation of the traffic or pedestrian signal. In the other one-half of the pedestrian accidents, the pedestrians were following the instructions of traffic or pedestrian signals but were struck by motorists who failed to observe or yield to pedestrians in time.

Correlation Analysis

The purpose of this analysis was to determine which independent variables (i.e., traffic, roadway, and signal variables) to include in subsequent analyses based on their relations to each other and accident data (dependent variables). Pearson's correlations were computed for various combinations of continuous dependent and independent variables to determine those combinations that have the strongest interrelations. The correlations between key dependent variables and independent variables were generally low (r-values of less than 0.6). The strongest of these relations were found between mean pedestrian accidents per year and both pedestrian volumes and vehicle volumes. Generally low correlations were expected due to the wide variety of features of the intersections that influence the pedestrian accident experience at a location. No attempt was made to improve the correlations through the inclusion of multiple independent variables by stepwise linear regression analysis or through data stratification. The decision was made to proceed to other analysis techniques (branching analysis and analysis of covariance) to further quantify the effect of individual variables on pedestrian accidents.

Branching Analysis

A branching analysis was conducted on 1289 signalized intersections to determine what traffic and roadway variables explain the most variation in pedestrian accident experience. Also, we hoped that the analysis would identify breakpoint levels of pedestrian and traffic volumes, based on pedestrian accident experience, for data stratification in subsequent analyses. The results of the branching analysis (shown in Figure 1) indicated the following:

1. Pedestrian volume is the variable that explains the greatest amount of variation in pedestrian accidents (14.9 percent of variance explained).
2. After several groupings of pedestrian volume were tested, the most-important breakpoint in pedestrian accidents occurs for a pedestrian average daily traffic (ADT) level of 1200. In fact, for the 609 locations that had pedestrian ADTs less than 1200, the mean annual pedestrian accidents per location was 0.178, compared with 0.553 for loca-

tions that had pedestrian ADT of 1200 or more. Another breakpoint occurred at a pedestrian ADT of 3500.

3. The most-important breakpoints in the traffic volume data (in terms of pedestrian accidents) occurred at ADT levels of 27 500 and 18 000.

4. Beside pedestrian and traffic volume, other variables that were found to be of some importance in explaining pedestrian accidents included bus operation (a bus route on one or more of the streets at the intersection), street operation (one-way versus two-way streets), percentage of vehicle turns, intersection design, area type (CBD, fringe, or residential), and street approach width.

Table 2. Summary of pedestrian accident data.

Classification	Characteristics	No.	Percent
Accident severity	Fatal injury	29	1.4
	Nonfatal injury	1935	93.0
	No injury	98	4.7
	Unknown	19	0.9
	Total	2081	
Age group[a]	0-15	225	22.0
	16-59	504	49.4
	≤60	186	18.2
	Unknown	106	10.4
	Total	1021	
Sex of pedestrian	Male	546	53.5
	Female	465	45.5
	Unknown	10	1.0
	Total	1021	
Driver intent	Straight	1256	60.3
	Right turn	308	14.8
	Left turn	468	22.5
	Other or unknown	49	2.4
	Total	2081	
Pedestrian action[b]	No hazardous action	475	49.2
	Hazardous action	449	46.5
	Unknown	42	4.3
	Total	966	
Driver action[c]	No hazardous action	600	41.5
	Hazardous action	805	55.7
	Unknown	41	2.8
	Total	1446	

[a]Excludes accidents from Washington, DC, and Chicago, IL.
[b]Excludes accidents from Washington, DC; Chicago, IL; and Richmond, VA.
[c]Excludes accidents from Washington, DC.

5. Although all intersections in the analysis had a traffic signal, the presence or absence of a pedestrian signal indication had no significant effect on pedestrian accident experience.

Further branching analysis was conducted separately for the following three groups of intersections:

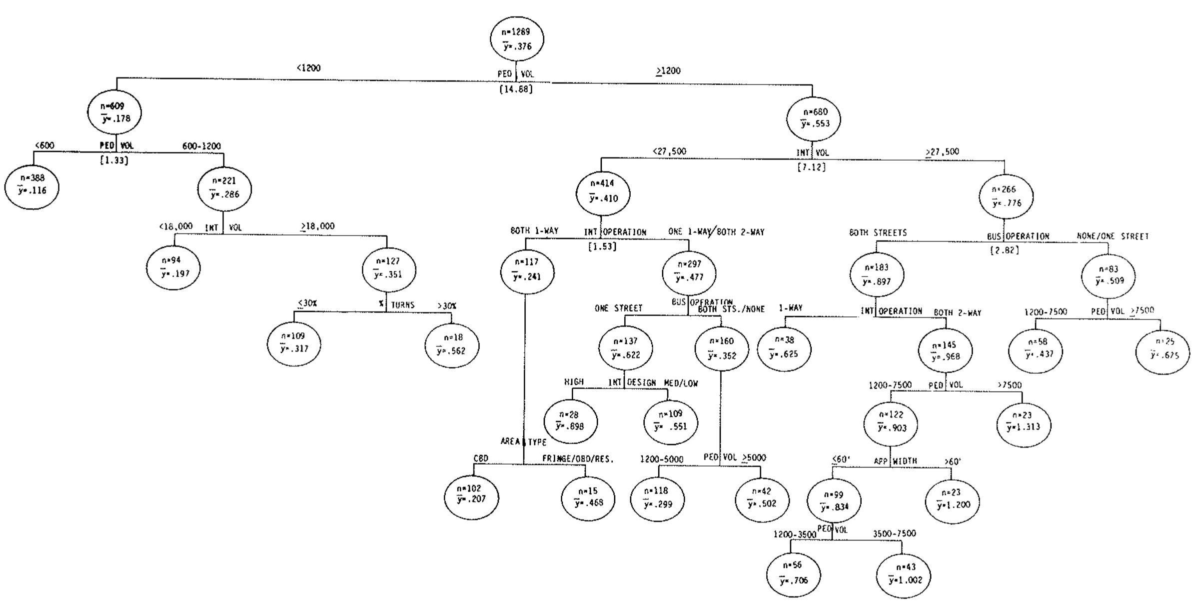
1. The 507 intersections that did not have pedestrian signals,
2. The 652 locations that had concurrent pedestrian signals, and
3. The 109 locations that had exclusive pedestrian signal timing.

The following general conclusions were found:

1. The presence of buses was found to be an important factor in pedestrian accidents for location groups above 1000 pedestrians/day for locations that had concurrent pedestrian timing and also for locations that did not have pedestrian signals;
2. For exclusive-timed signals that had pedestrian ADT above 8000, pedestrian accidents were much lower at the intersection of two one-way streets than for intersections of two-way street approaches;
3. For intersections that did not have pedestrian signals on bus routes and ADT above 1000, a higher accident experience was found at residential intersections compared with nonresidential areas; and
4. The presence of a wide street width (i.e., greater than 50 ft) was associated with higher pedestrian accidents for some categories of roads that had pedestrian ADTs above 1000.

Three classes of traffic volume were chosen based on breakpoints determined from the branching analysis to assess the sensitivity of pedestrian accidents as a function of pedestrian and traffic volume, as illustrated in Figure 2. Intersection classes of pedestrian and traffic volume were grouped together and the pedestrian accidents for three years were plotted. Three traffic volume groups and 11 pedestrian volume groups were used to illustrate the expected number of pedestrian accidents at an intersection for a three-year period.

Figure 1. Branching analysis by using mean pedestrian accidents per year.



The plots include intersections that did not have pedestrian signals and also those that had concurrent pedestrian signals (since no significant difference was found in pedestrian accidents between these two groups). The curves show the sensitivity of pedestrian accidents to traffic and pedestrian volumes. Calculation of correlation coefficients (Pearson's r) is not appropriate in this case because each data point represents the mean accident experience of numerous intersections in a particular volume class.

Based on the results of the branching analysis, a breakdown analysis was used to summarize average pedestrian accidents for various classifications of traffic volume, pedestrian volume, and signal-timing scheme (Table 3). This table provides a simplistic description of the trends in pedestrian accidents in relation to pedestrian signalization and volume factors. These results are not sufficient, however, to make conclusive statements relative to these trends without further testing of the true effects by using more sophisticated analysis of variance

Figure 2. Relation between pedestrian volume and pedestrian accident experience for three levels of vehicle volumes.

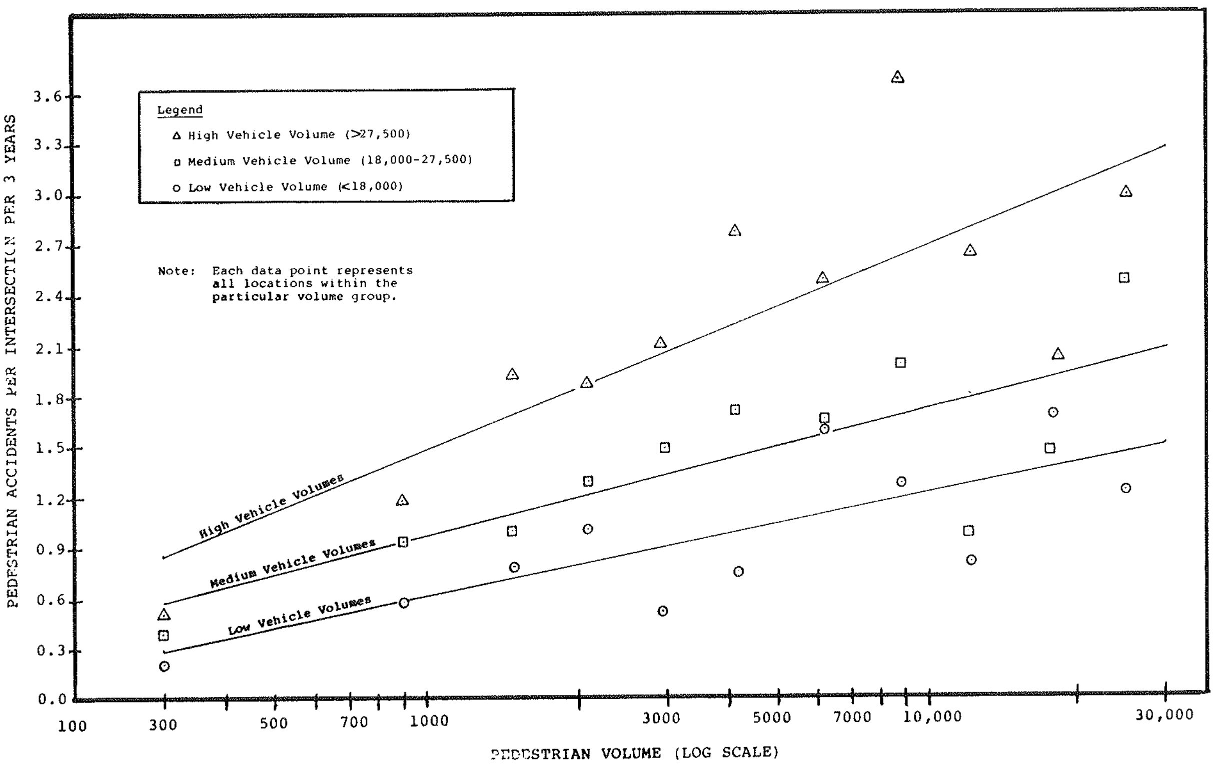


Table 3. Summary of pedestrian accidents per year per site by pedestrian signal type and volume class.

Volume Class	No Pedestrian Signal Indication: Mean Annual Pedestrian Accidents per Intersection	Intersections	Concurrent Pedestrian Signal Timing: Mean Annual Pedestrian Accidents per Intersection	Intersections	Exclusive Timing: Mean Annual Pedestrian Accidents per Intersection	Intersections
Low pedestrian volume, 0-1200, and low vehicle volume, 0-18 000	0.09	127	0.14	120	0.11	12
Medium pedestrian volume, 1201-3500, and low vehicle volume, 0-18 000	0.28	46	0.25	27	0.40	8
High pedestrian volume, >3500, and low vehicle volume, 0-18 000	0.25	16	0.50	22	0.29	21
Low pedestrian volume, 0-1200, and medium vehicle volume, 18 001-27 500	0.19	84	0.21	78	0.08	10
Medium pedestrian volume, 1201-3500, and medium vehicle volume, 18 001-27 500	0.41	51	0.41	61	0.20	8
High pedestrian volume, >3500, and medium vehicle volume, 18 001-27 500	0.65	37	0.52	89	0.21	25
Low pedestrian volume, 0-1200, and high vehicle volume, >27 500	0.23	74	0.28	92	0.17	12
Medium pedestrian volume, 1201-3500, and high vehicle volume, >27 500	0.52	47	0.73	79		
High pedestrian volume, >3500, and high vehicle volume, >27 500	0.88	26	0.91	90	0.66	13

Table 4. Distribution of locations by pedestrian accident experience and signal timing scheme.

Pedestrian Volume per Day	Pedestrian Accidents per 3 Years	Intersections with No Pedestrian Signals		Intersections with Concurrent-Timed Pedestrian Signals		Intersections with Exclusive-Timed Pedestrian Signals	
		No.	Percent	No.	Percent	No.	Percent
<1200	0	177	62.1	155	53.4	25	73.5
	>0 to 1	71	24.9	81	27.9	5	147
	>1 to 1.5	14	4.9	17	5.9	2	5.9
	>1.5	23	8.1	37	12.8	2	5.9
	Total	285		290		34	
≥1200	0	57	25.5	64	17.4	32	42.7
	>0 to 1	64	28.7	99	26.9	25	33.3
	>1 to 1.5	29	13.0	46	12.5	3	4.0
	>1.5 to 2.25	28	12.6	58	15.8	7	9.3
	>2.25	45	20.2	101	27.4	8	10.7
	Total	223		368		75	

Table 5. Summary of results from the chi-square analysis.

Comparison	Pedestrian Volume	Difference Distributions[a]	χ^2	df	Level of Significance
No pedestrian signal versus concurrent-timed pedestrian signal	<1200/day	No	5.630	3	
	≥1200/day	No	8.664	4	
No pedestrian signal versus exclusive-timed pedestrian signal	<1200/day	No	2.197	3	
	≥1200/day	Yes	13.492	4	0.01
Concurrent-timed pedestrian signal versus exclusive-timed pedestrian signal	<1200/day	No	5.410	3	
	≥1200/day	Yes	32.240	4	0.01

[a]Significant at the 0.05 level of confidence.

tests or considering in greater detail the influence of the many other geometric, traffic, and locational factors.

Chi-Square Analysis

The chi-square test was used to test for a statistically significant association between pedestrian accidents and pedestrian signal timing schemes (including the no-pedestrian-signal situation). The chi-square test was determined appropriate for use in this study because it can relate a continuous, nonnormal variable (i.e., Poisson distribution of accidents) to one or more categorical variables (i.e., categories of pedestrian signal timing).

Distributions of locations that had various pedestrian signal schemes were established separately for locations that had pedestrian volumes less than 1200/day and locations that had 1200 or more pedestrians/day (Table 4). Four to five ranges of pedestrian accidents (per three-year period) were developed for use in the chi-square analysis. The number and percentage of locations that fall into each category are given in Table 4, which indicates a highly skewed (i.e., Poisson) distribution for each group of locations. The break point of 1200 pedestrians/day was used to separate the data set because of its importance in explaining variation in pedestrian accidents (as found from the branching analysis).

The results of the chi-square analysis are summarized in Table 5 and indicate the following:

1. No significant difference was found in pedestrian accident distributions when comparing locations that did not have pedestrian signals to locations that had concurrent pedestrian signals. This was true for both groups of pedestrian volume (i.e., <1200 and ≥1200 pedestrians/day).
2. For intersections that had fewer than 1200 pedestrians/day, no significant difference was found in pedestrian accident distributions when comparing exclusive-timed pedestrian signals with both the no-pedestrian-signal groups and also locations that had concurrent pedestrian signals. The low number of exclusive-timed signals (34) in this volume category may have caused this result.
3. For intersections that have 1200 or more pedestrians/day, a significant difference was found between accident distributions for exclusive-timed pedestrian signals compared with locations that did not have pedestrian signals as well as locations that had concurrent pedestrian signals (0.01 level of confidence in each case). A higher proportion of exclusive-timed locations were in the low accident groups than in the concurrent signal group or the no-pedestrian-signal group.

Analysis of Variance and Covariance

The statistical investigations were pursued to a still higher level in an attempt to explain the findings of the previous analyses. This involved the use of analysis of variance and covariance techniques. The analysis of variance method was used to divide the observed variation in experimental data into parts, and each part is assigned to a known source or variable. The purpose of the analysis was to determine whether a particular part of the variation is greater than would be expected by chance. The null hypothesis generally assumed for the analysis of variance was that the mean of the sample data is not significantly different.

The analysis of covariance is similar to the analysis of variance, but it allows for the inclusion of covariates in the analysis to adjust the dependent variable (i.e., pedestrian accidents per year) for continuous variables where appropriate. For example, the continuous covariates selected and used in most of the analysis of variance tests were pedestrian volume and traffic volume. This allowed for determining the true effect of pedestrian signals on pedestrian accidents while controlling for the effects of varying levels of pedestrian and traffic volumes. Examples of the discrete (noncontinuous) variables included in the analysis included street operation, absence or presence of right-turn-on-red regulations, bus operation, area type, and others.

The final selection of variables used in the analysis of variance analysis was based on the results of the correlation analysis, the branching analysis, and preliminary analysis of variance runs. The dependent variables used the analysis of variance runs included various types of mean annual pedestrian accidents, including total accidents, right-turn accidents, left-turn accidents, and total turn accidents.

The independent variables used in one or more of the analysis of variance runs as covariates included the following:

1. Total traffic volume [annual average daily traffic (AADT)],
2. Right-turn traffic volume (AADT),
3. Left-turn traffic volume (AADT),
4. Total turning traffic volume (AADT), and
5. Total pedestrian volume (AADT).

The analysis of variance runs were made with varying combinations of the following classification variables:

1. Area type code,
2. Street operation,
3. Signal operation code, and
4. City code.

Numerous analysis of variance tests were undertaken by using the SPSS program to address several basic issues.

Issue 1: Are Mean Pedestrian Accidents per Year Significantly Affected by the Presence of Various Types of Pedestrian Signal Timing Schemes?

The mean pedestrian accidents per year are significantly affected by the presence of various pedestrian-timing schemes (at the 0.001 level) when adjustments are made for pedestrian volumes, traffic volume, and street operation. The lowest adjusted mean accidents per year occurred for exclusive timing (0.22), and the highest was for concurrent pedestrian signals (0.40). Other values included no pedestrian signal (0.36) and other semi-exclusive or protected signals (0.38).

Similar comparisons were also made for the mean turning pedestrian accidents per year. The independent variables included operation code (i.e., one-way or two-way combinations), pedestrian volume, and total vehicle turning volume. There were significant differences in the mean pedestrian accidents for the various signal timing schemes. For both types of pedestrian accidents, exclusive-timed locations had the lowest mean accidents per year. Details of the results are given in Table 6.

Issue 2: Is there a Significant Difference in Pedestrian Accidents between Intersections that Did Not Have Pedestrian Signals and Intersections that Had Concurrent Pedestrian Signal Timing?

The total mean pedestrian accidents are not significantly different (at the 0.05 level of confidence) between intersections with no pedestrian signals compared to intersections with standard pedestrian signals, when adjustments are made for pedestrian volume, traffic volume, and street operation code. This finding agrees with the findings from the chi-square test.

Similar comparisons were also made for mean turning pedestrian accidents per year. The independent variables included operation code, pedestrian volume, and total vehicle turning volume. There was a significant difference (at the 0.05 level) between no pedestrian signal locations and locations that had concurrent pedestrian signals for the mean pedestrian turning accidents per year. The analysis also indicated that locations that did not have pedestrian signals had significantly fewer pedestrian turning accidents than those that had concurrent pedestrian signals. However, the sample size for turning accidents is small, and further in-depth testing should be done to verify this apparent affect. Details of the results are given in Table 6. This finding may be the result of pedestrians' failure to be cautious of turning vehicles at locations with pedestrian signal heads.

Comparisons of mean annual pedestrian accidents were made between locations that did not have pedestrian signals and those that had concurrent pedestrian signals for the cities of Chicago, Detroit, Washington, Toledo, and Seattle (individually and as a group) to determine whether similar results were found in each major city. Again, the independent variables were traffic volume, pedestrian volume, signal operation, and street operation. No significant difference in pedestrian accidents was found in any city between intersections that did not have pedestrian signals versus intersections that had concurrent pedestrian signals. This finding tends to indicate that regional differences did not bias the results. The concurrent signal timing fared best in Seattle than in any other city when compared with locations that did not have pedestrian signals (although the difference in accident means was not significant in any city). This trend might be explained, since Seattle probably has lower pedestrian violation rates (and better compliance to pedestrian signals) compared with the other large cities in the sample. As signal compliance increases, their effect on pedestrian safety should improve. When a similar analysis was conducted for the five cities combined (also controlling for the differences in the local accident experience), no significant differences were found in either the total mean pedestrian accidents or the mean turning pedestrian accidents. These findings are summarized in Table 7.

Issue 3: Is the Difference Significant in Pedestrian Accidents between Intersections that Did Not Have Pedestrian Signals and Intersections that Had Exclusive Pedestrian Signal Timing?

The mean pedestrian accidents per year are significantly different between intersections with exclusive timing schemes compared with intersections with no pedestrian signals (at the 0.001 level) when controlled for street operation, pedestrian volume, and traffic volume. The mean adjusted pedestrian accidents at exclusive locations (0.15/year) is significantly lower than for no pedestrian signal (0.33/year). The chi-square analysis confirmed this finding for locations that had pedestrian ADTs above 1200.

Similar comparisons were also made for mean-turning accidents per year. The independent variables included operation code, pedestrian volume, and vehicle turning volume. In each case, the mean adjusted accidents per year were significantly lower at exclusive-time locations than at locations that did not have pedestrian signals (at the 0.01 level of confidence). Details of the results are given in Table 6.

Issue 4: Is there a Significant Difference in Pedestrian Accidents Between Intersections that Have Concurrent Pedestrian Signal Timing and Intersections that Have Exclusive Pedestrian Signal Timing?

The total mean pedestrian accidents are significantly different (at the 0.001 level) between intersections that have standard pedestrian signal timing and intersections that have exclusive pedestrian signal timing when accounting for the effects of street operation, pedestrian volume, and traffic volume. The mean adjusted pedestrian accidents at exclusive locations (0.27/year) is significantly lower than the mean pedestrian accidents for standard signal timing (0.43/year).

Similar comparisons were also made with mean turning accidents per year. The independent variables included street operation, pedestrian volume,

Table 6. Summary of analysis of variance results for different pedestrian signal alternatives.

Comparison	Dependent Variable	Alternative	Adjusted Mean	Control Variables	Significant Difference[e]	Level of Significance
All pedestrian signal alternatives	Mean pedestrian accidents per year	No pedestrian signal[a] Concurrent[b] Exclusive[c] Other[d]	0.36 0.40 0.22 0.38	Pedestrian volume, total traffic volume, street operation, pedestrian signal alternatives	Yes	0.001
	Mean pedestrian turning accidents per year	No pedestrian signal[a] Concurrent[b] Exclusive[c] Other[d]	0.13 0.17 0.01 0.20	Pedestrian volume, total traffic volume, street operation, pedestrian signal alternatives	Yes	0.001
No pedestrian signal indication versus concurrent pedestrian signal timing	Mean pedestrian accidents per year	No pedestrian signal[a] Concurrent[b]	0.36 0.40	Pedestrian volume, total traffic volume, street operation, pedestrian signal alternatives	No	0.130
	Mean pedestrian turning accidents per year	No pedestrian signal[a] Concurrent[b]	0.12 0.15	Pedestrian volume, total traffic volume, street operation, pedestrian signal alternatives	Yes	0.048
No pedestrian signal indication versus exclusive pedestrian signal timing	Mean pedestrian accidents per year	No pedestrian signal[a] Exclusive[c]	0.33 0.15	Pedestrian volume, total traffic volume, street operation, pedestrian signal alternatives	Yes	0.001
	Mean pedestrian turning accidents per year	No pedestrian signal[a] Exclusive[c]	0.11 0.00	Pedestrian volume, total traffic volume, street operation, pedestrian signal alternatives	Yes	0.001
Concurrent pedestrian timing versus exclusive pedestrian signal timing	Mean pedestrian accidents per year	Concurrent[b] Exclusive[c]	0.43 0.27	Pedestrian volume, total traffic volume, street operation, pedestrian signal alternatives	Yes	0.001
	Mean pedestrian turning accidents per year	Concurrent[b] Exclusive[c]	0.17 0.03	Pedestrian volume, total traffic volume, street operation, pedestrian signal alternatives	Yes	0.001

[a]n = 508. [b]n = 658. [c]n = 109. [d]n = 22. [e]Significant at the 0.05 level of confidence.

Table 7. Summary of analysis of variance results by city.

City	Dependent Variable	Alternative	Adjusted Mean	Sample Size	Control Variables	Significant Difference[a]	Level of Significance
Chicago	Mean pedestrian accidents per year	No pedestrian signal Concurrent	0.60 0.72	112 112	Pedestrian volume, total traffic volume, street operation, pedestrian signal alternatives	No	0.137
Detroit	Mean pedestrian accidents per year	No pedestrian signal Concurrent	0.44 0.44	62 108	Pedestrian volume, total traffic volume, street operation, pedestrian signal alternatives	No	0.919
Seattle	Mean pedestrian accidents per year	No pedestrian signal Concurrent	0.45 0.39	41 99	Pedestrian volume, total traffic volume, street operation, pedestrian signal alternatives	No	0.443
Toledo	Mean pedestrian accidents per year	No pedestrian signal Concurrent	0.15 0.25	66 113	Pedestrian volume, total traffic volume, street operation, pedestrian signal alternatives	No	0.078
Washington	Mean pedestrian accidents per year	No pedestrian signal Concurrent	0.37 0.40	68 104	Pedestrian volume, total traffic volume, street operation, pedestrian signal alternatives	No	0.591
Chicago Detroit Seattle Toledo Washington	Mean pedestrian accidents per year	No pedestrian signal Concurrent	0.41 0.44	349 536	Pedestrian volume, total traffic volume, street operation, pedestrian signal alternatives, city code	No	0.375
Chicago Detroit Seattle Toledo Washington	Mean pedestrian turning accidents per year	No pedestrian signal Concurrent	0.15 0.17	349 536	Pedestrian volume, total traffic volume, street operation, pedestrian signal alternatives, city code	No	0.193

[a]Significant at the 0.05 level of confidence.

and total vehicle turning volume. The exclusive-timed intersections had significantly lower accident experience than the standard-timed signal locations.

Issue 5: What Traffic, Geometric, and Operational Variables Have a Significant Effect on Pedestrian Accidents at Signalized Urban Intersections?

Based on numerous analysis of variance runs, variables that have a significant effect (at the 0.05 level) on total pedestrian accidents for some intersection groups include the following:

1. Urban area type (suburban streets had significantly higher pedestrian accidents than did those of other areas,
2. Street operation (intersections of two one-way streets had significantly lower pedestrian accidents than intersections of two, two-way streets), and
3. The presence of bus routes on one or both streets of the intersection was associated with higher pedestrian accidents for some intersection subgroups.

SUMMARY AND CONCLUSIONS

This paper summarizes the study undertaken to determine the safety benefits derived from the use of pedestrian signals. The research approach involved the collection, reduction, and analysis of accident, traffic, and design data for 1297 urban intersections (which had 2081 pedestrian accidents over a three- to six-year period) in 15 cities. The signalization options included no pedestrian signals, standard (concurrent) timing, exclusive timing, and other timing schemes. Insufficient samples were available from the other category for statistical analysis.

The use of concurrent-timed pedestrian signals was found to have no significant effect on pedestrian accident distributions (based on chi-square test) or pedestrian accident frequencies (analysis of variance and covariance) for a sample of more than 1100 locations that represented these two groups. This finding was also true for the five largest cities in the data sample (Chicago, Washington, D.C., Detroit, Seattle, and Toledo).

The presence of exclusive-timed, protected pedestrian intervals (including scramble-timed intersections) was associated with significantly lower pedestrian accident experience when compared with locations with either concurrent-timed pedestrian signals or no pedestrian signals, when controlled for other important data variables (analysis of covariance). This finding was supported by the result of the chi-square test for intersections that have pedestrian volumes above 1200. However, this finding was not found for intersections that had pedestrian volumes less than 1200/day, possibly due to the limited sample of exclusive-timed signal locations within that volume category.

The number of pedestrian accidents that involved turning vehicles was found to be significantly higher for locations that had concurrent-timed pedestrian signals than for locations that did not have pedestrian signals when other important variables were controlled (analysis of covariance). However, this finding is not conclusive and cannot be strongly supported due to a small sample of turning pedestrian accidents. Further testing is needed to confirm this finding, but such a trend could possibly be explained by the possibility that pedestrians are often less cautious or fail to look around for turning vehicles at locations that have a WALK signal, particularly if they feel an added sense of protection when they see the WALK signal.

Several operational variables were found to have a significant effect on pedestrian accidents at urban signalized intersections. The branching and regression analysis indicated that pedestrian volume is the single-most-important variable in explaining the variation in pedestrian accidents and a significant, direct relation exists. The most-important breakpoints occur at pedestrian volume levels of 1200 and 3500 pedestrians/day (branching analysis).

Traffic volume is the second-most-important variable in explaining pedestrian accidents, and it also has a significant, direct relation to pedestrian accidents (branching analysis, regression, and analysis of covariance). The important breakpoints occur at traffic volume levels of 27 500 and 18 000 vehicles/day. Other variables were also found to have an important effect on pedestrian accidents.

RECOMMENDATIONS

The results of the analyses show that standard-timed (concurrent) pedestrian signals have no significant effect on pedestrian accidents compared with locations that do not have pedestrian signals. In fact, one analysis indicated that a significantly higher number of turning accidents are associated with concurrent pedestrian signal timing compared with intersections that do not have pedestrian signals (although not conclusive). The presence of exclusive-timed pedestrian signals are associated with significantly lower pedestrian accidents compared with the absence of pedestrian signals and the presence of standard-time pedestrian signals, particularly for locations that have moderate-to-high pedestrian volumes (more than 1200/day). However, many U.S. cities discourage the use of exclusive-timed (or scramble) pedestrian signals, since increased pedestrian and vehicle delay have been associated with such timing.

Concurrent timing is by far the most commonly used pedestrian signal timing. However, the use of pedestrian indications with concurrent timing was not found to be effective in reducing pedestrian accidents. Several possible reasons for their lack of effectiveness in reducing pedestrian accidents include the following:

1. Pedestrian respect for and compliance with pedestrian signal indications is poor in most cities. Violations of the DON'T WALK message are higher than 50 percent in many cities. This disrespect and violation of the pedestrian signals is a major reason for their ineffectiveness in reducing pedestrian accidents.
2. The presence of a pedestrian signal indication may tend to create a false sense of security and may cause many pedestrians to have the mistaken impression that they are fully protected and have no reason to use caution. The absence of a pedestrian indication at a signalized locations sometimes gives pedestrians the feeling that they are on their own. This could cause many pedestrians to exercise more caution regarding turning vehicles.
3. The use of the flashing WALK has been shown in other studies to be ineffective in adequately warning pedestrians to watch for turning vehicles. In fact, one study found that only 2.5 percent of the pedestrians understood the intended meaning of the flashing and steady WALK indications. Also, many states have not incorporated the flashing WALK signal into their state policies, which has caused nonuniformity in the use of pedestrian signal messages in the United States.
4. Some studies have found that the flashing DON'T WALK indication (clearance interval) is also not well understood by many pedestrians, and many

pedestrians believe that traffic will be released during the flashing DON'T WALK interval.

5. Pedestrian-actuation devices are used too infrequently by pedestrians and, therefore, the use and respect for pedestrian signals may be minimized at such locations. One study showed that they are used by less than 35 percent of the pedestrians in crossing at many sites.

The results of this analysis, although they raise questions about the effectiveness of pedestrian signalization, are not believed to justify the elimination of pedestrian signals. We recommend that city and state agencies take a closer look before indiscriminately installing pedestrian signals at all traffic signalized locations. Such pedestrian signals are expensive to install and maintain (for a large number of sites), and they may not be justified at many locations. Based on the findings of this study, further research may be desirable to further quantify the optimal use of pedestrian signals, including the following topics:

1. Determine the effect of intersection type on pedestrian safety by considering differences in functional classifications, lane configuration, crosswalk length, and special signal phasing;
2. Assess the effect of regional differences in pedestrian behavior, accident reporting, and pedestrian enforcement policies;
3. Investigate further the influence of pedestrian activities related to accident experience by type of pedestrian signal timing; and
4. Assess the impacts of general pedestrian compliance and understanding of signal indications on accident experience.

Only after the completion of such additional research can revised policies and practices be implemented.

Also, further efforts should be made to determine means to improve the effectiveness of standard pedestrian signals by making them more understandable, particularly in terms of the flashing WALK and the flashing DON'T WALK intervals. Also, efforts should be undertaken to determine the appropriateness of the pedestrian signal warrants currently given in the Manual on Uniform Traffic Control Devices (7) to determine whether more-realistic warrants are justified.

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Pedestrian Flows at Signalized Intersections

MARK VIRKLER

Early techniques for dealing with pedestrian flows at signalized intersections were concerned with the minimum green time needed for crossing a street and often did not treat green time as a function of the number of people who cross. Recently, new knowledge has been gained about characteristics of pedestrian flow, including relations among speed, flow, and density. In the Interim Materials on Highway Capacity, a method is presented for pedestrian flows and queues at intersections. Some flaws in the method are examined here and a different approach for analyzing the problem is presented.

The presence of pedestrians can have important effects on the operation of signalized intersections. Pedestrian crossing times can often determine minimum green times, and, therefore, minimum cycle lengths (1, p. 810). If insufficient crossing time is provided, pedestrians in crosswalks may adversely affect vehicular capacity and, of course, their own safety. Various methods have been proposed for ensuring adequate pedestrian crossing times (1-3). Three of these methods are discussed below.

The Interim Materials on Highway Capacity (4, pp. 115-147) contain a more-comprehensive procedure for the analysis of pedestrian requirements at signalized intersections. The procedure provides for the analysis of space requirements (for queuing and circulation) on the sidewalk at intersections and for determining needed crosswalk widths. Unfortunately, the procedure has some severe shortcomings. The purpose of this paper is to review the above procedures critically and to provide insight to the analysis of the problem.

EARLIER PROCEDURES

The 1976 Transportation and Traffic Engineering Handbook (1, p. 810) contains a simple equation for determining the minimum green time needed for pedestrians:

$$G = D/v_p \tag{1}$$

where

- G = minimum green time (s),
- D = length of the longest crosswalk in use during the phase (ft), and
- v_p = pedestrian walking speed (ft/s).

The handbook states that a walking speed of 4 ft/s is a value frequently used.

Pignataro (2) has suggested a somewhat similar procedure that can be described by the following equation:

$$G + Y = t + (D/v_p) \tag{2}$$

where Y is the vehicle clearance interval (yellow or yellow plus all red time) in seconds and t is the pedestrian starting time, also in seconds.

Pignataro suggests that the pedestrian starting time be not less than 5 s. Where pedestrian (WALK-DON'T WALK) signals are used, the starting time would be 7 s and the vehicle clearance interval (Y) would not be included on the left side of Equation 2. Pignataro does not explain why this difference occurs. The 7-s starting time is in agreement with the 1971 Manual on Uniform Traffic Control Devices (5, p. 245). The 1978 edition of the manual calls for a walk interval of at least 4-7 s so that pedestrians will have adequate opportunity to leave the curb (6) before the DON'T WALK clearance interval is shown. Both the 1971 and 1978 manuals suggest a pedestrian walk speed of 4.0 ft/s; Pignataro suggests 3.5-4.0 ft/s.

A common weakness of the above procedures is that they do not explicitly consider the number of people who cross during a particular phase. A pedestrian toward the back of a queue must wait for the pedestrians in front to perceive and react to the signal change and then wait for them to proceed. The procedures do not guarantee an adequate starting time for those toward the back of the queue.

The Institute of Traffic Engineers developed a recommended practice, A Program for School Crossing Protection, which considers the number of pedestrians that cross at a given time (3). The procedure to determine an adequate gap uses the following equation:

$$\text{Gap time} = (W/3.5) + 3 + (N-1)2 \tag{3}$$

where W is the width of roadway to be crossed, in feet, and N is the number of five-person rows crossing the street (rounded up). The 3.5 was the assumed walk speed in feet per second, 3 was the assumed perception or reaction time (in seconds), and 2 was the assumed time interval between rows (in seconds). Children were assumed to walk in rows of five, with a two-second headway between rows. The obvious weaknesses of the procedure lie in use of the assumed parameters and the assumed orderliness of crossing.

INTERIM MATERIALS ON HIGHWAY CAPACITY

Recent knowledge of how pedestrians react to space availability and to the presence of other pedestrians provided the impetus for developing a more-comprehensive procedure for analyzing pedestrian characteristics at intersections. Level-of-service descriptions allow one to determine the quality of pedestrian flow if volume and walkway characteristics are known.

Pedestrian Flow Characteristics

Pedestrian flow has been described in terms of speed, flow, density, and pedestrian space module (the inverse of density). As in highway-traffic-flow theory, these variables can be related to one another through the fluid-flow analogy:

$$q = k\,u \tag{4}$$

or

$$q = u/M \tag{5}$$

where

- q = pedestrian flow volume (pedestrians/foot-width of walkway/min);
- k = pedestrians per square foot of walkway;
- u = pedestrian space mean speed (ft/min); and
- M = pedestrian space module (ft^2/pedestrian).

Various researchers have collected data that relate these variables to one another. Figure 1 shows some of these findings in terms of speed versus density (7). Fruin (8) has also gathered data that relate the probability of a pedestrian being able to freely choose a path to the space module [see Figure 2 (8)].

The Interim Materials on Highway Capacity (4, pp. 115-147) include a section on pedestrians that incorporates these findings on pedestrian flow characteristics. The interim materials also recommend definitions for level of service for walkways [Table 1 (4, pp. 115-147)] and for queuing areas [see table below (4)].

Level of Service	Avg Pedestrian Area Occupancy (ft^2/person)	Avg Interperson Spacing (ft)
A	≥13	≥4
B	10-13	3.5-4.0
C	7-10	3.0-3.5
D	3-7	2-3
E	2-3	≤2
F	≤2	Close contact

A procedure for analyzing the performance of an intersection for handling pedestrian flow is then given.

Intersections Analysis with the Interim Materials Method

The interim materials method (4, pp. 115-147) considers two critical conditions for a street corner at a two-phase signalized intersection. Each condition would occur when the signal is changing to a phase that will allow pedestrians at the corner to begin to cross the street [see Figure 3 (4, pp. 115-147)].

In Figure 3, each approach is designated by the letters A, B, C, and D. A and B are sidewalk approaches. The subscripts of the volume vectors (V) identify the movement on each approach. The designation 1 in a subscript indicates pedestrians walking toward the intersection, and the designation 2 indicates pedestrians leaving the intersection. Total signal cycle length (TS), curb radius (r), cross time (CT), and queue time (QT) for each signal must be known. All volumes are for 15-min peaks only.

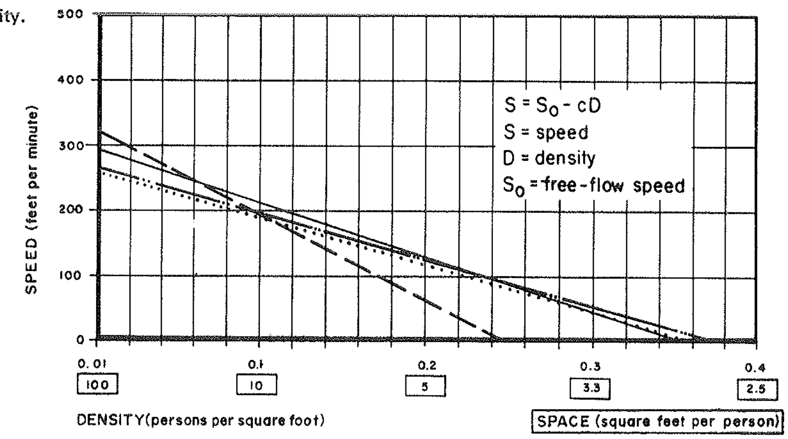
In step 1 the circulation areas (for pedestrians who are not waiting to cross) are computed for each condition. Step 1a involves conversion of 15-min pedestrian volumes for platooning (micropeaks within the 15-min design period).

In step 1b incoming pedestrian volumes V_{C1} and V_{D1} (people that reach the corner after crossing the street) are converted to peak volume. For example,

$$V_{D1(p)} = V_{D1} \times [TS/(CT_2 - 3)] \quad (6)$$

where TS is the total signal time (cycle length) in seconds, and $CT_2 - 3$ is the total cross time

Figure 1. Pedestrian speed versus density.



S_0 ft./min.	c	$\frac{S_0}{c}$ ped./ft²	$\frac{c}{S_0}$ ft²/ped.	PEDESTRIAN TYPES (RESEARCHERS)
268	714	0.36	2.77	.... SHOPPERS (Older)
267	722	0.37	2.70	——··——··—— COMMUTERS (Fruin)
295	835	0.35	2.83	———————— MIXED URBAN (Oeding)
320	1280	0.25	4.00	— — — — STUDENTS (Navin and Wheeler)

Figure 2. Cross flow traffic—probability of conflict.

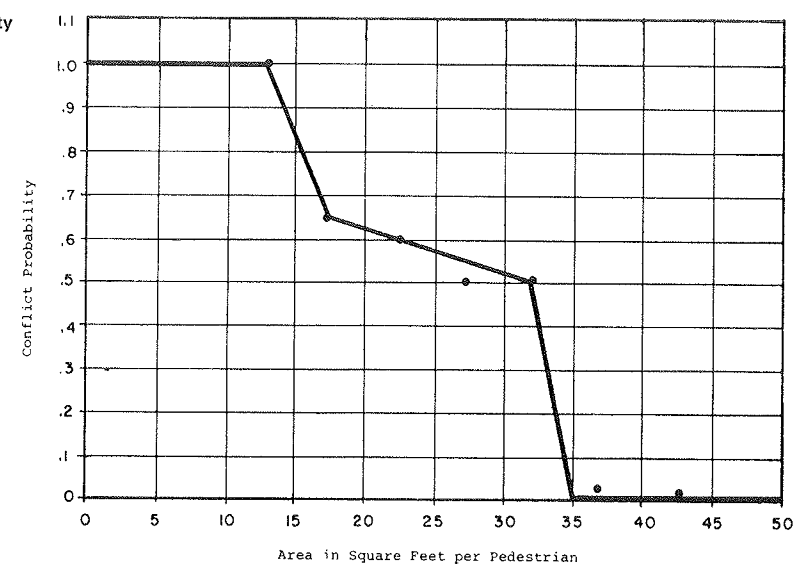


Table 1. Levels of service on walkways.

Level of Service	Space (ft^2/pedestrian)	Avg Flow Rate[a] (pedestrians/min/ft)	Mean Speed[b] (ft/min)	Volume/Capacity Ratio[c]
A	>40	<6	>250	<0.24
B	24-40	10-6	240-250	0.24-0.40
C	16-24	14-10	224-240	0.40-0.56
D	11-16	18-14	198-224	0.56-0.72
E	6-11	25-18	150-198	0.72-1.00
F	<6	0-25	0-150	0.00-1.00

[a]Flow rate relative to effective walkway width.
[b]Speeds are calculated based on space and flow rate variables.
[c]Assumed capacity is 25 pedestrians/min/ft.

Figure 3. Curb areas for pedestrian movements.

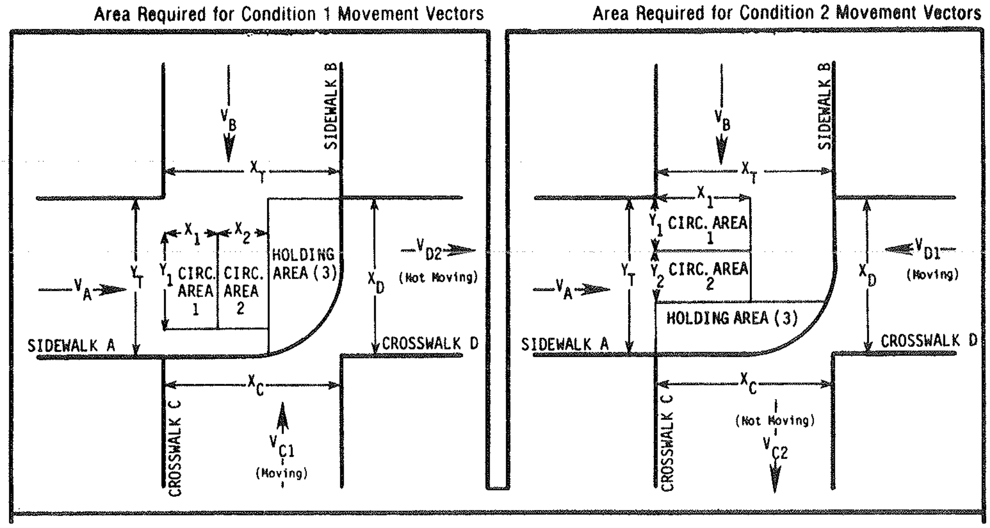
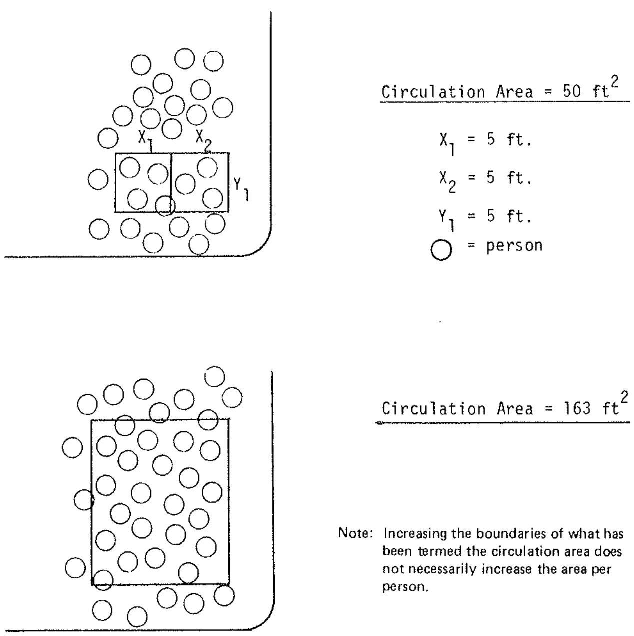


Figure 4. Effect of increasing circulation area.



(i.e., green time) less 3-s start-up delay. In the interim materials example,

$$V_{D1(p)} = 400 \times [80/(32 - 3)]$$

A flow of 400 pedestrians/15 min is converted to an equivalent 1100 pedestrians/15 min. The implicit assumption is that the pedestrian flow will be uniform for 29 s per 80-s cycle. For this to be true, the time required to cross the street would have to be zero and queued pedestrians would have to spread themselves out to achieve the uniform flow rate. The interim materials example does not include street width, but if one were to assume a width of 70 ft and a walk speed of 3.5 ft/s, it would take 20 s for a pedestrian to cross. With the 3-s start-up delay, a time band of only 9 s would be available for crossing (anyone waiting would have to leave the curb at between 3 and 12 s after the initial green indication in order to reach the opposite curb before the signal turned). Perhaps a more-appropriate pedestrian rate $[V'_{D1(p)}]$ would be

$$V'_{D1(p)} = V_{D1} \times [TS/(CT_2 - 3 - W)] \tag{7}$$

where W is the walk time to cross the intersection.

$$V'_{D1(p)} = 400 \times [80/(32 - 3 - 20)] = 3556 \text{ pedestrians/15 min.}$$

With the assumptions used, the equivalent flow rate would be more than three times that found through the interim materials method. (A later procedure in the interim materials for calculating needed crosswalk width is based on the same faulty reasoning.)

In step 1c effective walkway widths for circulation on the street corner are determined. In step 1d the number of pedestrians in the circulation area is determined. This information is then misused in step 1e, determination of circulation area requirements. In Figure 3 (4, p. 135) the variables X_1, X_2, and Y are determined. These define the area assumed to be available for circulations, $Y_1 \cdot (X_1 + X_2)$. Then, the number of people within the circulation area at a given time is found. In the interim materials example (4), X_1, X_2, and Y_1 all equal 5 ft. Therefore 50 ft² are available in the defined circulation area. The number of people found to be in the area is 6.8. To have a probability of conflict equal to 0.5, 24 ft² per pedestrian are needed. The interim materials then calculate the needed circulation area, A $circ_1$:

$$A\ circ_1 = 6.8 \text{ pedestrians} \times 24 \text{ ft}^2/\text{pedestrian} = 163 \text{ ft}^2.$$

This would indicate that the area available of 50 ft² is not sufficient (with 7.35 ft²/pedestrian, the probability of conflict equals 1.0). Instead of recognizing this, the interim materials tell one to see whether 163 ft² are available (4) (e.g., could the circulation area be 16.3 ft x 10). This ignores that, if the boundaries of the circulation area are increased, the number of people within the area is increased (see Figure 4). The remaining steps of the procedure involve determination of the holding (queuing) area required for people waiting to cross, then comparison of the total space requirements with the total space available at the corner.

ANOTHER APPROACH FOR ANALYZING PEDESTRIAN FLOW AT INTERSECTIONS

The purpose of this section is to (a) analyze how

Figure 5. Time required for herd to cross street at various levels of service.

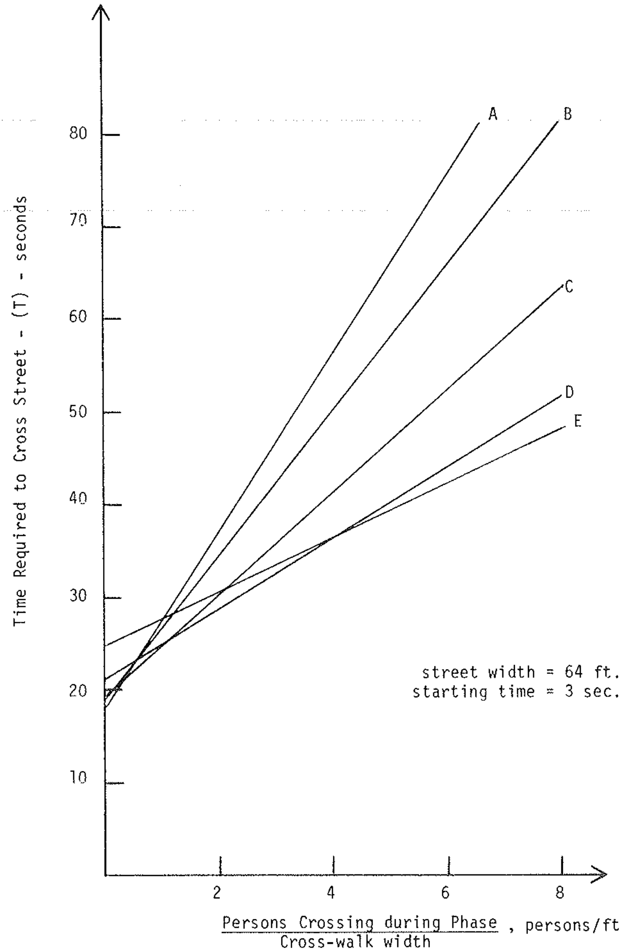
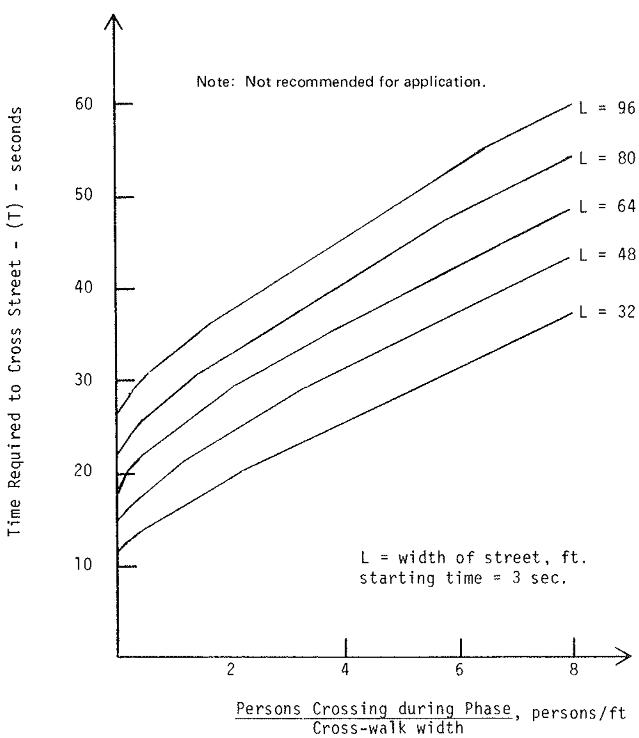


Figure 6. Time required for herd to cross street at optimal levels of service.



people are capable of crossing intersections and (b) shed some light on how one might determine space requirements on a street corner. A basic consideration is that a herd (or group) of people will be already queued up waiting to use a crosswalk. The herd will walk across at some average speed and some average density. If one knew the average speed and average density, then determination of the green time required would be a simple task.

$$T = t + (L/v_p) + P \tag{8}$$

where

- T = cross time, from when signal allows pedestrians to begin crossing until the last person clears the intersection,
- t = pedestrian starting time,
- L = length of the crosswalk,
- v_p = pedestrian walking speed, and
- P = time headway from front to back of herd.

Note that

$$ab = NM \tag{9a}$$

or

$$a = NM/b \tag{9b}$$

where

- a = length of herd,
- b = width of herd (effective walkway width),
- N = number of people in herd, and
- M = pedestrian module in herd.

Also,

$$P = a/v_p \tag{10}$$

Therefore,

$$T = t + (L/v_p) + [(NM/b)/v_p] \tag{11}$$

One might assume that people would select to walk at a combination of speed and density that one could expect to find on a normal walkway. Then, the pedestrian walk speed and module would be related to pedestrian level of service, as in Table 1. A graphical representation of crossing time versus number of people crossing per effective crosswalk width is given in Figure 5. The combinations of speed and module were taken from Table 1.

One would have to know the level of service to determine the appropriate crossing time. However, one might make a further assumption: The herd will select a combination of speed and density (that could exist on a normal walkway) in order to minimize the time it takes for the last person to cross the intersection. A graphical representation of this is given in Figure 6.

The relation between speed and density in Figure 1 can be used with Equation 11 to develop an expression for pedestrian module in the herd to minimize crossing time.

$$M = [C + \sqrt{C^2 + S_0 CL(b/N)}]/S_0$$

where S_0 is the free-flow speed and C is the negative of the slope in Figure 1, speed versus density. Equation 11 can then be used to find crossing time.

As the number of persons per foot of width of crosswalk increases, the lower level of service becomes more attractive for minimizing crossing time.

One might say that, for a large number of people crossing, the benefit of the high density (low module) of a poorer level of service more than outweighs the benefit of higher speed associated with a better level of service. As the length of the crosswalk increases, the break-even points (the points at which the poorer levels of service provide the lower cross time) move to the right, due to the greater importance of walk speed.

Some obvious problems with the above approach are as follows:

1. People may not choose to walk at a combination of speed and density that they would on a normal walkway,
2. People may not choose to walk at the optimum combination of speed and density, and
3. The presence of turning vehicles may disrupt pedestrian flow.

The lack of an optimum combination would tend to make Figure 5 overly optimistic. On the other hand, people may be willing to walk at a higher density for a given speed in a crosswalk than they would on a much longer walkway. This condition would be similar to the experience observed near highway on-ramps, where a particular lane can carry a volume higher than its expected capacity, apparently because people are willing to put up with a higher combination of speed and density for a short period of time. Further, the herd consists of a relatively small number of people. The people in front will not have their speed constrained by others. This may tend to reduce the time required for crossing.

The interim materials procedure (4, pp. 115-147) implicitly assumes that people would desire the best level of service possible. However, over a relatively short distance (e.g., 20-100 ft), pedestrian level of service might not be as relatively important to the pedestrian as would level of service over a much longer walkway. Further, some researchers (9,10) have reported higher average walk speeds at intersections and in the middle of city blocks than is indicated in Table 1 for level of service A.

Figure 6 is a coarse representation of how a herd is capable of negotiating a crosswalk. It should be thought of as a starting point for further investigation.

A related problem is providing a sufficient circulation area at a corner. The most-critical condition would occur when we have a herd of people just leaving a crosswalk and reaching the corner, some people seeking to use the same circulation area as will be used by the herd, and a queue waiting to use an adjacent crosswalk (e.g., V_{C1}, V_A, and V_{D2} in Figure 3). If the herd (V_{C1}) were at level of service E, then anyone wishing to cross the herd (for instance, someone from V_A) would be unable to do so. This problem might be slightly improved by increasing the effective width of the herd's path on reaching the corner. Also, since the herd would be using the space for a relatively short period of time, anyone wishing to cross the herd's path could wait until the herd has passed. If the herd were at a level of service better than E, the people wishing to cross the path might be able to weave their way through the herd. The most-severe problems would, of course, occur if one large group of people needed to cross the path of another large group.

Another related problem is providing a sufficient crosswalk width so that two herds that pass each other in the middle of the street would have sufficient space to avoid delay. If the effective crosswalk width is not increased, the assumed crosswalk width would not be appropriate when two relatively large herds pass. To deal with this problem, one would have to determine the needed reduction in effective crosswalk width due to the presence of the opposing herd. This might be done in proportion to the expected size of each herd. Otherwise, herds would be forced to walk outside of the crosswalk in order to reach the opposite curb within the signal phase.

SUMMARY

Recent studies of pedestrian movement can provide aid for dealing with pedestrian movement at signalized intersections. The procedure given in the Interim Materials on Highway Capacity (4, pp. 115-147) uses this relatively new knowledge of pedestrian movement, but faults within the procedure make it inappropriate for use. A different application of the principles of pedestrian flow was presented to provide a more-realistic starting point for the analysis of pedestrian flows at intersections and for ways to determine required walkway widths and lengths of signal phase. Still, some assumptions used might need to be modified when new information becomes available.

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Effects of Pedestrian Signals on Safety, Operations, and Pedestrian Behavior—Literature Review

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During the past 20 years, cities throughout the United States and Europe have installed different types of pedestrian signals in an effort to improve the safety and operational aspects of urban intersections. The purpose of this paper is to summarize the state of the art of pedestrian signals in terms of their effect on safety, operational impacts, and the behavioral aspects of pedestrians. Of the six studies reviewed on pedestrian signals and safety, only one attempted to analyze pedestrian accident data, but the small sample size and infrequency of pedestrian accidents prevented the researchers from making statistically sound conclusions. Other studies used compliance as a safety measure and generally concluded that pedestrian signals result in increased compliance and thus contribute to increased safety, although there was no conclusive proof of increased safety benefits. The five papers reviewed on operational impacts generally indicated that pedestrian signals will almost always increase pedestrian delay, and, at locations that have heavy vehicular volume, overall vehicular delay is also likely to increase. Several authors noted that pedestrians often jump the gun, regardless of the presence or absence of pedestrian signals. Concerning pedestrian signals and behavior, the literature generally indicated that (a) flashing signals were found to be no more or less effective than steady signals and (b) the presence of a clearance interval with a pedestrian signal tends to increase compliance rates. The studies also indicated that pedestrians are likely to ignore signals under low vehicular volume conditions, particularly when clearance intervals exceed the minimum.

Recent research in the area of pedestrian safety has uncovered a number of problems regarding pedestrian signalization alternatives. In some cases, signals installed have failed to command adequate attention of pedestrians. In other cases, they have failed to convey a clear meaning, and yet in others, the intent of the signal has been totally misinterpreted. As a result, questions have been raised by traffic experts regarding the effectiveness of signals in improving the safety and operational features of the intersection.

The current version of the Manual of Uniform Traffic Control Devices (MUTCD) provides general guidelines on the installation of pedestrian signals (1). According to MUTCD, pedestrian signals may be installed under the following circumstances:

1. The crossing is at an established school crossing,
2. The pedestrian cannot see the traffic signal,
3. An exclusive pedestrian crossing interval or phase is provided at one or more crossings,
4. Any volume of pedestrian activity requires the use of a pedestrian clearance indication to minimize pedestrian-vehicle conflicts and assist pedestrians in making a safe crossing,
5. Multiphase intersections cause confusion to pedestrians, and
6. Pedestrians cross part of a street to an island during an interval and will not have sufficient time to cross another part of the street.

MUTCD describes a total of eight signal warrants, four of which have direct or indirect pedestrian implications. These are the pedestrian volume warrant, the school crossing warrant, the accident warrant, and the combination of warrants. However, consideration of pedestrian factors in actual signal installation is not very common. For example, a study conducted for the National Cooperative Highway Research Program (NCHRP) found that, out of a sample survey, only 21.2 percent of the traffic signals were installed based on warrants that consider pedestrian factors and only 1.3 percent of the traffic signals were installed based on the pedestrian volume warrant (2).

The basic types of signal timing for pedestrian signals include (as provided in MUTCD)

1. Concurrent or standard timing (where the pedestrians walk concurrently with moving traffic),
2. Early release (where pedestrians are allowed to leave the curb before traffic is allowed to turn),
3. Late release (which holds pedestrians until a portion of the phase is given to turning vehicles), and
4. Exclusive pedestrian interval, where pedestrians have a protected crossing interval.

Scramble or Barnes dance timing is a form of exclusive timing where pedestrians are allowed to cross diagonally across an intersection.

In order to justify the installation and use of pedestrian signals, it is important to know their effects on safety, operations, and behavior. The use of accident data is considered the most desirable method of measuring the safety effectiveness of countermeasures. However, when accident data are inadequate or not available, indirect measures must be used. One promising method of assessing the safety benefits of pedestrian countermeasures is the use of behavioral observations. Nonaccident behavioral analysis has been used in the past for identifying unsafe pedestrian actions and for evaluating the effectiveness of pedestrian countermeasures (3). In a few other studies, researchers have attempted to evaluate the operational effects of pedestrian signals and have used different types of delay measures to determine whether pedestrian signals or different pedestrian signal-timing schemes will result in operational improvements.

The purpose of this paper is to review research studies on pedestrian signals to obtain a better understanding of (a) whether pedestrians signals improve pedestrian safety, (b) whether any operational improvements result from these signals, and (c) whether or not pedestrian signals result in better compliance.

Several criteria were used to review the articles related to pedestrian signals, including

1. Appropriateness of the analysis methods,
2. Adequacy of the data base used,
3. Validity of the conclusions reached, and
4. Overall applicability of the study results.

SAFETY IMPACTS OF PEDESTRIAN SIGNALS

A number of technical articles were reviewed in the general area of pedestrian safety and signals.

Fleig and Duffy

Fleig and Duffy, in a study in New York City during the early 1960s, examined behavioral data at a given intersection and limited accident data at a number of urban intersections before and after the installation of pedestrian signals (4). Pedestrian behavior, rather than accidents, was used as a primary measure of effectiveness of signals because of the problem of sample size and necessary lead time.

The authors identified a number of pedestrian actions or violations as unsafe acts and determined the trends in these unsafe acts before and after the installation of a pedestrian signal with a Barnes dance (scramble) phasing. For the accident study, they analyzed the pedestrian accident data at a total of 11 intersections one year before and one year after the installation of the pedestrian signals [see table below (4)].

	No. of Pedestrian Accidents	
Intersection No.	1 Year Prior to Signal Installation	1 Year After Signal Installation
1	7	4
2	2	4
3	1	0
4	2	1
5	3	1
6	2	4
7	3	0
8	1	3
9	1	1
10	3	3
11	2	4
Total	27	25

They found no significant reduction in the proportion of unsafe acts before and after the installation of pedestrian signals. Based on this evidence, the authors concluded that pedestrian signals are not an effective method for reducing pedestrian accidents.

The number of pedestrian accidents at the 11 intersections studied were reduced slightly (27 versus 25) after installation of the pedestrian signals. However, the small sample of intersections and the low number of accidents in the sample did not allow for a conclusive statistical analysis. This limitation was also recognized by the authors.

The use of several years of accident data, a large number of sites, and a better experimental design (i.e., use of randomized control sites, comparison sites, or trend analysis) would have greatly enhanced the experimental plan and could have resulted in some important findings relative to pedestrian signals and their effect on safety. Another way to have enhanced the study would have been to review each accident report carefully to omit any pedestrian accidents that are totally unrelated to the pedestrian signals. For example, accidents that are attributable to unrelated factors such as vehicle failure or drunk driving should be screened out in such an analysis. The authors do not report on any such screening effort. This study, however, is one of the few that attempted to analyze actual pedestrian accident data to assess the effectiveness of pedestrian signals. The study does not show any conclusive evidence about either a positive or a negative effect of pedestrian signals with respect to accidents.

Mortimer

Mortimer compared the compliance rates of pedestrian crossings at intersections with and without pedestrians signals (5). His methodology consisted of (a) identifying similar signal-controlled intersections with and without pedestrian signal (WALK and DON'T WALK) indications, (b) collecting data at these intersections on pedestrian compliance as well as on the incidence of successful completion of crossing, and (c) developing two types of hazard indices and other statistics on pedestrian crossings. Mortimer found that

1. Better signal compliance was found at intersections with pedestrian signals than at those without them;
2. Fewer illegal starts and more successful crossings were made at intersections with pedestrian signals than at those without pedestrian signals;
3. Hazard-index values calculated for intersections with pedestrian signals were slightly lower than those calculated for intersections without pedestrian signals;
4. Potentially serious pedestrian-vehicle conflicts were reduced substantially at intersections with pedestrian signals; and
5. The use of pedestrian crossings was instrumental in improving compliance and in providing more information to pedestrians, which resulted in more comfortable crossings and fewer crossing hazards.

This study provides some useful information regarding pedestrian compliance, specifically in comparing intersections with and without pedestrian signals. However, without any known quantifiable relation between pedestrian compliance and accidents, the true effects of pedestrian signals on safety (i.e., pedestrian accidents) cannot be determined.

Skelton, Bruce, and Trenchard

Skelton, Bruce, and Trenchard, in a study related to the effectiveness of pelican crossings, conducted surveys at a number of sites in the city of Newcastle-upon-Tyne and in a town in rural Northumberland, England (6). Pelican crossings are pedestrian-actuated crossings, used extensively in England, Australia, and some European countries, in which the pedestrian phase is initiated by a pedestrian push button. Zebra crossings are crossings that have alternate black and white stripes and are occasionally marked with flashing beacons. The study did not analyze any accident, operational, or compliance data but mainly focused on an opinion survey among pedestrians on the understanding and effectiveness of pelican crossings. The study concluded that the general public (pedestrians as well as drivers) lacked understanding of the way in which pelican crossings were designed to function. The study recommended that, if the potential of the crossing devices are to be fully realized, significant operational and design improvements must be made.

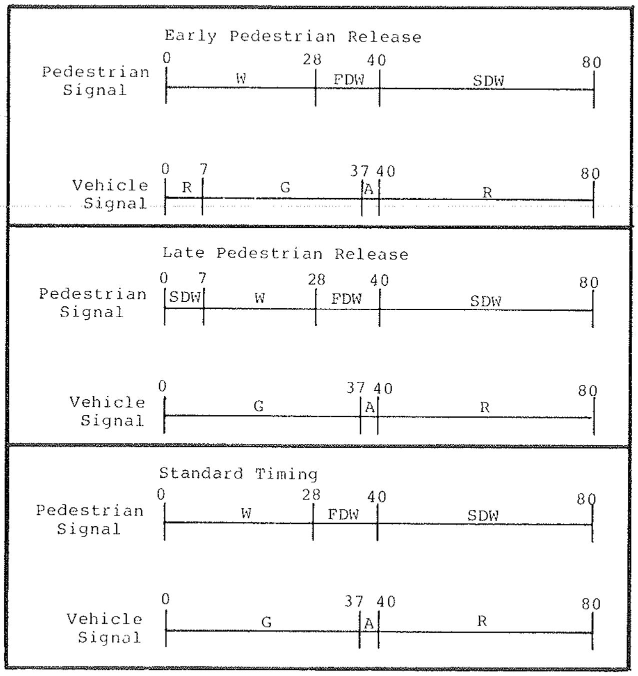
The above study cannot necessarily be categorized as a safety study, since it does not deal with any accident or compliance data. However, it provides information relative to the effectiveness of new or innovative control devices and public acceptability. The study suggests that adequate publicity and appropriate placement are necessary prerequisites to the successful use of any new control device. The message of these devices must be properly received and understood by the motorists and pedestrians if the intended purpose is to be achieved.

Abrams and Smith

Abrams and Smith (7), in their effort to address the safety (and delay) aspects of pedestrian signals, analyzed three types of signal phasing (i.e., early release, late release, and scramble timing) [Figure 1 (7)]. The authors performed compliance studies in Sioux City, Iowa, and concluded that

1. The early release of pedestrians may provide a measure of additional safety, but the benefits were not precisely determined;

Figure 1. Timing used in the analyses of early and late release of pedestrians.



Note: W = WALK interval; FDW = flashing DONT WALK interval; SDW = solid DONT WALK interval; G = green indication; A = amber indication; and R = red indication

2. Higher compliance rates associated with the late-release technique are indicative of increased pedestrian safety; and

3. Scramble timing has the capability of increasing pedestrian safety by completely eliminating pedestrian-vehicular conflict; however, violation rates for scramble timing were found to be higher, particularly at narrow streets.

The authors' postulated association between safety and compliance appears to be based primarily on judgmental factors as opposed to any specific data analysis. The higher violation rates at scramble-timed intersections (i.e., where an exclusive pedestrian phase exists with diagonal crossings permitted) is indicative of the higher pedestrian delay generally associated with these locations. This study provides useful information regarding pedestrian behavior and compliance relative to various pedestrian signal-timing schemes (i.e., early release, late release, standard, and scramble timing), which may or may not be indicative of pedestrian safety.

Williams

Williams discussed the evolution of the pelican concept in England and in Australia and summarized the findings and experiences of different researchers about safety, operation, and behavior (8). The discussion is primarily oriented toward a comparison with its predecessor, the zebra crossing. The author mentions that uncontrolled zebra crossings, originally introduced in 1951, were reported to cause delay and congestion in heavy vehicle and pedestrian flows. Pelican crossings appeared to present considerable advantages over zebra crossings. Williams also mentions that at least one study in Australia found that accidents decreased by 60 percent at a sample of pelican crossings that were originally zebra crossings.

Based on these findings, Williams suggests that it is not possible to definitely conclude that pelican crossings significantly increase pedestrian safety. He mentions that, in most of the sites where positive safety benefits were indicated, the results appear to be masked by the presence of other factors. A number of other countermeasures were installed at these sites (e.g., antiskid surfacing and guardrails), and the effects of these treatments are very difficult to isolate from the overall safety effect of the pelicans. Although the studies reported by Williams do not provide conclusive evidence of the positive safety benefits of pelican crossings, there was also no indication of any adverse effect of the devices.

Inwood and Grayson

Inwood and Grayson, in a study conducted for the Transport and Road Research Laboratory, England, analyzed injury accident data, pedestrian counts, and vehicle flows for lengths of road on and near pedestrian crossings (9). The prime objective of this study was to compare pedestrian accident rates at zebra and pelican crossings. The candidate crossings were located in similar conditions at sites throughout England and were selected on the basis of good visibility and not being too close to busy intersections. The study showed no evidence of a difference in pedestrian accident rates between pelican and zebra crossings. However, pelican crossings tended to have a lower total injury accident rate than zebras when the road length in the vicinity of the crossings is taken into account.

It appears that push-button signals (pelicans) are not any more effective than pavement markings with flashing beacons (zebras) in reducing pedestrian accidents. However, when injury accidents are considered, pedestrian-actuated signal crossings are more effective than zebra crossings.

OPERATIONAL IMPACTS OF PEDESTRIAN SIGNALS

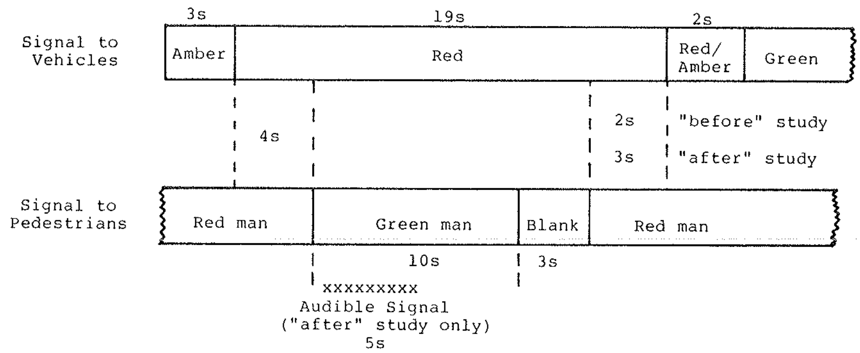
The impact of pedestrian signals on traffic operation at or near urban intersections has been studied by a number of researchers. Traffic engineers, in particular, have been concerned about the possible effect of pedestrian signals on delay to pedestrians and motorists and on intersection capacity.

Abrams and Smith

Abrams and Smith evaluated the delay effects of three types of pedestrian signals--early release, late release, and scramble timing--relative to standard (concurrent) combined vehicle-pedestrian interval (7). They used time-lapse photography to record events and computed delay from the recorded data. For this study, delay was defined as "the difference between the time required for a right-turning movement with pedestrians in the crosswalk and the time required for a right-turning movement without pedestrians in the crosswalk."

The study showed that the standard (concurrent) pedestrian-vehicle interval will almost always result in lower overall pedestrian and vehicle delay than will other pedestrian signal-timing schemes (i.e., scramble, early release, or late release). The only exception to this occurs in cases of long queues of vehicles in a right-turn lane (or left-turn lane of a one-way street) caused by pedestrian-vehicle conflicts. The specific conclusions were that the early-release technique always increases total intersection delay. The late-release technique may result in a reduction of total intersection delay only under certain combinations of volume

Figure 2. Signal phasing diagram used by Wilson.



and geometrics. Scramble timing always increases pedestrian delay.

Pretty

Pretty analyzed the relative delays to pedestrians and vehicles with two methods of signal-timing schemes: (a) exclusive pedestrian phase (scramble timing) and (b) shared-phase (concurrent) with motor vehicles (10). He used a deterministic numerical technique [developed by Miller (11)] that is commonly used in Australia to compute bicycle crossing intervals, signal settings, and delays. He estimated pedestrian and vehicle delays for varying cycle lengths that corresponded to the two types of signal control timing schemes. Pretty also assumed that pedestrians arrive at a uniform rate throughout each cycle and that the number of pedestrians desiring to cross both streets is twice the number that cross one street. His assumption of pedestrian arrivals contradicts assumptions made by some researchers in the United States.

The numerical examples presented by Pretty do not lend themselves to a direct comparison between the two types of control. It appears from the results presented that scramble timing increases both pedestrian and vehicular delay significantly although the signal parameters (e.g., cycle length) analyzed in the two cases are different. The author does not address the question of whether the differences in the total intersection delay are due to the types of control or to the signal parameters. Pretty also shows that pedestrians are always likely to benefit from shorter cycle lengths (due to reduced pedestrian delay) and that increased pedestrian volume significantly increases pedestrian delay.

Smith

Smith discusses problems associated with the lack of consistency in the timing of pedestrian clearance intervals as well as different phasing schemes (12). He computed both the vehicle right-turn delays and pedestrian delays for two hypothesized timing schemes: (a) minimum clearance alternative (i.e., the clearance interval is only long enough to cross the street that provides a longer WALK interval) and (b) minimum WALK alternative (i.e., the WALK interval is only a few seconds long and a longer flashing DON'T WALK interval exists) (12).

Vehicular right-turn delay was computed by using a relation that was developed from data collected for 68 h of time-lapse photography at intersection approaches in Washington, D.C.; Phoenix, Arizona; Akron, Ohio; and Cambridge, Massachusetts. Pedestrian delay was calculated by using a bilevel arrival rate with the assumption that such arrivals are highest during and just prior to the WALK interval and approximately half that rate following the WALK interval.

Smith's study showed that the minimum WALK alternative reduces vehicle right-turn delay because of no interference between pedestrians and vehicles after the initial platoon of vehicles has crossed the street. Smith also concluded that the increase in pedestrian delay of the minimum WALK alternative over the others was significantly greater than the decrease in vehicle right-turn delay. He concluded that clearance intervals longer than the minimum generally increase overall intersection delay.

Wilson

Wilson, in his study conducted at the Transport and Road Research Laboratory, England, assessed the operational and behavioral effects of installing an audible signal for pedestrians at intersections that have pedestrian indications (13). The concept of an audible pedestrian signal has been introduced in recent years as a possible aid to blind or visually impaired pedestrians. A speaker, attached to the pedestrian signal, emits a beeping, buzzing, or chirping noise during certain signal phases to supplement the visual pedestrian display. The signal phasing diagram used by Wilson is shown in Figure 2 (13). He used time-lapse photography to record adult pedestrian crossings at a signalized intersection before and after the installation of the audible signal. Wilson's major conclusions are as follows:

1. Pedestrian delay at the curb was not affected by the installation of the audible signal;
2. Time taken to cross the road by pedestrians crossing during the "green man" phase decreased by 5 percent;
3. For those pedestrians who started to cross during the green man phase, a significant reduction was obtained in the proportion who failed to complete their crossing before the vehicle green signal began; and
4. Significant differences in pedestrian behavior and delay were observed between the before and after data that seemed to be indicative of positive safety effects of audible signals.

Audible signals might be particularly beneficial to visually handicapped pedestrians. On the other hand, these signals may also cause potential confusion to pedestrians regarding which direction to cross. One argument against the use of audible pedestrian signals for handicapped pedestrians is that, unless they are used everywhere, they may cause more problems than they solve, since the blind cannot always count on having the audible message at every intersection. The paper by Wilson points out small but statistically significant reductions in delay to nonhandicapped pedestrians.

PEDESTRIAN SIGNALS AND BEHAVIOR

The possible effects of pedestrian signals on the behavior of pedestrians and motorists have been a topic of research among traffic engineers and psychologists for a number of years. The aspect of such behavioral studies is of importance to many safety analysts because, in the absence of sufficient accident data, behavioral changes associated with pedestrian signals may often be regarded as indicative of safety benefits or disbenefits.

Williams

Williams discussed pedestrian behavior relative to pelican crossings in England (8). His paper included a review of numerous papers on the subject. Based on his findings, Williams makes the general conclusion that pedestrians tend to accept natural gaps in traffic rather than wait for the signal to provide a protected crossing interval. This behavior may not be harmful if pedestrians accept only safe gaps. However, unnecessary motorist delays (and unnecessary risks to pedestrians) may be caused by pedestrians who cross on red signals after activating the pelican signal.

Smith

Smith discussed the importance of compliance to signal indications by pedestrians and suggests that the purpose of a pedestrian clearance interval is likely to be defeated if such clearance intervals are longer than the minimum required intervals (12). Studies were performed at two intersections each in the cities of Washington, D.C.; Phoenix, Arizona; and Buffalo, New York, to determine pedestrian compliance to a flashing DON'T WALK interval that was longer than the minimum clearance. At each intersection several timing schemes were installed, which ranged from the minimum clearance interval to long clearance intervals, and compliance data were collected.

The data showed a trend of lower compliance (lowest percentage begins to walk during the WALK interval) for those timing alternatives that have the least amount of time allocated to the WALK interval (longer clearance intervals). Pedestrians appeared to show a higher degree of disregard for flashing DON'T WALK clearance intervals that are longer than the minimum. The author states that the reason for the decrease in compliance for clearance intervals longer than the minimum appears to be that average pedestrians are not fooled into thinking they have less time to cross the street before vehicles in the cross street are released.

Based on these results, pedestrian signals should generally be set with the minimum clearance interval and the WALK interval should not be less than some minimum period. Of course, when setting any clearance interval, care should be taken to allow adequate time for slower-than-average walkers (i.e., elderly and handicapped pedestrians).

Robertson

Robertson, in a paper that was developed as a part of a Federal Highway Administration (FHWA) study on pedestrian safety, reported on user preference and understanding of symbol displays (as opposed to word messages) and on the field testing and evaluation of these displays (14). Five preference surveys were conducted: two of traffic engineers and safety experts, two of pedestrians in 12 cities, and one of school children.

The author discusses different conceptual forms of symbolic signal displays and presents the result of each preference survey with appropriate details. The data show a great deal of difference in opinion and response to symbols and colors among engineers, adult pedestrians, and school children. Overall, the walking man symbol (WALK phase) message and the hand signal (DON'T WALK interval) were recommended.

Kyle

Kyle attempted to evaluate the effectiveness of dynamic pedestrian signals in controlling pedestrian movements (15). A major difference between a conventional signal and the dynamic signal is that a conventional signal is likely to change to DON'T WALK while pedestrians are in the crosswalk. The dynamic signal allows the pedestrian to see the WALK indication the entire time he or she is crossing.

In his study, Kyle used a before-after experimental design in which pedestrian observation data were collected at two experimental and two control intersections. Time-lapse photography and manual counting methods were used to record pedestrian movements at the candidate locations in the Champaign, Illinois, urban area. Kyle's study showed that the dynamic pedestrian signal tended to reduce the number of illegal pedestrian movements in the intersection area. A greater percentage of pedestrians crossed during the clearance interval in the after phase when the dynamic signal was in operation than during the before phase. However, other problems encountered with the mechanics of dynamic signals hampered their use (15).

Stoddard

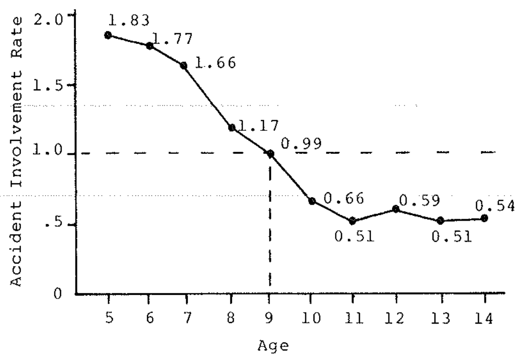
Stoddard also conducted a study, similar to that of Kyle, to assess the effectiveness of dynamic pedestrian signals in controlling pedestrian traffic (16). Two types of analyses were conducted. First, a comparison of before and after reactions was conducted by using a pedestrian compliance count at a specified intersection. Second, pedestrians were interviewed to determine pedestrian reaction to the new type of signal. A total of 558 pedestrian interviews were conducted two months after the new signals were installed.

The study showed that a significant number of pedestrians were cleared from the crosswalk that had the dynamic signal, and the author recommended that this type of pedestrian control would be appropriate for intersections where the pedestrian interval is short or the crosswalk distances are relatively long. The interviews showed that only a small percentage of the pedestrians are likely to be confused by the new signal (16).

Retzko and Androsch

Retzko and Androsch studied pedestrian behavior at signalized intersections in Dusseldorf and a few other cities in the Federal Republic of Germany (17). The authors investigated pedestrian behavior at a number of signalized intersections with and without an amber phase in the pedestrian signals. Data were collected on pedestrian walking patterns and at 24 crosswalks of similar geometrics for a total of 5000 cycles during 1972-1973. The authors found that the presence of an amber phase generally resulted in better pedestrian compliance. Furthermore, in the absence of an amber phase, pedestrians tended to walk against the red. Based on this finding, the authors recommended the installation of an amber phase (clearance interval) for pedestrian signals.

Figure 3. School trip pedestrian accident involvement rate of students by age.



Based on 1,910 school trip accidents.

Sterling

Sterling attempted to quantify pedestrian reaction to flashing WALK as well as the steady WALK indications (18). He describes two measurable aspects of pedestrian attributes as reflective of pedestrian behavior:

1. Observation rate--the percentage of legal crossings and
2. Conflict rate--the percentage of crossings with specifically defined interruptions.

The quantification of these variables was used to develop conclusions with respect to pedestrian reaction regarding the flashing WALK and steady WALK intervals.

Sterling collected pedestrian behavior data at locations that have a high concentration of pedestrian and vehicular volume during twelve 1-h periods. In virtually all comparisons, the reaction to flashing WALK was less favorable than that to steady WALK. Although the percentage difference in conflict rate is not so drastic as in the compliance rate, the effectiveness of flashing WALK signals appears questionable from these results. The specific conclusions of this study are that a significantly higher percentage of legal crossings occurred with the steady WALK as compared with the flashing WALK. A significantly higher percentage of illegal conflict crossings occurred with the flashing WALK than with the steady WALK.

The results of this study point out the general misunderstanding of the flashing WALK (or flashing man) indication as a warning to pedestrians to watch for turning vehicles. Many states still do not use the flashing walk concept either because they have reservations about its value or because some of their signal hardware is not easily adaptable to a flashing mode.

Jennings and Others

Jennings and others studied pedestrian behavior at a number of signalized locations that had experienced a large number of pedestrian accidents in the City of Portland (19). The authors used video recording techniques to observe pedestrian behavior at signalized intersections. They found that pedestrian behavior could be described in terms of its unsafe aspects:

> Numerous pedestrians do not obey the DON'T WALK signal. Numerous pedestrians do not look in the presence of either a WALK or DON'T WALK signal before crossing the street. Moreover, the pedestrians who do not stop also do not look. In short, there are a reasonable number of pedestrians who do not appear to assess the traffic situation before crossing the street.

This study did not directly address the question of behavioral changes associated with the addition of pedestrian signals, but it provides useful information. The study indicates that unsafe behavior is associated with intersections that experience high frequencies of pedestrian accidents. However, the above inference can be questioned, since the authors did not report on any effort to collect similar behavioral data at intersections with little or no history of pedestrian accidents and did not test how pedestrian behavior at these intersections compared with behavior at the original intersections studied.

Reiss

Reiss discussed the behavior of young pedestrians (ages 5-14) during street crossings for typical school trips (20,21). Students in the eastern United States were observed walking to school and were then surveyed regarding their behavior and the underlying knowledge associated with their habits as pedestrians. By using accident and age distribution data collected by the American Automobile Association, Reiss showed that (20, 21) "there is a near-monotonic relationship between age and accident involvement rate for the 5 to 14 year old population." The youngest students are considerably over-represented in the school trip accident data, as illustrated in Figure 3 (20).

Reiss' study shows that, with an increase in age, a greater proportion of the students will cross with the green signal. This increased knowledge of traffic control devices with student age closely matches the decreasing rate of student involvement in accidents. Further, students' propensity toward taking risks may increase with age. However, as the accident data indicate, this may be offset by improved knowledge and ability to interpret the signal indication with increasing age and by an increased ability of the matured pedestrians to take evasive actions in cases of an approaching vehicle.

Robertson

Robertson analyzed pedestrian behavior, compliance, and understanding for different types of word messages (22). The author reports on three experiments conducted that included (a) comparison of steady DON'T WALK to flashing DON'T WALK, (b) comparison of DON'T START with DON'T WALK, and (c) comparison of steady WALK with flashing WALK messages. All three experiments were conducted simultaneously in Buffalo, New York, and Phoenix. A before and after study design was employed to conduct the experiments. It was found that

1. A steady DON'T WALK clearance display appears to have the same effectiveness as a flashing DON'T WALK clearance display. Evidence is not sufficient to conclude that a steady clearance is better than a flashing clearance.
2. The DON'T START message offers little or no improvement over the current DON'T WALK message.
3. A flashing WALK is not an effective means of warning pedestrians about turning vehicles.

CONCLUSIONS

Research in the area of pedestrian safety has gained considerable prominence over the last decade. Pedestrians have historically accounted for a large

Table 1. Summary of pedestrian signal studies in the safety area.

Authors	Location of Study	Use of Accident Data	Use of Compliance Data	General Conclusion
Abrams and Smith	Sioux City, Iowa	No	Yes	Improve compliance observed since installation of pedestrian signals
Mortimer	Eastern Michigan University	No	Yes	Decrease in conflict, illegal starts, and hazard-index values since the installation of pedestrian signals
Fleig and Duffy	New York	Yes, limited	Yes	A small reduction in pedestrian accidents at 11 intersections does not provide statistically reliable conclusions; no significant reduction in unsafe acts noticed
Inwood and Grayson	England	Yes	No	No significant difference in pedestrian accidents between zebra and pelican intersections
Skelton and Trenchard	England	No	No	Opinion survey indicated a lack of understanding of operating characteristic of pelican crossings
Williams	England and Australia	Yes	No	General reduction in pedestrian accidents observed with installation of pelican signals; however, presence of other countermeasures make it difficult to isolate the effect of pelican signals

number of highway fatalities. The occurrence of most of these pedestrian fatalities at or near urban intersections has led traffic experts to believe that the use of pedestrian signals would improve pedestrian safety. A number of cities have experimented with this concept and have installed different types of pedestrian signals that have included word messages, symbolic messages, flashing and steady signal indications, and even audible messages for pedestrians.

The overall purpose of this paper was to ascertain exactly what is known regarding the effects of pedestrian signals on (a) safety, (b) operation, and (c) behavioral aspects of pedestrians.

Pedestrian Signals and Safety

Six papers were reviewed that addressed the question of the relation between safety and pedestrian signals. Three of these were related to experiences in the United States, and the other three were on the experience of pelican crossings in England and Australia. Critical features of these studies are summarized in Table 1. This table shows that only one study attempted to analyze accident data [Fleig and Duffy (4)], but data limitations prevented the researchers from obtaining statistically sound results. If pedestrian compliance is indeed a true measure of safety (as postulated by many researchers), then pedestrian signals could possibly contribute to increased pedestrian safety. However, none of the studies in the literature developed a quantifiable relation between pedestrian accident experience and pedestrian behavior and compliance.

To some extent, experiences with pelican crossings in England and Australia reveal similar trends. Again, the nonavailability of accident data posed major problems for the researchers. None of the studies showed indications of adverse safety effects of pelican crossings. However, in cases where definite positive effects were discerned (after the installation of pelican crossings), it was difficult to isolate the singular effect of pelican crossings from other countermeasures installed. The overall general conclusion that can be made from these studies are that, although there are indications from compliance and behavior data that pedestrian signals could be beneficial in some instances, there is no conclusive evidence from the literature to support the contention that pedestrian signals increase pedestrian safety. The lack of understanding and uniformity of pedestrian signals may be one of the reasons for the apparent lack of their effectiveness.

Pedestrian Signals and Traffic Operation

Five papers were reviewed that related to the effect of pedestrian signals on traffic operations, of which two were based on studies conducted in the United States, two in England, and one in Australia. The content and coverage of these papers were, however, somewhat varied in nature.

These reviews showed that pedestrian signals are almost always likely to increase pedestrian delay, and in some instances, depending on the vehicular volume and the signal parameters, overall vehicular delay is also likely to increase. Little effort is indicated in the literature regarding equitable allocation of delay among pedestrians and motorists. In particular, the question of how to treat pedestrian delay (relative to motorists who are more comfortably seated within the enclosed environs of the automobile) has received insufficient research attention.

A number of the authors indicated that pedestrians often attempt to get a head start by crossing against a red signal, a maneuver that is associated with higher risk. Thus, the general conclusion to be drawn is that pedestrian delay is likely to increase with the installation of pedestrian signals; and in many cases, vehicular delay is also likely to increase.

Pedestrian Signals and Behavior

The question of behavioral changes is a topic of considerable interest to psychologists and safety researchers. Traffic experts have been interested in this topic primarily because of a possible relation between pedestrian behavior and safety. A total of 10 papers were reviewed on this topic, of which 7 related to experiences in the United States, 1 in England, 1 in Germany, and 1 in Australia. The following conclusions can be drawn:

1. Under low vehicular volume conditions, pedestrians are likely to ignore signal indications, particularly when the clearance interval is longer than the minimum. Pedestrians have a general ten-

dency to accept natural gaps in traffic.

2. The compliance rate for steady WALK signals is higher than that for flashing WALK. Overall, the compliance rate for flashing signals appears to be lower than that for steady signals.

3. Students' propensity toward risk (in crossing streets) increases with age; however, their greater ability to interpret signal indications and to take protective measures in unsafe situations may offset the effect of risk. Accident experiences are lower between about 14 and 60 years of age.

4. The presence of a clearance interval in a pedestrian signal tends to increase compliance rates.

Recommendation for Further Studies

Further research efforts are necessary to provide a more complete answer to the many questions on pedestrian signals.

Create Comprehensive Pedestrian Accident Data Base to Study Effects of Pedestrian Indications

Limited information is reported in the literature regarding actual pedestrian accident data to answer safety-related questions. Some studies have analyzed accident data at individual intersections, but the data base used in these studies is too small to permit the development of any general conclusions. At the level of individual intersections, pedestrian accidents are rare events, notwithstanding that these accidents constitute alarming proportions in the context of all urbanized intersections.

Creation of a larger pedestrian accident data base from different cities that have different types of pedestrian signals and different geometric, operational, and traffic characteristics is considered to be the first critical step in addressing safety-related questions. Next, an analysis should be conducted to extract the effect of various extraneous factors, following appropriate experimental design procedures, so that the net effect of different types of signals can be ascertained.

Establish Relation Between Compliance and Safety

Most researchers have postulated that increased compliance is indicative of improved safety. Although the above hypothesis appears reasonable and logical, further studies are necessary to establish (in quantitative terms) a specific relation between these two factors. Such relations, once developed, may be used effectively by traffic experts and researchers to evaluate more accurately the safety effects of different pedestrian signal modifications and other possible pedestrian safety measures.

Establish Relation Between Pedestrian Behavior and Safety

Several studies have used pedestrian behavior as a measure of safety effectiveness. These studies have used various definitions of unsafe behavior. However, no research has indicated whether any relation exists between pedestrian accidents and pedestrian behavior measures.

Allocate Delay Among Pedestrians and Motorists

The literature review indicates that pedestrian signals will generally result in increased pedestrian delay and often also in increased vehicular delay. If a delay-based signal warrant is to be developed, a prerequisite to this step would be the development of a procedure to allocate delay equitably among pedestrians and motorists. This allocation must also be sensitive to the differential exposure consideration of these two groups (i.e., pedestrians exposed to weather and motorists within the enclosed environment of the automobile).

Define Tolerable Delay

It is not known what constitutes tolerable delay to the average pedestrians (i.e., the maximum waiting time prior to the crossing of the street) before pedestrians might accept unsafe gaps in the main traffic stream. Again, such information will be helpful in the development of a delay-based warrant, particularly at nonsignalized intersections or at midblock crossings.

Develop Optimum Clearance Interval for Pedestrians

The literature review revealed two interesting trends:

1. Clearance intervals generally increase compliance rates and
2. Under low-volume conditions pedestrians are likely to ignore signal indications when clearance intervals are exceedingly long.

Thus, there is a threshold value of clearance interval beyond which pedestrians will accept natural gaps in the traffic stream. If the concept of clearance interval is to be used effectively, one must define this optimum value and develop signal-timing patterns around this optimum value.

Identify Best Signal Indication

Currently, different types of pedestrian signal indications are used in the United States (e.g., word messages and symbols) in different operating modes (e.g., flashing or steady). The literature review did not indicate clearly which combination of pedestrian signals is the most effective in commanding attention of the pedestrians and in increasing their compliance rates. Many studies have indicated a general misunderstanding on the part of pedestrians about the meaning of pedestrian indications. Further studies are needed to answer this question so that local traffic agencies can implement a more uniform signal indication.

Develop Improved Indications to Warn of Potential Conflicts to Pedestrians and Motorists

Some research has been completed in the area of warning of potential conflicts between pedestrians and motorists, but little success has been achieved in developing a clear and effective device to warn pedestrians and motorists of potential conflicts. The flashing WALK indication is currently used in some jurisdictions to warn pedestrians of turning vehicles, but it is not used everywhere. Studies have shown a general misunderstanding of the meaning of the flashing WALK indication.

Possible alternatives to minimize this problem may include more uniformity in signal use, better education of the meaning of the signal indications, and active warning devices for pedestrians, motorists, or others.

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Transportation Control Measure Analysis: Bicycle Facilities

SUZAN A. PINSOF

The Clean Air Act Amendments of 1977 require that areas that have air quality problems examine their transportation system and implement measures to reduce automobile emissions. One of these measures is the improvement of bicycle facilities. The purpose of this paper is to determine the air quality impact and cost-effectiveness of bicycle facilities as a transportation control measure (TCM) for air quality in northeastern Illinois. A case study, based on a survey of the users of commuter bicycle parking at a commuter railroad station and a rapid transit station in a Chicago suburb has been used to determine these impacts. In addition to the air quality benefits and associated costs of current levels of commuter bicycling in Wilmette, some theoretical benefits and costs are calculated by extrapolation from the survey data and application of available ridership information for one of the stations. From the current level of bicycle trips to the station and the theoretical limit of potential trips, a range of possible emission reductions and costs are calculated. Actual potential air quality benefits and costs lie somewhere within this range. Bicycle facilities are implemented locally and bicycling activity varies considerably from one community to another. For these reasons, the impacts of bicycling are best considered at the local scale. For comparison to other TCMs, cost-effectiveness figures can be used. Even fairly expensive bicycle support facilities are found to be very cost effective for air quality improvement in relation to other measures. Each TCM must be evaluated for its socioeconomic as well as its air quality impact. This evaluation is also presented. Bicycle facilities are found to have few socioeconomic drawbacks.

Bicycle facilities are one of the transportation control measures (TCMs) identified by the Clean Air Act Amendments of 1977 for evaluation as a technique to decrease dependence on automobiles and thereby improve air quality. The Clean Air Act of 1970 requires that each TCM be evaluated for its feasibility for use by regions that have air quality problems. The common measure of feasibility used in northeastern Illinois (a six-county area that surrounds and includes Chicago) is the cost per ton of pollutant eliminated by a TCM. This report evaluates the cost-effectiveness for air quality improvement of bicycle facilities for commuters at transit stations. In addition, a socioeconomic impacts assessment required for all TCMs is included.

The analysis of air quality benefits potentially attributable to bicycling depends on estimates of existing and potential bicycling patterns in the region. Bicycle trips neither cause nor decrease air pollution--only when bicycle trips divert trips from other modes, primarily automobiles, can air quality improvements be credited to them. Buses cause pollution too, but an increase in bicycle use would have to be great before it caused a reduction of bus services. Therefore, it is the shift from automobiles to bicycles that must be quantified.

Transportation surveys have generally neglected to collect information on bicycle travel in this region, as well as elsewhere (1, p. 10). More specifically, the U.S. Environmental Protection Agency's (EPA) Bicycling and Air Quality Information Document, notes only one study (of a bicycle bridge in Eugene, Oregon) in which the modal shift from car to bicycle associated with a bicycle measure was quantified (1, p. 70). One other example is the Chicago Area Transportation Study's (CATS) analysis of new storage facilities installed by the Illinois Department of Transportation at commuter rail stations (2).

These studies represent the most direct way to forecast potential demand for bicycle facilities and resulting air quality improvements. The ideal study to forecast potential demand for bicycle facilities would combine the measurement of actual changes in use in response to new facilities with surveys that question users about mode change. If controlled for other variables, this information could then be generalized to similar locations where facilities are being planned.

In northeastern Illinois, general information on bicycle use was calculated in 1978 by the Northeastern Illinois Planning Commission (3). This study provides an estimate of regional bicycle use and potential modal shifts. By using a method developed by Carl Ohrn for Barton-Aschman Associates, it was determined that 10.4 percent of all home-based trips could be attracted to the bicycle if a complete grid of bikeways and appropriate support facilities were provided. By applying regional data developed by CATS, an estimate can be made of automobile vehicle miles of travel (VMT) that could be diverted to bicycles.

There are two major problems with this approach. First, Ohrn's percentages assigned to potential bicycle use are based on a comprehensive grid system of class 1 and 2 bikeways in the region. Such a system would be technically infeasible as well as economically and politically impossible in many parts of the region. Also, the idea that bikeway construction is generally the best way to encourage bicycling is a subject of controversy. Second, the costs associated with this regional estimate are impossible to determine. Since cost-effectiveness figures are needed to compare one TCM with another, a different approach is needed to establish air quality benefits attributable to bicycle facilities.

For this study, a demonstration project was not possible and a review of the literature did not uncover a reasonable model for estimating modal shift from cars to bicycles. Instead, an existing facility in the region is analyzed by observing the bicycle use and by surveying bicyclists about their modes of transportation when they are not bicycling. This type of survey reveals the existing use patterns associated with this facility. Results of the survey are used to calculate VMT and associated pollution diverted by the users of this facility. The costs associated with this system and the cost per ton of emissions diverted are then calculated.

The survey results, however, do not indicate what the use pattern was before the present facilities were installed nor what further use could be stimulated by other facilities. One approach to the problem of estimating future use is to calculate a demand range. A demand range defines the lowest and highest possible bicycle use for a given purpose or facility (4). A reasonable estimate of the potential use that might be stimulated by measures to increase bicycle use lies somewhere between these extremes.

In this study, a demand range is used to identify these extremes and a range of emissions reductions and costs that are associated with bicycle use are then calculated. Potential demand is not identified because to do so would require as complete an analysis as possible of the many factors known to influence bicycle users. Also, a demand range can be generalized with greater confidence than can a potential level of use estimated for a specific community.

Twenty-seven factors associated with bicycle use have been identified by EPA (1, p. 16). One factor, climate, is relatively quantifiable and generally consistent throughout the northeastern Illinois region. Previous estimates of potential bicycle use for this region have not attempted to take account of weather. Since climate determines the number of cycling days for most bicyclists, it is an important factor in calculating use and therefore has been taken into account in calculations of the emissions saved in this case study. Climate is especially significant in relation to air quality. For example, although bicycling occurs primarily within just a seven-month period from April through October, this period coincides with the season of highest ozone readings.

We could argue that the air quality impact of a shift from cars to bicycles should be weighted to reflect this coincidence. This operation is beyond the scope of this report; however, note that the overall air quality benefits attributable to bicycling might be greater than those calculated due to the coincidence of the bicycling and ozone seasons.

METHODOLOGY

This report analyzes the air quality benefits attributable to bicycling by commuters who park their bicycles at the two commuter stations in Wilmette, a suburb of Chicago--the Chicago North Western Railroad (CNW) and the Chicago Transit Authority (CTA). The trip from home to transit station has good potential for diversion from automobiles to bicycles since it often involves a reasonable distance to bicycle by people who are carrying a light load.

The community of Wilmette was chosen because it has relatively better facilities than other communities for bicycle and transit commuters. Bicyclists were surveyed at both locations and emissions reductions attributable to bicycle use are calculated for trips to both. A demand range is calculated for only the CNW station because the necessary residential information was available for that station alone.

The bicyclists were surveyed at the two stations on May 7, 1980, during the morning-rush period (6:15-9:30 a.m.). Each cyclist answered three questions concerning bicycling to the transit stations:

1. How far does he or she cycle to get to the station?
2. How often does he or she cycle to the station? and
3. How does he or she get there when not bicycling--walk, bus, drive, automobile passenger, or drive to the city?

Virtually every cyclist was approached and out of a total of 96 bicycles parked at the two locations, 88 cyclists responded to these questions.

For those respondents who use a car for some or all of their trips (when not cycling) the VMT not traveled by car when the person cycled is calculated. Some of these trips involve automobile drivers (park-and-ride), some automobile passengers (kiss-and-ride), and some who drive to the city when not cycling to the station. The kiss-and-ride trips are assigned twice the VMT as the park-and-ride trips. Those who always bicycle and those who walk or take the bus when not cycling are not counted as having diverted automobile VMT when bicycling.

Wilmette has a large transit ridership and a good bus system. The bus service in some communities is not as good as in Wilmette. In Wilmette, even a very high level of bicycling would probably not affect bus ridership enough to cause a reduction of bus service. Therefore, diversions from bus to bicycle are not expected to have any air quality impact. In communities that have poor or nonexistent bus service, bicycle-support facilities might have a greater air quality impact because more trips per capita are currently being made by car that could be diverted to bicycles. For this reason, the VMT diversions and the air quality impacts for Wilmette as the transportation mix now exists and as it might be without a bus system are calculated. The automobile VMT that would be diverted by bicycling if no bus service existed is calculated by dividing the bus trips between park-and-ride and kiss-and-ride in the proportion represented by the survey responses.

CLIMATE

Weather was mentioned by many of the survey respondents, often spontaneously, when asked about the frequency of cycling to the station. The number of cycling days on which bicycle use could reasonably be expected are calculated and the yearly calculations of actual and potential emission reduction are based on that number of days.

April through October has been chosen as a reasonable bicycling season in the Chicago area. At least half of the days in these months have low temperatures no colder than 40°F. Low temperatures are usually late night or early morning readings so the daytime temperature on a day when the low is 40°F will usually be between 45° and 80°F. In addition to cold weather, precipitation is a deterrent to bicycle riding. Since this study deals with commuters, the probable monthly bicycling days based on 5 days per week, or an average of 21 days per month, are calculated.

The number of cycling days per year are determined by subtracting the number of days with measurable precipitation from these months. This number represents the minimum number of cycle days; a number of respondents indicated that they also cycle in inclement weather. A Boston survey found that cycling activity decreases when the temperature is below 40°F; however, the effect of temperature may be overestimated. The Boston survey found that 10 percent of the student population bicycled for 10-12 months of the year, and 22 percent bicycled for 6 to 9 months (1, p. 18).

Precipitation is probably more inhibiting than temperature (1, p. 18). The average number of days that have measurable precipitation for the seven-month bicycling season is 9.5 days/month. Therefore, the average number of cycling days per 217-day season is

$$31 - 9.5 = 21.5 \times 7 \text{ months} = 150.5 \text{ cycling days/year.}$$

If this number is factored for a five-day workweek (21-day work month and 147-day season), the commuter cycling days can be determined as follows:

$$9.5 \div 31 = 0.306 \times 21 = 6.43 \text{ rain days}$$

$$21 - 6.43 = 14.57 \times 7 \text{ months} = 102 \text{ commuter cycle days/year.}$$

The VMT per day diverted were multiplied by the number of commuter cycling days and the emissions diverted were calculated according to EPA Mobile 1 model for 1982 emissions. The same calculations were done for the without bus service scenario with the reported bus trips divided between the two types of automobile trips according to their actual distribution among those surveyed. These calculations are done for the CNW and CTA stations.

A demand range is calculated for the CNW station alone. The actual use observed and emissions eliminated by the present facilities serve as the low end of the demand range. The high end (or theoretical limit) for bicycle demand for this trip is calculated by using information on the percentage of CNW riders who live within average bicycling distance of the station (4). Emissions reductions and costs are calculated for the low and high ends of the demand range. The costs attributed to the lowest (current) level of bicycling to the station are those of the current parking facilities. Assigned to the highest level (theoretical limit) of bicycling are the costs of improved parking facilities and a rules-of-the-road enforcement program that, although not wholly attributable to commuter activity, would be necessary if bicycling increased greatly during the morning rush hour. The lowest level of bicycle demand (present actual use) and the theoretical limit for demand thereby define a range of potential demand, associated emissions reductions, and costs per ton of pollutant eliminated.

RESULTS

The number of bicycles in the racks at the end of the survey period was 75 at the CNW station and 21 at the CTA station. These numbers are consistent with the numbers counted on May 6 and are probably typical for a fair day at this time of year. The 75 cyclists at the CNW station represent 5.3 percent of the regular commuter riders at that station based on a daily average ridership of 1421 (5). The 21 cyclists parked at the Linden CTA station represent 0.85 percent of the morning rush-hour ridership (6). The average trips of the bicyclists surveyed were 1.33 miles at the CNW station and 1.7 miles at the CTA station. The longest trip to the CNW station was 3 miles and to the CTA station, 5 miles.

The survey answers also indicate that most of those commuters who bicycle to the station do so daily during the bicycling season. This finding indicates that bicycles are being used as a regular transport alternative.

Emissions Calculations

Most bicycle trips to commuter stations are less than 2 miles. Cold-start emissions, at an average speed of 18 mph, last for 505 s, in which time the car would travel approximately 2.5 miles. Therefore, most bicycle trips to commuter stations that replace automobile trips are replacing trips made in the cold-start phase. The calculations for diverted emissions are based on EPA 1982, 100 percent cold-start emission factors (7).

Calculations are made for the three dominant pollutants associated with automobile use: hydrocarbons (HC), carbon monoxide (CO), and nitrogen oxide (NO_x). The calculations for the actual automobile VMT diverted (with and without bus service) by current bicycle trips to the two stations are summarized in Table 1. These calculations represent the air quality benefits attributable to the low end of the demand range. They are based on the number of automobile trips being diverted by the current bicycle trips to the commuter stations.

Table 1. Low estimate of demand range.

Item	CNW Station (n = 71)	CTA Station (n = 17)
VMT/day by bicycle	181.4	51.8
VMT/day diverted from automobile	54.08	47.6
VMT/day that would be diverted from automobile if no bus service available	155.3	87.4
Air quality benefits with bus service (tons/year)		
HC[a]	0.035	0.031
CO[b]	0.472	0.415
NO_x[c]	0.015	0.014
Air quality benefits for similar community without bus service (tons/year)		
HC[a]	0.102	0.057
CO[b]	1.35	0.762
NO_x[c]	0.044	0.025

[a]Calculated by multiplying estimated hydrocarbon emissions (5.83 g/mile) by estimated VMT/day diverted from automobile travel by number of commuter cycling days/year (102).
[b]Calculated by multiplying carbon monoxide emissions (77.57 g/mile) by estimated VMT/day diverted from automobile travel by number of commuter cycling days/year (102).
[c]Calculated by multiplying nitrogen oxide emissions (2.54 g/mile) by estimated VMT/day diverted from automobile travel by number of commuter cycling days/year (102).

Reductions in automobile emissions due to bicycle use depend on the level of bicycle use and how many bicycle trips are replacing automobile trips. Estimates of potential reductions in emissions depend, in turn, on estimates of potential bicycle use or demand. Walsh (4) has suggested that a demand range be calculated by using a low estimate based on actual use and a high estimate based on the number of trips that could be made by bicycle. The length of the trips in the high estimate would be that of the average current trip made by bicycle.

The emissions diverted by the Wilmette bicycle trips represent the smallest benefit potentially attributable to that facility, the low estimate. The quantity of emissions diverted by all possible trips within the average trip-generating radius represents the highest benefit theoretically possible for that facility, the high estimate. Potential benefits lie somewhere in between.

A demand range can be calculated for the CNW bicycle commuters by using information on the residential distribution of CNW commuters. The average length of a one-way bicycle trip to the CNW station is 1.33 miles. Twenty-one percent of the regular commuters live within approximately 0.5 mile of the CNW station (8). These riders are most likely to walk to the station. [Seven of our respondents bicycled only 0.5 mile, but this must be compared with the 49 bicyclists within the 1- to 1.5-mile radius. Ohrn and others also identified 0.5 mile as a walking radius (9).] Almost 41 percent of the ridership live within the quarter-sections adjacent to the CNW station, which places them within the 0.5- to 1.5-mile radius associated with cycling by this survey and other studies (1, p. 17; 9). Daily ridership at the Wilmette station is 1421; 40.8 percent of this ridership (580 riders) would live within the average bicycling radius. Thus, the demand range varies between a low of 75 riders (71 surveyed plus 4 bicycles already parked) or 5.2 percent and a high of 580 riders or 40.8 percent ridership. Potential demand lies somewhere within the demand range. Walsh suggests that 50 percent of potential short-distance trips could be diverted to bicycles (4); in this case, 20.4 percent of the ridership. This is by other estimates a high figure. Bikeways in Northeastern Illinois suggests that 7.5 percent of trips to transit stations could potentially be diverted to bicycles (3, p. 39). In Madison, Wisconsin, 13 percent of all vehicle trips are made by bicycle (1, p. 10).

For the purposes of this analysis, the emissions that could be diverted at the high end of the demand range are calculated and, together with the low end calculations, serve as a range of emissions diverted. Some of the costs associated with approaching that limit are also computed and annualized. Minimum improvements needed for maximum bicycling activity would include increased storage facilities and a bicycle rules-of-the-road enforcement program.

If 5.2 percent of the CNW ridership diverts 54.08 miles of automobile traffic/day, then 40.8 percent of the CNW ridership would divert approximately 418.2 miles of automobile traffic/day with bus service. Were there no bus service, 155.3 miles would be diverted by current bicycling; 40.8 percent of the CNW boardings would divert approximately 1218 miles of automobile traffic/day without bus service. The high range of emissions associated with these VMT are summarized in the calculations below. These figures represent a theoretical limit rather than a potential estimate.

For the Wilmette CNW Station with bus service,

HC emissions = 5.8 g/mile x 418.2 mile = 2425.56 g/day x 102 days = 247 407.12 g/year = 0.273 tons/year.
CO emissions = 97.57 g/mile x 418.2 mile = 32 439.77 g/day x 102 days = 3 308 856.9 g/year = 3.65 tons/year.
NO_x emissions = 2.54 g/mile x 418.2 mile = 1 062.23 g/day x 102 days = 108 347.3 g/year = 0.119 tons/year.

For a similar community without bus service,

HC emissions = 5.8 g/mile x 1218 mile = 7064.4 g/day x 102 days = 720 568.8 g/year = 0.794 tons/year.
CO emissions = 77.57 g/mile x 1218 mile = 94 480.26 g/day x 102 days = 9 636 986.5 g/year = 10.62 tons/year
NO_x emissions = 2.54 g/mile x 1218 mile = 3093.72 g/day x 102 days = 315 559.44 g/year = 0.348 tons/year.

The range of emissions that can be saved by bicycling to the Wilmette CNW station and a similar station not served by bus are summarized in the list below.

The range of emissions eliminated in Wilmette with bus service is as follows:

HC, from 0.035 tons/year to 0.273 tons/year;
CO, from 0.472 tons/year to 3.76 tons/year; and
NO_x, from 0.015 tons/year to 0.119 tons/year.

For a similar community without bus service, the following range of emissions would be eliminated:

HC, from 0.102 tons/year to 0.794 tons/year;
CO, from 1.35 tons/year to 10.62 tons/year; and
NO_x, from 0.044 tons/year to 0.348 tons/year.

Cost-Effectiveness

Many costs and benefits can be attributed to bicycling. If the proportion of all bicycling for different purposes is known, then a portion of the costs of the entire bicycle support system (e.g., bicycle routes, automobile, parking, education, and promotion) can be assigned to each purpose. The amenities of the entire Wilmette bicycle system--on-street routes, signing and improvements, parking facilities, and a modest promotion and education program--serve commuters, but the cost of these facilities cannot be solely attributed to commuter bicycling. The costs of parking facilities at the two commuter stations are the main costs attributable to commuter bicycling.

Item	Cost ($)
CNW station parking	
Shelter for bicycle racks	14 000
Bicycle racks, two locations	1 200
	15 200
CTA station parking	
Bicycle racks	600
Total	15 800

If we assume a 10-year life for these facilities, the costs are annualized by using a 12 percent discount rate and a capital recovery rate of 0.177:

CNW: $15 200 x 0.177 = $2690.4/year.
CTA: $600 x 0.177 = $106.2/year.

The costs of eliminating air pollution with commuter bicycling to Wilmette's commuter rail and public transit stations are summarized in Table 2.

The minimum costs associated with provision for the 580 bicycles associated with the high end of the demand range would include 430 new parking spaces and a bicycle rules-of-the-road enforcement program such as was implemented in Niles, Illinois, to help alleviate some of the traffic conflicts that might occur with a large increase in bicycling (1, p. 48).

Niles adopted an enforcement program in which summer wardens stopped 6000 cyclists and issued warnings and instructions about proper bicycling techniques. Bicycle-related accidents in the town went from 17 to 3 during the summer of 1975. The cost of this program was the salary of the officer in charge of the program and approximately $14 000 for summer wardens and their uniforms. The $14 000 (inflated for 1980 dollars) can be solely attributed to bicycling improvement. An inflated cost of $22 400 for enforcement could justifiably be attributed to the high end of the demand range, but the police officer's time would probably be used in some other way were there no enforcement program. That salary will not be attributable to bicycling improvements.

If we assume that the new parking was comprised of 30 bicycle lockers and sheltered rack spaces for 400 bicycles, the costs of the high end of the demand range would be as follows:

Item	Cost ($)
30 bicycle lockers at $300/bicycle	9 000
400 bicycle rack spaces at $14 000/100	56 000
Enforcement program at $22 400/year	224 000
Total	289 000

Again, assuming a 10-year life for these facilities and a capital recovery factor of 0.177, the cost-effectiveness figures would be based on a cost of $53 843.40/year. These costs would be added to the costs already incurred at the CNW facility in Wilmette to determine the cost-effectiveness of the high end of the range:

$289 000 + 15 200 = $304 200.

The costs of eliminating one ton of each pollutant at the high end of the demand range are summarized in comparison to the low-end figures in Table 3.

Socioeconomic Impacts

The socioeconomic impacts for all of TCM evaluations for this region were prepared by James Jarzab of the Northeastern Illinois Planning Commission (10).

Improved bicycle facilities have few negative socioeconomic impacts associated with them. Improvements may entail the provision of racks and lockers, bicycle paths, and safety education programs; however, few of these activities can be considered drawbacks to the TCM. The two important factors that must be considered when contemplating improved bicycle facilities are safety and traffic operations. Increased bicycle use may cause additional safety problems and traffic might be disrupted if automobile and bicycle operators do not obey state vehicle operating procedures. A bicycle rules enforcement program and improved drivers' education about the bicyclist's rights and responsibilities are appropriate complements to improved facilities.

Positive impacts include greater amenities to residents in terms of reduced air pollution, increased recreational potential, the availability of an alternative mode of transportation, improved motorist awareness of bicycle users, and related social benefits (10). Little capital expenditure need be involved in the provision of bicycle facilities, and aggregate benefits appear to far outweigh project costs.

Table 2. Costs of eliminating air pollution with commuter bicycling facilities.

Pollutant	Amount/Year (tons)	Low Cost Estimate ($/ton) CNW[a]	CTA[b]
With bus service			
HC	0.0312	86 230.77	3425.80
CO	0.415	6 482.89	255.90
NO_x	0.0136	197 823.53	7273.97
Without bus service			
HC	0.057	47 200.00	1863.15
CO	0.762	3 530.70	139.37
NO_x	0.025	107 616.00	4248.00

[a]Calculation based on cost of $2690.40/year.
[b]Calculation based on cost of $106.20/year.

Table 3. Facilities for bicycle commuters to CNW station in Wilmette: summary of demand range.

Demand Range	Automobile Range (VMT/year diverted)	Emission Range (tons/year diverted) HC	CO	NO_x	Cost Range ($/year)	Cost-Effectiveness Range ($/ton) HC	CO	NO_x
With bus service								
Low estimate	5 516.06	0.035	0.472	0.015	2 690	86 231	6 483	197 824
Potential high estimate	42 656.4	0.273	3.65	0.119	53 843	197 229	14 752	452 466
Without bus service								
Low estimate	15 840.6	0.102	1.35	0.044	2 690	47 200	3 531	107 616
Potential high estimate	124 236	0.794	10.62	0.348	53 843	67 813	5 070	154 722

Table 4. Direct socioeconomic impacts due to implementation.

Sector Affected	Impact	Unit of Measurement	Preliminary Impact Assessment[a]	Description of Change from Existing Conditions
Residential	On no. of dwelling units	Dwelling units demolished; potential for new units	0	This TCM is expected to require neither the taking of nor the development of dwelling units
	On no. of households	Households added or dislocated	0	Same as above
	On existing residences	Positive or negative effects on use and enjoyment of residences	+	Provision of bicycle facilities should reduce the amount of vehicle noise and air pollution on and around residential areas, and thereby improve conditions in areas of implementation
Business	On no. of businesses	No. of businesses dislocated or potential for additional businesses	0	No significant impacts are anticipated
	On existing businesses	Potential increase or decrease in business activity	0	No significant impacts are anticipated
Parking	On availability of off-street parking	Gain or loss of parking spaces	+	Some off-street parking space availability should result because of decreased automobile use
	On availability of on-street parking	Gain or loss of parking	+ 0 -	Some on-street parking may be lost due to curb lane dedication to bicycle use; however, also possible is that other spaces would be freed by bicycle users diverted from automobiles
Employment	On no. of temporary jobs	Gain or loss of temporary jobs	+	Some temporary jobs may be created for the purpose of public information, marking and installation of facilities, and bicycle safety promotion
	On no. of permanent jobs	Gain or loss of permanent jobs	0	No impacts are anticipated
Goods movement	On efficiency of goods movement	Positive or negative impact	+ 0	Goods movement will be affected to the extent that automobile traffic is reduced (positive impact) or truck traffic will be delayed (negative impact); in any case, the impact is expected to be small
Municipal services	On delivery of municipal services	Positive or negative impact	+ 0 -	Same factors that affect goods movement may affect delivery of municipal services
Land use	On sensitive land uses	Increase or decrease in number of acres of sensitive land uses	+	Bicycle use is consistent with a policy of road improvement rather than road and highway expansion; use of bicycles as an alternative to automobile use might, therefore, contribute to land preservation
	On land use	Change in land use from a higher-value use to a lower-value use or from a lower-value use to a higher-value one	+	Bicycle facilities increase value of land by increasing its usefulness for recreation and transportation
Land values	On surrounding land values	Increase or decrease in land values	+	Additional amenities like bikeway facilities can influence land values by making property more attractive
Taxes	On local assessed valuation	Increase or decrease in assessed valuation due to property added to or removed from tax rolls and change in land use patterns	+	Areas that have bicycle facilities will be more attractive to home buyers; this should be reflected in increased property value and subsequently assessed valuation
Taxi service	On service availability and safety	Positive or negative impact	0	Taxi service may be affected somewhat because of the availability of bicycle facilities for short trips in good weather; however, this diversion is expected to be small; traffic movements will be affected as any other vehicle would be affected

[a]+ = positive, - = negative, and 0 = no impact.

As a TCM measure, improved bicycle facilities have few socioeconomic drawbacks and, for the most part, are supportive of regional socioeconomic policies, goals, and objectives. The socioeconomic impacts of bicycle facilities are summarized in Tables 4 and 5 (10, pp. 35-44).

DISCUSSION OF RESULTS

The high end of the demand range serves as a theoretical limit to the air quality benefits obtainable through the promotion of bicycling to transit stations in Wilmette. The value of the demand range is that the low end of the range illustrates that feasible bicycle support facilities can produce air quality benefits at a low cost. The high end of the demand range demonstrates the limit within which bicycle use for a given purpose can be expected to produce air quality improvements.

Given our assumption about costs associated with bicycling, air quality improvements appear to become more expensive as bicycle use increases. The major reasons for this are (a) the addition of an enforcement program (as in Niles) because of the increased need to integrate bicycle and automobile traffic and (b) the addition of storage lockers that are roughly twice as expensive as the sheltered racks at the CNW station. The higher cost per ton of the more-extensive system still represents a cost efficiency well above that of many other TCMs evaluated. The emission reduction cost-effectiveness varies from \$13 319 to \$985 349/ton for public transportation improvements and from \$98 918 to \$731 250/ton for park-and-ride lots (11). At the high end of the demand range the cost-effectiveness of commuter bicycle facilities as a measure to reduce hydrocarbons varies from \$67 813 to \$197 229/ton. As bicycling increases for purposeful trips, various road and traffic improvements, such as signs, special signalization, storage facilities, lane markings, and enforcement personnel, may become necessary. At the same time, money might be saved by having to provide fewer parking spaces for automobiles. The use of the bicycle as a transportation vehicle necessitates

Table 5. Indirect socioeconomic impacts due to implementation.

Variable	Impact	Unit of Measurement	Preliminary Impact Assessment[a]	Description of Change from Existing Conditions
Land use	On long-term land use patterns	Change in land use, from a higher-value use to a lower-value use, or from a lower-value use to a higher-value one	+	Bicycle facilities increase value of land by increasing its usefulness for recreation and transportation
Safety	On vehicular and pedestrian safety	Potential increase or decrease in vehicle and pedestrian accidents	+	Increased bicycling on public highways may result in an increase in automotive-bicycle and pedestrian-bicycle accidents; accidents may, however, be decreased by implementation of appropriate education programs and traffic management
Health	On susceptible population groups	Positive or negative impact due to improvement or deterioration of air quality	+	Automobile trips diverted to bicycle trips will reduce emissions and thereby improve community health
Accessibility	On accessibility for mobility-limited persons	Increased or decreased accessibility	0	Accessibility for the mobility-limited will not be affected
	On neighborhood accessibility	Increased or decreased accessibility to adjacent neighborhoods	+	Increased bicycle trips should be generated both intra- and inter-community due to increased bicycle facilities
Traffic	On amount of traffic in sensitive areas	Increase or decrease in traffic in sensitive areas	+	Positive impact would be expected from the diversion of some automobile trips to bicycles
Buildings	On buildings in vicinity of TCM	Positive or negative impact on building maintenance due to air quality changes and building vibrations	+	Reduced emissions should result in reduced building maintenance costs

[a]+ = positive, – = negative, and 0 = no impact.

and justifies costs more consistent with the costs of other modes.

It is possible to speculate on some of the reasons for the large difference in the number of bicycle riders at the CTA and CNW stations. Only 21 bicycles were parked at the CTA station; 75 were parked at the CNW. The survey responses show that cyclists to the Linden CTA ride longer distances than those at the CNW stations (CTA average was 1.7 miles; the longest trip was 5 miles; CNW average was 1.33 miles; the longest trip was 3 miles). The Linden CTA stop is the end of the line and those North Shore riders who wish to take the CTA rather than the CNW come to the Linden or Evanston stations. The North Shore corridor is, on the other hand, served by many CNW stations. The longer trips to the Linden station might discourage bicycle use. Another, and more probable contributory factor to the difference in bicycle ridership is that the parking facilities at the CNW station are superior and more numerous. The CNW station has 100 spaces covered by a roofed structure and 50 additional spaces south of the station. The CTA has conventional unprotected racks that provide parking for up to 40 or 50 bicycles.

The case study can be generalized to other communities with commuter rail stations. If each of these communities were to provide parking facilities like Wilmette's, then the percentage of commuters who ride their bicycles to the station might be expected to be the 5 percent observed at Wilmette. Inducements and deterrents other than these facilities would, of course, affect ridership. It is probably fair to suggest that communities that have land use patterns similar to those in Wilmette would gain a similar bicycle ridership given similar facilities. This could mean a fairly significant increase for some communities where moderate inducements might remind and encourage residents to exercise the bicycling option. The air quality benefits would be especially significant in communities that currently have poor bus service.

In addition to generalizing the findings at the Wilmette CNW station, communities could reproduce the survey used in this case study for any trip generator. To calculate a demand range, some data would be necessary on the residential distribution for those who travel to the trip generator under consideration.

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Bicycle Traffic Volumes

CATHY A. BUCKLEY

This paper provides information on bicycle traffic volumes in the metropolitan Boston area. This information allows a better understanding of factors that affect this mode and their relative importance to the bicyclist. The data reported here were collected between 1974 and 1981. All bicycle counts were done manually, mainly during weekday peak periods. The volumes have grown at a 7.5 percent annual rate compounded over the past six years. Volumes varied significantly according to season of the year and weather. These two influences operated independently of each other to some extent. Twelve-hour counts indicated definite peaks in the morning and evening. The morning peak hours occurred in a narrower time band; the evening-peak-hour volumes were an average of 20 percent higher. Bicycle traffic increased by 300 percent on a day when transit was unexpectedly out of operation. During an evening-peak-period count, half of the cyclists used a bicycle path and half used three adjacent arterials. Average daily bicycle traffic volumes in the inner metropolitan area are presented for 1976. Possible correlations between volumes and the number of reported bicycle accidents are discussed.

This paper reports on bicycle traffic volumes collected in the metropolitan Boston area since 1974. Some implications of the data, including the need for further research, are discussed.

BICYCLE TRAFFIC DATA

A number of bicycle traffic counts have been conducted in the Boston area since 1974, most of them by the Central Transportation Planning Staff of the Boston metropolitan planning organization (MPO).

The majority of the counts have been done in Boston and Cambridge, just north of Boston. Counts have also been done in Brookline and Newton, west of Boston, and in Somerville, Arlington, Belmont, Lexington, and Bedford, north and northwest of Boston (see Figure 1).

The impetus for the collection of bicycle traffic volumes was the U.S. Environmental Protection Agency's transportation control plan for Boston. This 1975 document required that encouragement of bicycle use be included in the area's attempt to reduce pollution. Virtually no information was available on actual bicycle traffic volumes, so some was collected in 1975 and 1976. Data have continued to be collected since then to improve our understanding of the issues discussed in this paper.

Bicycle counts have been done at more than 50 locations in the Boston area. Most of the counts were on weekdays, during the morning or evening peak period. The emphasis was on commuter, not recreational, traffic. Unless otherwise noted, precipitation neither occurred on nor was forecast for the days of the counts.

All counts were done manually, included turning movements, and were calibrated in 15-min segments. All of the data reported here were collected by staff members or adult volunteers and are considered to be reliable.

GROWTH IN BICYCLE TRAFFIC

Bicycle volumes were collected at 11 intersections on Thursday, October 9, 1975, as part of the transportation control plan work. Bicycle volumes at 9 of the intersections were collected again exactly five years later, on Thursday, October 9, 1980. The weather was similar--temperatures were in the low 50°s at 7:00 a.m. and the wind was approximately 10 mph on both days; it was partly cloudy in 1975 and sunny in 1980.

The volumes for each day are shown in Figure 2. The evening-peak-hour volume was higher at all nine intersections in 1980; the increase varied from 5 to 60 percent. The total evening-peak-hour volume for the nine intersections was 1922 bicycles in 1980 40 percent higher than in 1975.

Volumes are compared in Figure 3 for four intersections at which counts were done on Wednesday, May 5, 1976, and five years later on Wednesday, May 13, 1981. The weather on these two dates was identical--at 7:00 a.m., temperature was 51°F, wind was 15 mph, and skies were mostly sunny.

The increases in the morning peak-hour volume at the four intersections varied from 35 to 236 percent. The total morning peak-hour volume for the four intersections was 575 bicycles in 1981, 57 percent higher than it was five years before.

The October 1975 and May 1976 volumes added together and compared with the total for October 1980 and May 1981 yield an average increase of 44 percent. This is an average annual compound increase of 7.5 percent. A 7.5 percent increase would be considered a high annual increase for automobile traffic. Because the bicycle's share of traffic is lower, a high annual increase is relatively less significant. For example, assume that the bicycle's share of commuter traffic was 1 percent in 1980. If a 7.5 percent annual compounded growth rate were to continue for 20 years, the bicycle's share would only be 4.2 percent by 2000. A 7.5 percent annual increase does suggest, however, that the bicycle is not just holding its share of the commuting market but steadily increasing it.

Figure 1. Bicycle count locations.

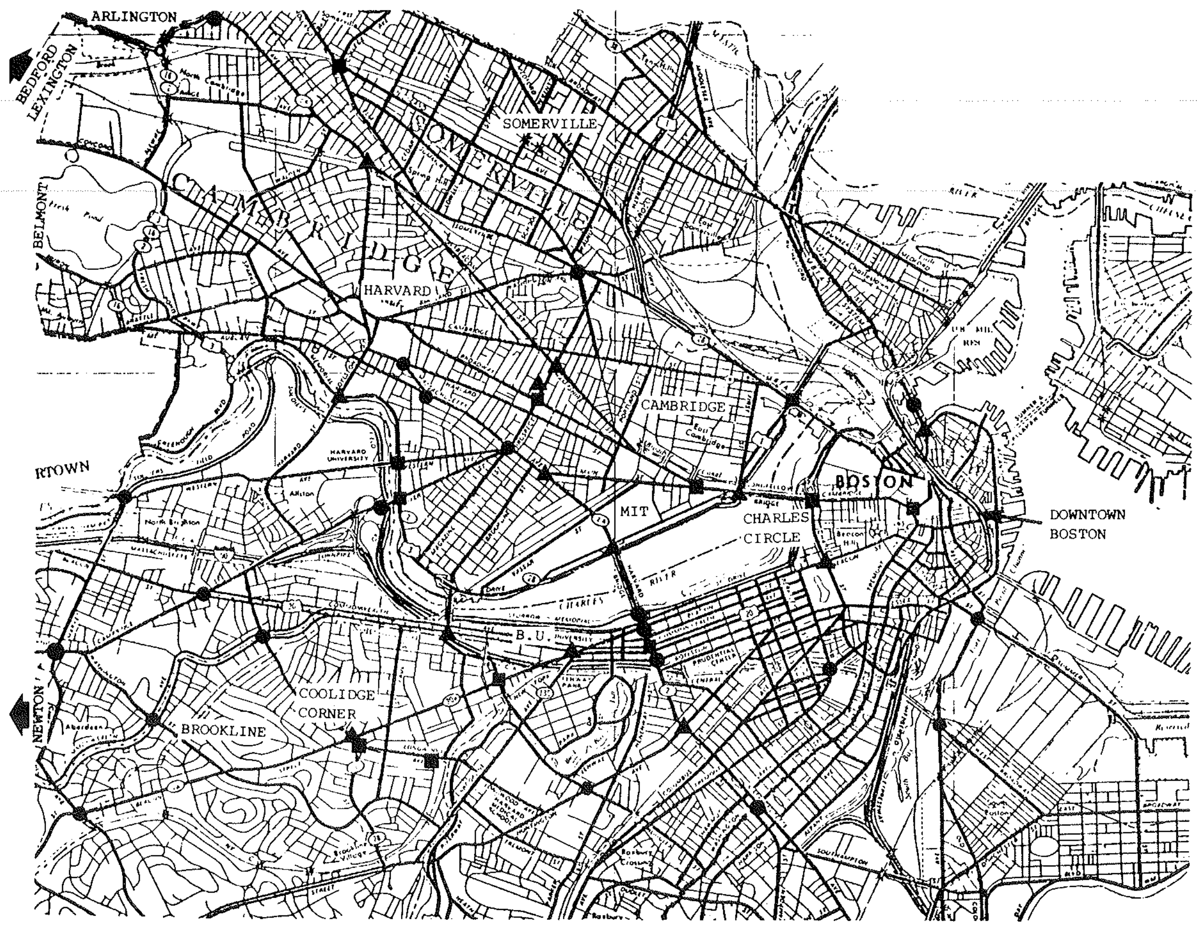


Figure 2. Growth in bicycle traffic, 1975-1980.

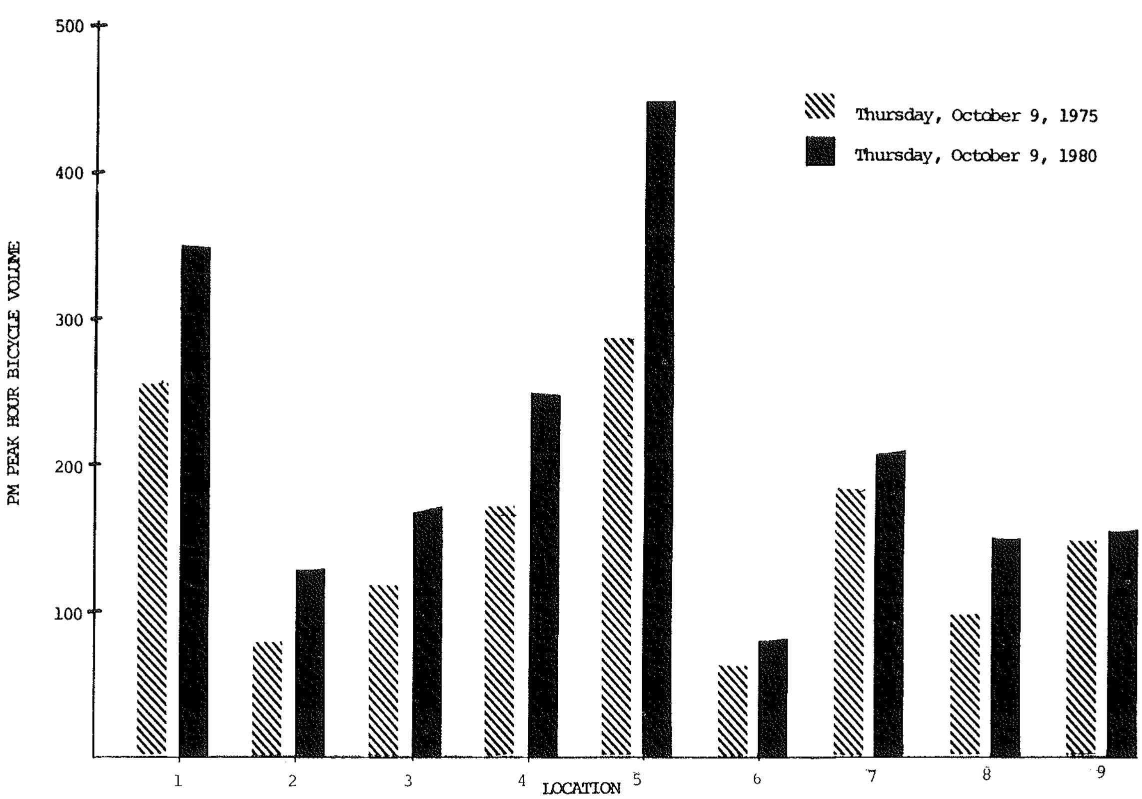
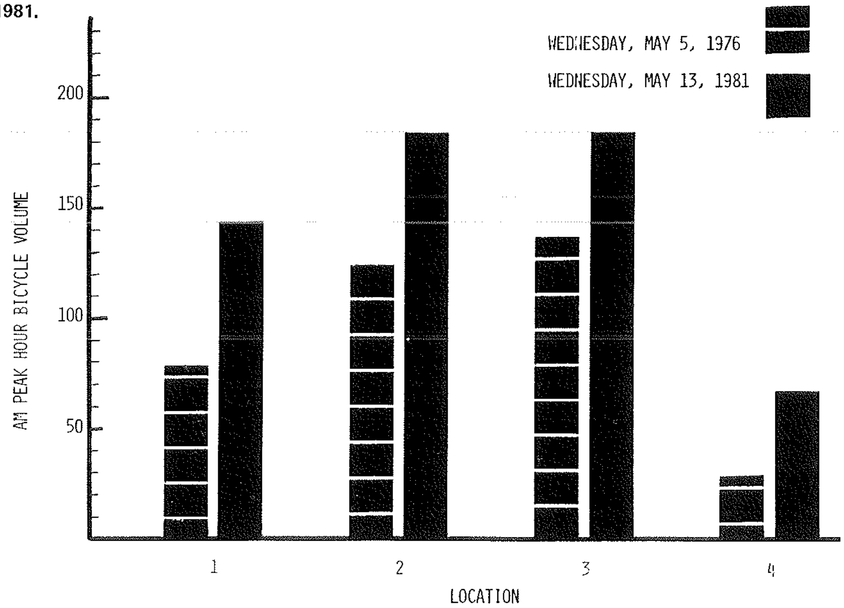


Figure 3. Growth in bicycle traffic, 1976-1981.



EFFECT OF OTHER MODES

Bicycle commuters prefer their mode for a variety of reasons. Some may cycle for health or simply because it is enjoyable. Others may cycle because it costs little or is convenient compared with the alternatives.

In the five-year period from October 1975 to October 1980, for example, the price of a gallon of regular, leaded gasoline rose from \$0.59 to \$1.19. The basic bus fare remained at \$0.25 but the rapid-transit fare doubled in July 1980 to \$0.50. These changes probably contributed to the 40 percent increase in peak-hour bicycle traffic during those five years.

Although cost is an important criterion in mode selection, availability is even more crucial. The disruption in fuel supplies in 1973 has received a great deal of the credit for the bicycle boom of that year.

An interruption of public transportation in Boston on Thursday, July 6, 1978, presented an opportunity to measure the effect of the unavailability of transit on bicycle use. Transit workers called a one-day wildcat strike, which completely shut down the system. Only the users who listened to the radio before heading to their buses or trains were aware of the strike.

Because the strike was unexpected, the bicycle count of Charles Circle that day had not been planned in advance and did not start until 8:45 a.m., well into the normal peak hour there. (Of nine morning peak-period counts at Charles Circle in 1980, five peak hours began at 8:00 a.m. and the other four at 8:15 a.m.) It is possible that the bicycle volumes prior to 8:45 a.m. were close to normal levels; many of those who improvised their bicycle trips because of the transit interruption probably got off to a late start.

Figure 4 compares bicycle volumes on the day of the strike with those of one week later. The 8:45-9:45 a.m. volume was more than 300 percent higher on the day of the strike. (On July 6 at 7:00 a.m. the temperature was 63°F, winds were 10 mph, and skies were partly cloudy. On July 13 at 7:00 a.m., the temperature was 70°F, winds were 7 mph, and skies were sunny.)

The most-striking aspect of these data is that a large number of commuters own bicycles, have quick access to them, and use them when their regular modes are unavailable. The counts also indicate that most of these one-day cyclists returned to their regular mode when service resumed. Finally, the data support inclusion of the bicycle in energy contingency planning.

SEASONAL VARIATIONS

Morning peak-hour volumes recorded throughout the year at one location are shown in Figure 5. The average volume for the three counts in March was 60 bicycles. The average of the two June counts was 198, which is 230 percent higher than the March average. The average of the two July counts was 235 bicycles, which is 290 percent higher than the March average. The volumes from the solitary counts in August, September, and October are, on the average, 25 percent lower than the July volumes.

Two 12-h counts were done at Coolidge Corner in Brookline, one in March 1974 and the other two months later. The total 12-h volume of May 8, 1974, was 100 percent higher than that of March 5, 1974 (959 bicycles were counted in May, 479 in March).

Although the above data are not sufficient to define seasonal factors, they demonstrate that most Boston bicycle commuters choose alternative modes in the winter. This is understandable, considering the colder temperatures and fewer daylight hours during late fall and winter. Other cold-weather circumstances that may discourage bicycle use are snow or ice, which make roadways slippery, and snowbanks, which decrease road space and visibility.

EFFECT OF WEATHER

Details of the three March 1980 counts cited in Figure 5 are presented in the table below. This information suggests the effect of weather within a given season.

	Conditions at 7:30 a.m.			Morning
Date	Wind (mph)	Sky	Temperature (°F)	Peak-Hour Volume
March 12, 1980, Wednesday	25	Clear	23	35
March 19, 1980, Wednesday	8	Clear	37	85
March 27, 1980, Thursday	13	Overcast	38	60

Figure 4. Effect of transit strike.

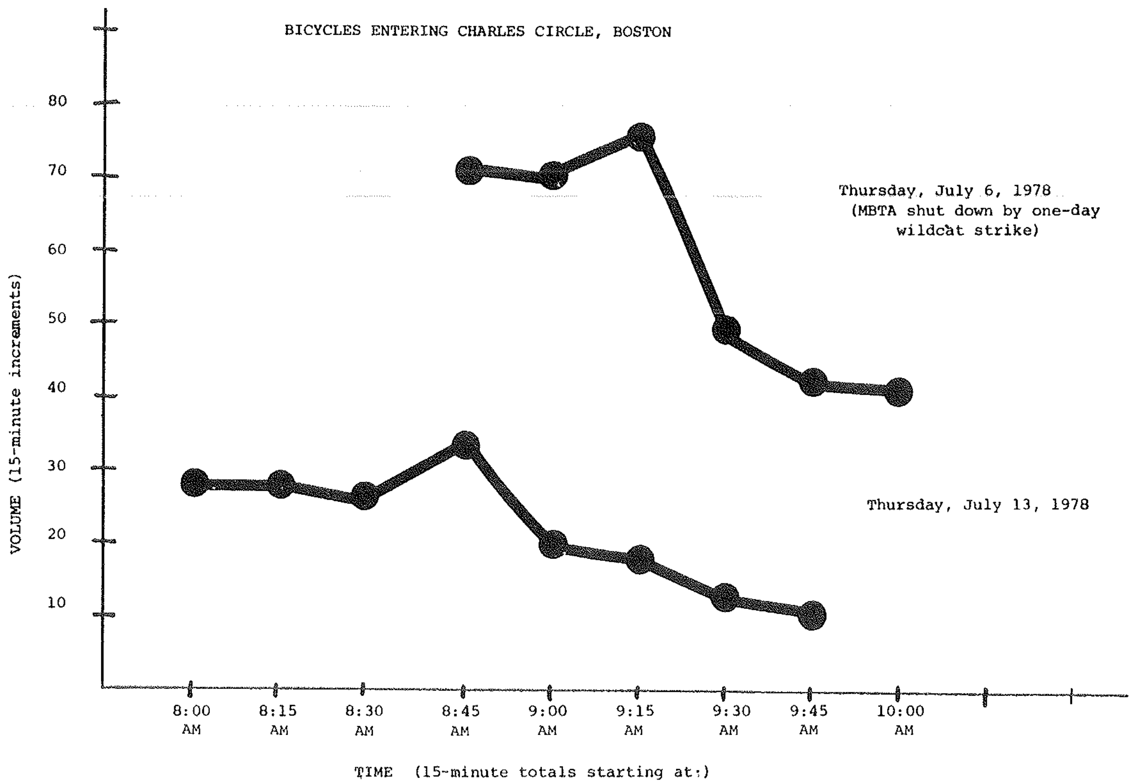
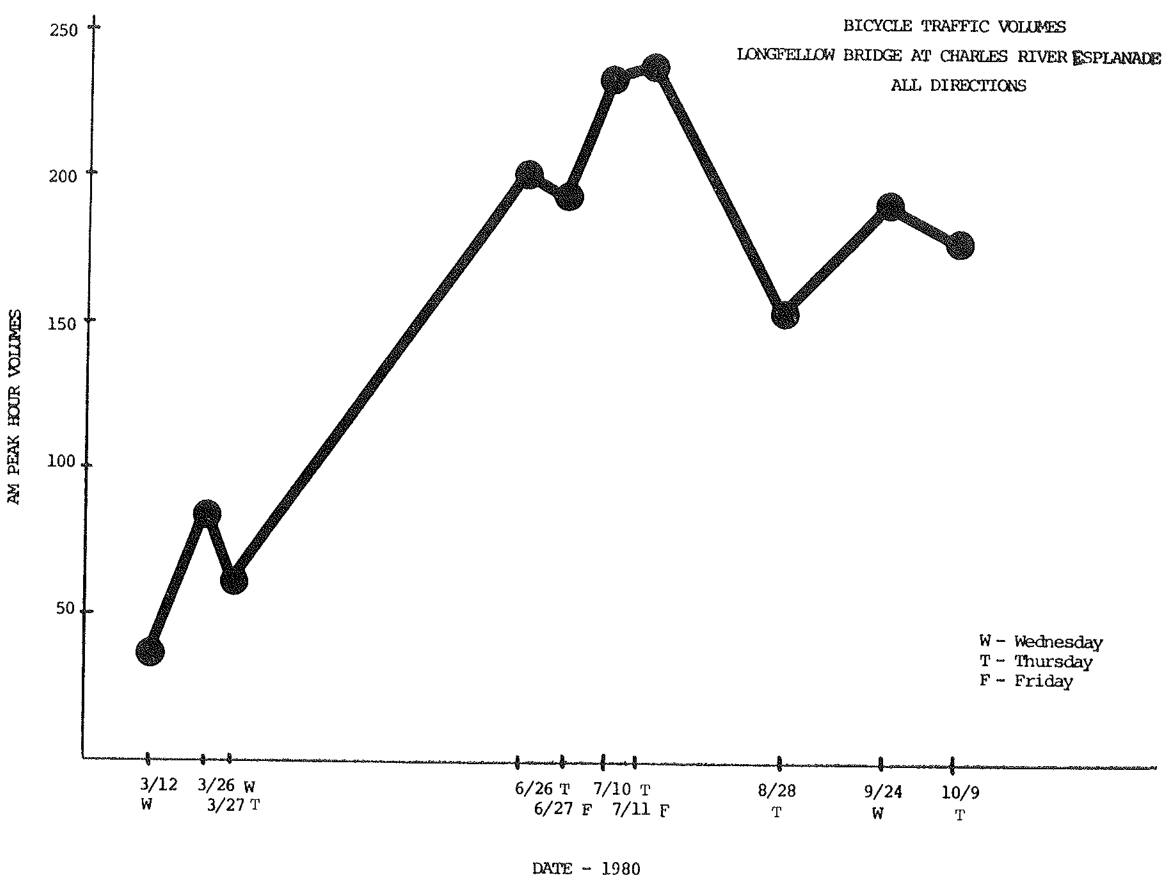


Figure 5. Seasonal variations.



A small fraction of cyclists commute year-round; however, only a portion of those do so when the 7:30 a.m. temperature is 23°F and the wind is 25 mph, as occurred on March 12. The morning peak-hour volume was 140 percent higher two weeks later on March 26, a warmer and less windy day.

The March 27th volume is 30 percent lower than the March 26th volume. This is probably too large a difference to be due to daily variations. The only notable difference in weather between the two days is that the sky was clear on the 26th and overcast on the 27th. Although it was not raining on the morning of March 27 and no rain was forecast, the clouds might have discouraged some cyclists.

Bicycle counts at six intersections were arranged for Wednesday, May 6, 1981. Because of fog and a light rain, only two intersections were monitored that morning, and the full-scale count was postponed until the following Wednesday. Figure 6 indicates the effect of fog and rain on bicycle volumes at the two intersections that were monitored twice. The morning peak-hour volume at Charles Circle, Boston, was 39 percent lower in the fog and drizzle than on the partly sunny day one week later. At Coolidge Corner, Brookline, the volume was 52 percent lower in the rain.

When comparing volumes on different days, not only the time of year, but also weather conditions must be considered. Weather variations within a season are important. Nevertheless, seasonal variations in bicycle volumes seem to occur regardless of the weather. For example, as cited in the preceding section, a 12-h volume in May 1974 was twice as high as one in March 1974. Yet the 7:00 a.m. temperature was 56°F on the March day, 10° higher than on the May day. The overcast sky and winds of 25 mph in March must be taken into consideration--the May count occurred on a sunny day with winds of 16 mph. However, the large difference in volumes suggests that some people simply do not cycle during what they consider to be the off-season, even when good weather conditions prevail.

OCCURRENCE OF PEAK HOURS

Forty-nine locations have been counted during morning peak periods and 52 during evening peak periods since 1974. Note the time at which the morning and evening peak hours occurred. The temporal distribution of those peak hours is shown in Figure 7.

As can be seen, the morning peak hours are more clustered in time than are the evening hours. Eighty-six percent of the 49 morning peak hours began at either 8:00 or 8:15 a.m. Eight percent started at 7:45 a.m., 4 percent at 8:30 a.m., and 2 percent at 8:45 a.m.

The evening peak-hour distribution is more spread out--63 percent occurred at either 4:45 or 5:00 p.m. One began as early as 3:45 p.m. and two as late as 5:30 p.m.

Counts from 7:30 to 10:00 a.m. would have captured all morning peak hours with 15-min margins on each end. To capture all of the evening peak hours would require a counting period of 3:30 to 6:45 p.m.

A cyclist might stop or visit on the way home from work, leave early for an appointment, or stay late to finish a task. In the morning, such diversions are probably less likely. Evening peak-period traffic would also be more likely than morning traffic to include cyclists on shopping or recreational trips. These considerations might explain why the morning peak hours occur within a narrower time band than do the evening peak hours.

TWELVE-HOUR COUNTS

Two intersections, Coolidge Corner and Charles Circle, were monitored for 12-h periods, from 7:00 a.m. to 7:00 p.m. Coolidge Corner, Brookline, is a commercial area at the intersection of Beacon and

Figure 6. Effect of rain.

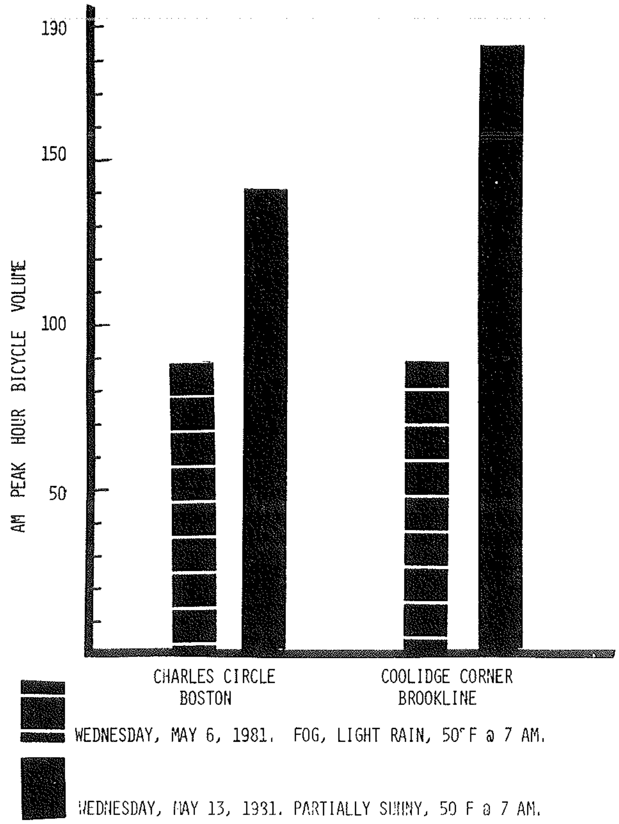
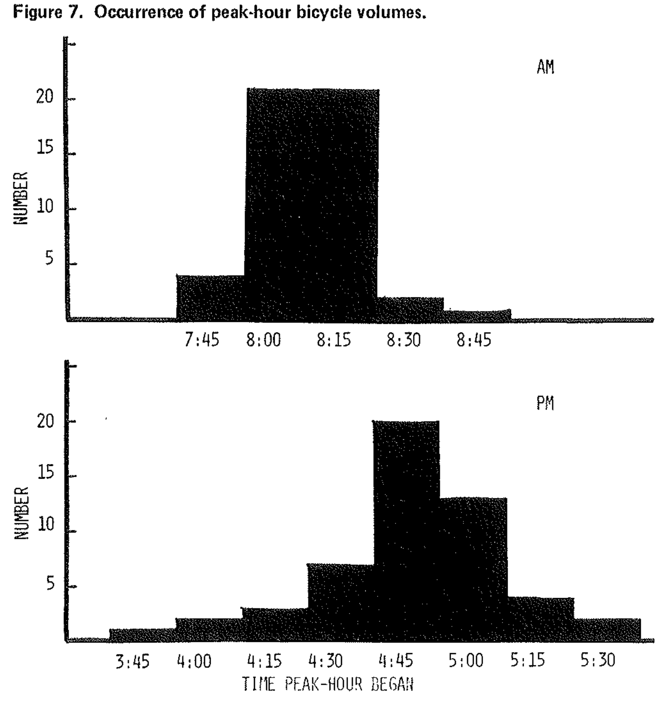


Figure 7. Occurrence of peak-hour bicycle volumes.



Harvard streets (see Figure 1). Beacon Street is a popular commuting route to Boston from points west. Boston University is a mile northeast of Coolidge Corner.

Counts at Charles Circle, Boston, include traffic both from the Longfellow Bridge, used by commuters from Cambridge and other communities west and north of Boston, and from the bicycle path along the Charles River. Traffic generators close to Charles Circle include the Massachusetts General Hospital and Government Center (see Figure 1).

Twelve-hour counts were done at Coolidge Corner on March 5, 1974, May 8, 1974, and May 13, 1981, and at Charles Circle on May 5, 1976, and May 13, 1981.

As shown in Figures 8 and 9, the five 12-h counts follow similar patterns. There is a definite morning peak, which begins around 8:00 a.m. The volumes then drop by 10:00 a.m. and stay relatively low until noon or early afternoon. A gradual increase in volume usually begins around 1:00 p.m. and continues until the evening peak hour, which is from about 5:00 to 6:00 p.m.

Table 1 presents information on the 12-h counts, including data on the peak hours. The ratio of the morning peak-hour volume to the total 12-h volume varied from 0.11 to 0.14 and averaged 0.12. The same ratio for the evening peak hour varied from 0.13 to 0.18 and averaged 0.15. On the average, the evening peak-hour volumes were 20 percent higher than the morning peak-hour volumes.

Traffic during the morning and evening peak periods probably is predominantly commuters and

Figure 8. Twelve-hour volumes, Coolidge Corner.

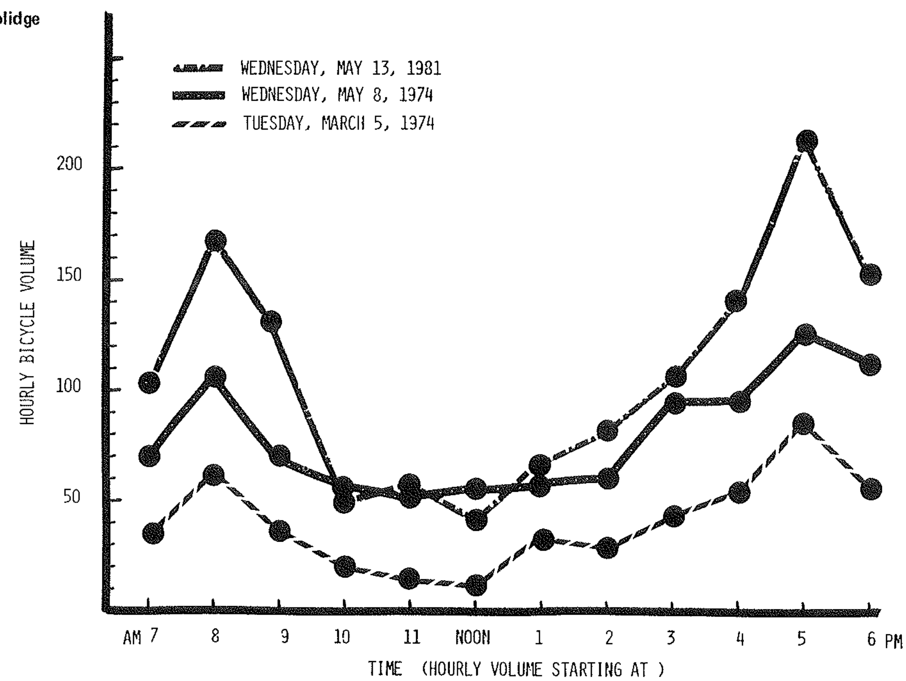


Figure 9. Twelve-hour volumes, Charles Circle.

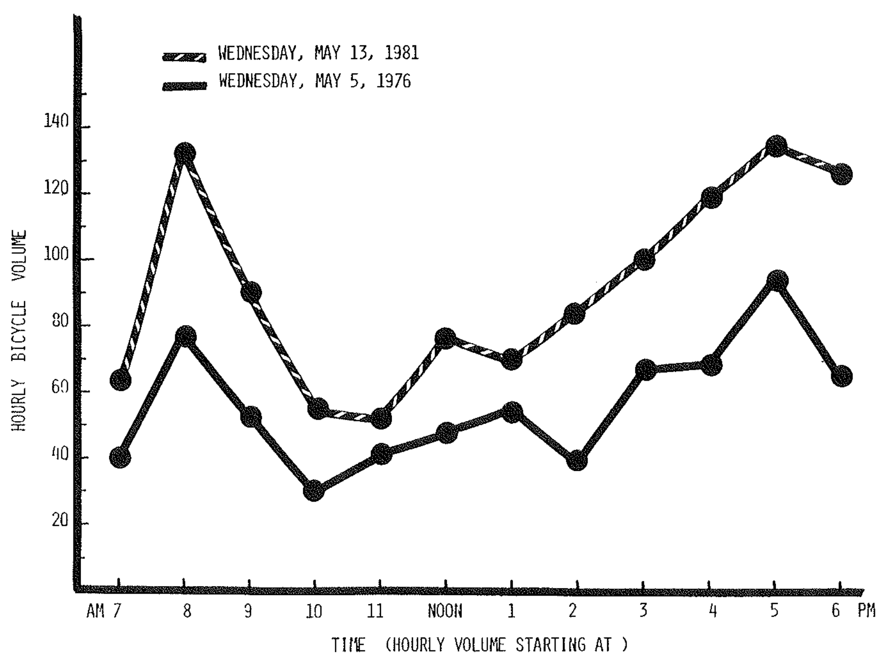


Table 1. Bicycle traffic counts, 7:00 a.m.-7:00 p.m.

Item	Coolidge Corner			Charles Circle		
	Tuesday, March 5, 1974	Wednesday, May 8, 1974	Wednesday, May 13, 1981	Wednesday, May 5, 1976	Wednesday, May 13, 1981	Avg
Weather conditions at 7 a.m.						
Temperature (°F)	56	46	50	51	50	
Wind velocity (mph)	25	16	14	15	14	
Sky	Overcast	Sunny	Partly sunny	Sunny	Partly sunny	
Volumes						
12-h total	479	959	1317	685	1110	
Morning peak hour						
Volume	62	106	184	78	141	
Time	8:00-9:00 a.m.	8:00-9:00 a.m.	8:15-9:15 a.m.	8:00-9:00 a.m.	8:15-9:15 a.m.	
Ratio to 12-h total (%)	13	11	14	11	13	12
Evening peak hour						
Volume	86	127	215	96	144	
Time	5:00-6:00 p.m.	5:00-6:00 p.m.	5:00-6:00 p.m.	5:00-6:00 p.m.	4:45-5:45 p.m.	
Ratio to 12-h total (%)	18	13	16	14	13	15
Ratio of evening to morning peak-hour volume	1.39	1.20	1.17	1.23	1.02	1.20

students. The direction of traffic flows during those periods suggests this. The midday trip might be utilitarian trips (shopping, going to lunch or meetings, making deliveries) or school trips (students attending late classes).

That the evening peak-period volumes are higher than those in the morning could be explained by the convergence of a greater variety of trip purposes later in the day. That is, commuters and full-time students are more or less alone in the morning but, in the late afternoon, they are joined by recreational cyclists and cyclists going shopping or to restaurants, night school, or meetings. In addition, some full-time students leave for school after the morning peak period but return home during the evening peak period.

NIGHT VOLUMES

No bicycle counts have been done in the Boston area between 7:00 p.m. and 7:00 a.m. We hoped that estimates of night volumes could be derived from the frequency of reported bicycle accidents by time of day. Therefore, the correlation of accident frequency and volumes was tested by using daytime bicycle volumes.

Bicycle accidents, as shown in Figure 10, do not correlate closely with bicycle volumes during the evening peak period, as shown in Figures 8 and 9. Although the 12-h bicycle counts indicate that evening peak-hour volumes are approximately 20 percent higher than morning peak-hour volumes, the accident reports suggest an evening accident rate 300 percent higher. There are several possible explanations for this. First, 35 percent of the accidents shown in Figure 10 involved bicyclists 14 years of age or younger. They presumably do much of their cycling after school and, therefore, are probably more likely to be involved in accidents during the afternoon and evening. At the same time, they presumably ride less often than does the average cyclist on major arterial streets, where the 12-h counts were done and, therefore, were not reflected in the volume figures being compared with the accident figures. Another explanation of the higher accident rate in the afternoon and early evening is the higher automobile volumes, also shown in Figure 10. In addition, darkness sets in before 7:00 p.m. during most of the year in Boston.

Bicycle volumes from 7:00 p.m. to 7:00 a.m. are probably lower than the number of reported bicycle accidents might suggest. Darkness would be one reason, especially because many cyclists are not well illuminated. More accidents that involve drunk drivers occur in the evening and early morning hours. Finally, because fewer cyclists are out at night, motorists may be less likely to expect them.

AVERAGE DAILY TRAFFIC

Peak-period bicycle counts done in 1975 and 1976 were used to develop average daily traffic (ADT) volumes for 1976. The counts were done on arterials where major bicycle traffic flows were judged to occur. The counts were spread out geographically so that volumes throughout the inner metropolitan area could be compared (see Figure 1). The peak-period counts were expanded by using accident records by hour of the day, day of the week, and month of the year.

Just as bicycle accident frequency and bicycle volumes did not correlate closely by time of day, they may not correlate by day of the week and month of the year either. However, accident data can be used to develop an approximate estimate of ADT. The errors inherent in this method are applied to all locations, so comparisons between locations should be valid.

It is impossible to determine from the data available how much error results from assuming a correlation between accident frequency and volumes by day of the week and month of the year. As shown in Figure 11, the weekday accident rates do not vary a great deal; Monday and Friday have the two highest rates. The Saturday rate is slightly lower than the weekday average but twice that of Sunday.

The proportion of inexperienced cyclists may be larger on weekends, and they may suffer higher accident rates. This may distort the correlation between accident frequency and volumes. Any such distortion may not be reflected in the data, however. These cyclists may stay away from motor vehicles, and accidents that do not involve motor vehicles are less likely to be reported.

Also shown in Figure 11 are accidents by month of the year. The general outline of the graph seems valid; it suggests that volumes increase from March through the summer and then decrease through February. The hazards associated with winter cycling suggest that the number of accidents might be higher in proportion to volumes then. On the other hand, only experienced cyclists, those presumably less likely to have accidents, are likely to be out in the off-season.

The 1976 ADTs estimated for selected routes in the inner metropolitan area are shown in Figure 12. The largest volumes occur near major schools: 1200 near Boston University, 1150 near Massachusetts Institute of Technology (see Figure 1 for assistance in identifying locales). There are also large volumes near Harvard Square, Cambridge, a commercial and office area adjacent to Harvard University. Several arterials that lead to the downtown Boston area have ADTs of around 400.

Remember that only selected arterials were counted, so not all bicycle volumes are shown. Also remember that these are average daily volumes. The volumes are therefore much higher than those in January and much lower than those in June.

USE OF A BICYCLE PATH

Counts were done on Thursday, May 21, 1981, to ascertain the relative use of three arterials and a nearby bicycle path. The path is in Boston, on the south side of the Charles River. The arterials are further south and parallel to the path. The evening-peak-period counts were done at the intersections of the path and the arterials with Massachusetts Avenue, which passes over the path but is at grade with the streets.

The 12-ft-wide path is totally separated from motor vehicle traffic for about 1 mile east and 2 miles west of Massachusetts Avenue. On each arterial, there are seven intersections between Massachusetts Avenue and a point 1 mile east. The grid street system does not continue west of Massachusetts Avenue, so the three arterials have different numbers of intersections in that direction. All of the arterials have parking.

Figure 13 shows westbound, evening-peak-period counts for the path and arterials. Only westbound volumes are shown because the predominant evening movement is westbound and because two of the arte-

Figure 10. Automobile volumes and bicycle accidents versus time of day.

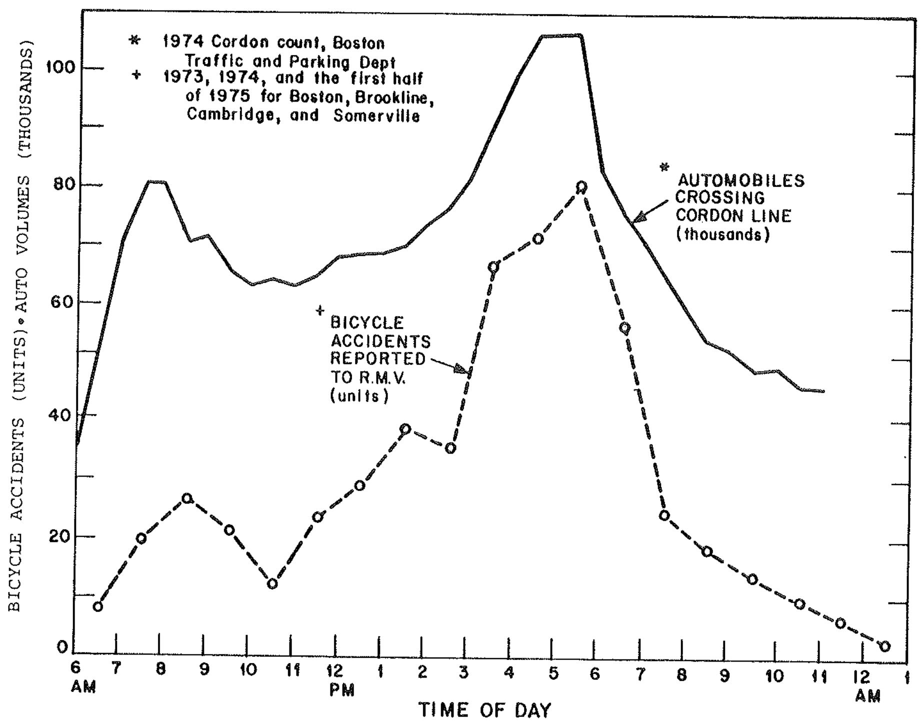
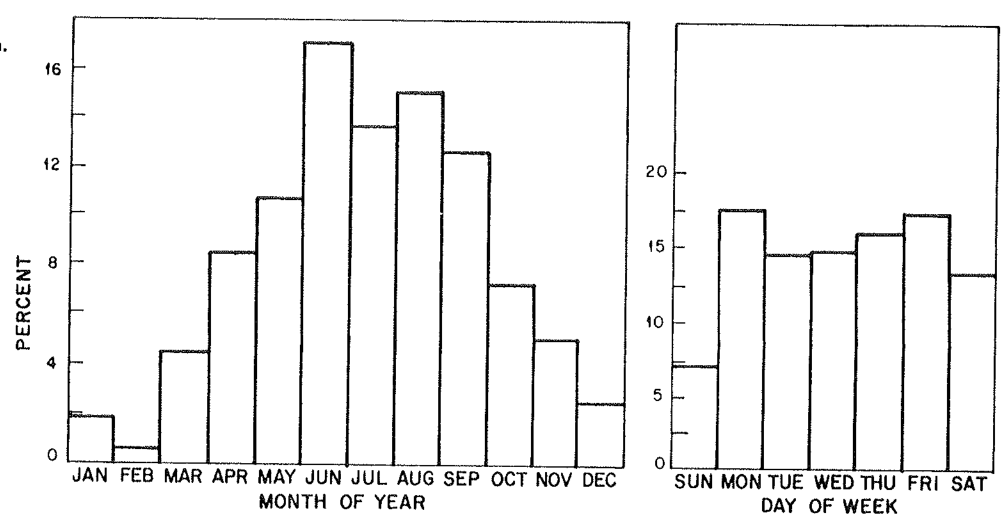


Figure 11. Reported bicycle accidents in metropolitan Boston.



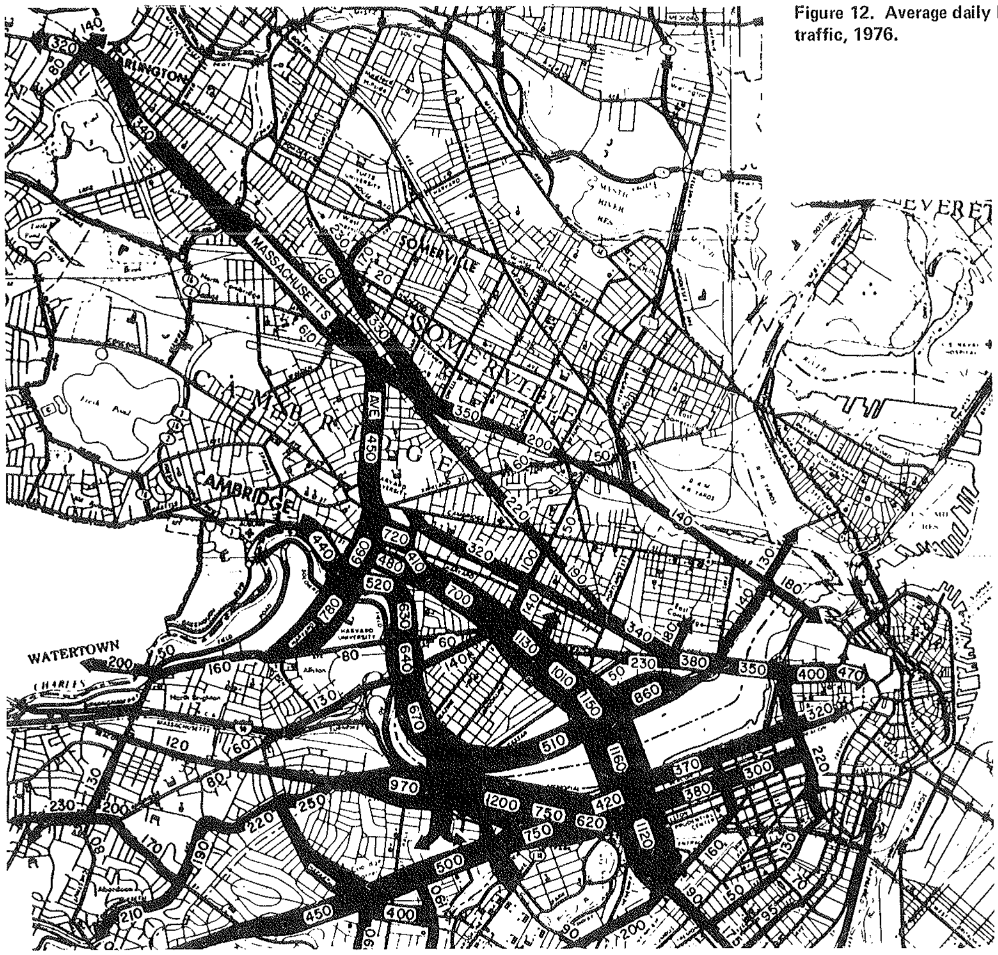
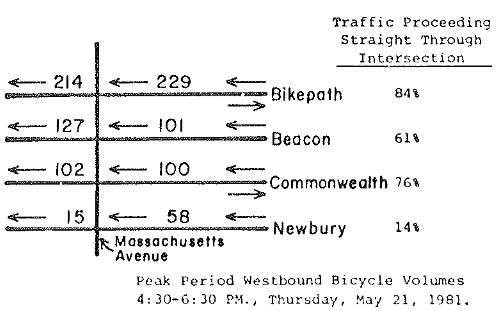


Figure 12. Average daily bicycle traffic, 1976.

Figure 13. Relative use of path and arterials.



rials, Newbury and Beacon, are one-way westbound. Commonwealth Avenue is two-way.

Of all the westbound bicycle traffic approaching Massachusetts Avenue, 47 percent was on the path and 53 percent on the three arterials combined. The right-hand column in Figure 13 indicates the percentage of the westbound cyclists who proceeded straight at Massachusetts Avenue. Of all westbound cyclists who did not turn at Massachusetts Avenue, 57 percent used the bicycle path and 43 percent used one of the three arterials. Cyclists on the path who wish to reach Massachusetts Avenue must carry their bicycles up a long flight of stairs. This helps to explain why, of all the westbound cyclists who turn at Massachusetts Avenue, only 25 percent used the path.

Cyclists must use ramps or stairs to get to the bicycle path, which is separated from the city by the limited-access, high-speed Storrow Drive. That virtually half of the cyclists counted did this suggests a preference to share the path with roller skaters and joggers than to share the arterials with parked and moving motor vehicles. If access to the path were more convenient, particularly at Massachusetts Avenue, its use would probably be markedly higher.

It should not be forgotten, however, that slightly more than half of the cyclists did not use the path. A survey would be necessary to find out why. Two reasons are probable: The arterials are more direct and the path is not designed for high speeds.

Bicycle volumes on the path on a holiday, presumably composed mainly of recreational cyclists, were even higher. A count was done on Memorial Day (Monday, May 25, 1981), four days after the count cited above. At the intersection of the bicycle path and Massachusetts Avenue, 675 cyclists were counted between 2:30 and 3:30 p.m. This volume is 60 percent higher than the evening peak-hour volume measured on the workday four days earlier.

FUTURE DIRECTIONS

This paper has presented information on bicycle traffic volumes in Boston. More information is needed, both to corroborate these findings and to ascertain how universal they are. The degree of use of a path or roadway, for example, is influenced by design standards and accessibility: How much do volumes actually vary with variations in these characteristics? How much do they vary with climate? Are the effects of weather similar in different regions? Are cyclists in Seattle as discouraged by light rain as those in Boston? Is the rate of increase of commuter cycling in Boston applicable nationally or dependent on such local factors as present cycling volumes, transit fares, and highway congestion?

A particular and important question is, What are the relationships between bicycle volumes and accident frequency by time of day, day of the week, and month of the year? Information on this would both improve our understanding of the causes of accidents and expand the usefulness of the data on bicycle-traffic volumes that have been and will be collected.

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Acceptance of Policies to Encourage Cycling

WERNER BRÖG

Research in the Federal Republic of Germany has rarely dealt with nonmotorized traffic. This applies to the collection of reliable behavioral data as well as to the application of these data in suitable planning models to forecast possible behavioral changes. Too little is known about the population's acceptance of such planning policies. Due to this lack of information, we can only guess about the effect of specific policies. But more important, since so little information is available, it is impossible to gear such policies to the needs, wishes, and interests of the persons affected by the policies. Thus, in order to encourage bicycle use in communities that have a medium or small population, many integrated measures must be used, and there are major differences of opinion concerning the concrete individual parts of such a bundle of measures and the effect of each specific measure. Frequently, attempts to solve this problem apply those instruments used by public opinion researchers. This paper wishes to demonstrate that this demoscopic approach is not suitable to deal with the topic discussed here. The paper presents an alternative approach to solve the problem—an approach that has the advantage of combining model design with estimates regarding the acceptance of different measures and deals with both in one concept—the situational approach. It can be shown that a whole series of measures must be integrated in planning if we wish that policies that encourage cycling be accepted so that more persons change to bicycles. Construction or extension of the bicycle infrastructure is of secondary importance, although important to stabilize those persons who have changed to the use of bicycles.

The bicycle is the healthiest and most ecologically oriented mode of transportation. However, although a number of cities in the Federal Republic of Germany have taken steps to encourage bicycle travel, the situation for cyclists is generally not particularly favorable. Therefore, the Federal Environmental Office decided to sponsor a model project called A Town for Cyclists (1) for towns that have a population of 30 000-100 000.

The model project will be concerned with the construction of a cycling infrastructure for all travel with person-powered vehicles (2). All cities included in the project will be involved in an intensive exchange of information. Planning seminars will be held to pass on knowledge and to share experiences with other participants. When the project is completed, the results will be evaluated and guidelines for planning will be made available to other cities (3).

Information concerning the quantity and quality of nonmotorized travel and measures to encourage such travel can be greatly improved by this model project; however, present knowledge concerning the acceptance of such measures by the populace is still limited. A more-precise analysis of a study on potential that has just been completed (4) can be of help here.

CONCEPT OF THE STUDY ON POTENTIAL

The study on potential was done in communities that had a population of 80 000 or less. Data were collected for three areas in the Federal Republic of Germany--one had a good, one a medium, and one a poor cycling infrastructure (5). The survey was done in two steps. In the first step, present travel behavior was determined on a specific sampling day in the spring of 1980 for the population surveyed. Of all trips made on this day, 16.4 percent were by bicycle. The percentage of individualized modes of transportation, which was 55 percent in the survey on potential, was extraordinarily high. As a result, the percentage of persons who use public transit was only 6 percent. This was due to the size of the communities selected to be included in the survey. Also note that, in selecting persons for inclusion in the survey, middle-aged persons were given preference and immobile persons were partly excluded. This survey was a pilot study and dealt primarily with persons who might use bicycles rather than with persons (e.g., elderly or immobile persons) who are unlikely to use bicycles. The second part of the survey dealt with the 84 percent of the trips that had not been made by bicycle on the day of sampling. The reasons why bicycles were not used were studied in intensive interviews in which all of the household members were present. Interactive measurement methods were used (6).

As a first step in the analysis, all those trips made with other modes were excluded if the one-way distance to the destination was more than 15 km. For these trips (a total of 24 percent), cycling would be a feasible alternative only in borderline cases. Given the conditions on the day of sampling, only 3 percent of the trips made could have been made by bicycle. Two-thirds of the trips made on the sampling day were restricted to the mode actually used on that day; due to constraints, it would have been impossible to use a bicycle (7).

The size of this group that has the option of using a bicycle on the sampling day increases from 3 to 30 percent when the restrictions on the day of sampling are eliminated. But, for the status quo conditions, this shows certain limits that should put a damper on too optimistic expectations.

If potential is thus determined, the results of demoscopic surveys will be viewed with scepticism. Demoscopic surveys assume a direct relation between stated opinions and actual behavior. However, generally, this is not so (8). Ninety-five percent of the interviewed mobile persons claimed, for instance, that they would be happy if their towns were to participate in a town for cyclists so that the bicycle network might be extended (44 percent). However, only 66 percent of the persons interviewed thought that riding bicycles had certain advantages.

On the other hand, a more in-depth analysis showed that only 49 percent of the persons interviewed who do not ride bicycles are honestly probicycle; the rest simply claimed to be in favor of bicycles. The present maximum potential for persons to change their modes in favor of bicycles is, as has already been mentioned, 30 percent--a respectable figure but considerably less than the demoscopically determined values.

In the project quoted above, the likelihood that persons would change from public transportation to the bicycle was the greatest and least likely was that persons would change from the use of cars to the use of bicycles. This insight is important, for if one assumes that towns are probably not particularly eager to lose their public transit passengers and if one remembers that one reason for encouraging cycling is to reduce congestion caused by cars, then it is obvious that, if potential is not reliably determined, the use of bicycles might well rapidly increase but the increase might be at the cost of the wrong mode.

This means that important measures to increase bicycle use might possibly, in light of traffic congestion caused by cars, apply restrictive measures to car use. Because such restrictive measures were probably not considered by the persons interviewed, they might lead to a rapid change in public opinion. However, a sensibly executed simultaneous study could identify these negative processes before it is too late and counteract them with appropriate measures.

Therefore, when attempting to determine the acceptance of such measures, it is of utmost importance not to simply rely on stated opinions. Rather, the estimate of likely behavioral reactions is a much more reliable way to identify the acceptance of planning policies.

PROBLEMS OF EMPIRICALLY MEASURING POSSIBLE CHANGES IN BEHAVIOR

The use of demoscopic measurement methods is problematical when estimating possible changes in behavior. Alternative research concepts have been developed in the meantime and their application has already been tested. So-called situational analysis (9) is an alternative of this sort.

Beside a new model philosophy (10), such an approach requires empirical data of a particular quality. In order to acquire such data, a combination of different measurement methods must be used, whereby the qualitative methods, especially, must fulfill certain requirements. However, new methodological advances have been made in this area--the so-called interactive measurement methods.

By using these measurement methods, the areas of information that are important here can be covered and the necessary data can be collected:

Data	Description
Basic descriptive	Sociodemographic characteristics of individuals and their households, complete description of all transportation modes available in the households, detailed information on all bicycles present within household
Behavior descriptive	Complete activity patterns of all household members for specific, defined periods of time (including choice of routes); general activity budget of household members for specific periods of time; general action radii of household members
Contextual	Division of chores and duties within household and possibility of changing the internal household organization, objective and subjective availability of vehicles in general and for those activities that have actually taken place, options and constraints that allow for or exclude use of bicycles
Explanatory	Behavior can be explained as the result of individual's subjective interpretation of experience, perception, orientation, attitude, and reaction
Experiential	Basic experiences with alternative modes, concrete experience with alternative modes for activities that have already taken place, specific experiences with bicycle for different activities and destinations
Perceptive	Perception of individual situation, cycling infrastructure, bicycle's characteristics, and riding bicycles
Orientational	Influence of general values on behavior, influence of peer group, dominance of particular household members
Attitudinal	Subjective willingness to use bicycle in general and for particular activities, subjective experiences with bicycles, prestige value of riding bicycles
Reactional	Possibility of reorganization of individual activity patterns so that bicycles can be increasingly used given present conditions, working out ways of increasing extent to which activities can be thus reorganized, reorganization of individual activity patterns when external conditions have been thus changed

By using these types of data (after the data have been adequately coded), one can estimate how large the present potential for bicycle use is relatively accurately and with which measures this potential can or cannot be attained. The much more common variation of the (demoscopic) what if... questions, however, cannot attain this goal.

However, this comprehensive data requirement plan and its relatively complicated survey technology may leave one with the impression that it is exaggerated. This impression would not do justice to a major aspect of such research concepts. If alternative transport planning wishes to adapt itself better to the wishes and needs of the affected population, then it must also use those research methods that have the same goal (11). "Reality exists for empirical science only in the empirical world, can only be sought there, and can only be verified there" (12).

This goal, however, cannot be attained with empirical concepts that interpret surveys as a stimulus-reaction mechanism. Rather, explorative and interactive measurement methods must be used in addition to quantitative measurement methods. Such a "qualitative methodology favors an approach to study the empirical social world that demands that the researcher interpret the real world from the perspective of the subject being studied" (13).

USE OF SITUATIONAL APPROACH TO DETERMINE POTENTIAL FOR CHANGE

The majority of mobile persons are basically free to use bicycles if they wish. That they do not do so is caused, to a large extent, by their personal attitudes toward the use of bicycles. This insight clearly shows the limitations of forecasting mode split with conventional methods. Forecasting models that have been used in transport planning up until now are not able to depict subjective attitudes and possible changes adequately.

An approach is needed that is oriented to the individual and in which specific out-of-house activities are grouped into activity patterns. These activity patterns are reflected in the situational context of the given households. Combined with the household-oriented activity pattern, substitution, reorganization, and flexibility can broaden the approach considerably (14). When this approach is used, travel behavior can be better understood and explained.

Because transportation planners need information that deals with trips, the basic unit of a model for mode choice must be the specific trip made by the individual. These trips should not be viewed in isolation but should always be seen in their relation to trip chains, activity patterns, and the activity programs of the individuals and their households (15). The trips are not made in a vacuum. Environmental conditions influence the realization of the trips, the trips chains, activity patterns, and activity programs. However, if out-of-house mobility is viewed as a derived demand, then the activity programs also limit the situational context for this mobility, which must be included in a pertinent model-like depiction. If one wishes to understand mobility in relation to the above, then all the factors must be identified that determine behavior in these situational contexts. A simple use of sociodemographic variables is insufficient even if these variables are seen in relation to some characteristics of the existing infrastructure (16).

Note that persons do not perceive their situational contexts as they objectively exist (10). But, since subjectively experienced situations determine behavior, a realistic model must also include subjectively perceived variables. This means that, in every single instance, one must investigate how the given macrostructure is reflected in the microstructure of the pertinent individuals. Therefore, perception is one of the keys to relate macrostructure to microstructure. However, these mechanisms of perception have not yet been studied comprehensively enough to explain them sufficiently.

If one wishes to explain observed (mobility) behavior, it is necessary to classify the given subjective behavioral situations into dimensions that determine behavior. However, since the reasons that cause out-of-house mobility are highly complex (11), different dimensions must be selected to take the feasibility of their application into consideration. It seems that five dimensions are sufficient to determine bicycle potential (7):

Dimension	Description
Option of using bicycle	Bicycle available, trip <15 km
Constraints against using bicycle or requiring use of specific mode	Transport of baggage, weather conditions, health, car needed at work
Perception of route	No bicycle paths, too many hills, dangerous intersections
Perception of riding bicycle and time required	Too slow, too tiring, clothes get dirty
Subjective willingness	Willing to use bicycle mode

General behavior, as well as decisionmaking pertaining to modal choice, are not constant. This means that behavior observed on one day will not necessarily be repeated on the next day. The solution usually used for this problem (i.e., to collect information on relevant variables over a period of time) is costly and methodologically very difficult. If the latter problem is perhaps even more important than the prior, it is rarely considered. Sensitization (17), which was developed for use in the situational approach, offers an alternative that is of almost equal value. By defining threshhold groups, those trips in each dimension can be identified for which the factors that determine mode choice are temporally variable: Observed behavior is generalized.

An aggregate evaluation of these dimensions for all trips that were not made by bicycle (in the study on potential) results in the observed behavior on the day of sampling and the generalization of this behavior that is depicted in Table 1.

Such an approach is only possible if each case is considered individually and remains an independent unit of action in modeling. This is important because the individual interrelation of the different categories and threshhold values are what make possible identification of those options that are actually open to persons. An individualized approach of this sort also makes it possible to identify persons who have comparable decisionmaking situations. These decisionmaking situations are the first key to aggregation, which is naturally also necessary in this type of model approach.

Situational groups are the basis for this type of aggregation. Note that situational groups and situational contexts are comparable; they classify trip categories not persons. Situational groups are deterministic (i.e., behavior of persons in these groups is predetermined) if they do not permit certain types of behavior due to specific conditions, whether or not individuals in the groups wish to realize a particular behavior for their trips. In the area of mode choice, most of the situational

Table 1. Dimensions pertaining to use of bicycle.

Dimension	Behavior Observed on Day of Sampling (%)		General Behavior (%)	
	No	Yes	No	Yes
Option of using a bicycle	14	86	11	89
Constraints make bicycle use possible	70	30	41	59
Due to perception of route, bicycle is acceptable	20	80	14	86
Due to perception of characteristics of bicycle itself and time needed to travel by bicycle, bicycle is acceptable	43	57	29	71
Subjective willingness to use bicycle	81	19	51	49

Note: Only trips of less than 15 km not made by bicycle were considered.

groups are determined either by objective reasons (no alternative), personal reasons (car needed at work), or by subjective reasons (persons are insufficiently informed or have subjective aversions to certain modes). But usually, a smaller number of trips remains for which use of an alternative mode is subjectively as well as objectively possible. This situational group (group with options) defines the maximum potential for changed behavior given status quo conditions.

From an aggregate point of view, one could have assumed that almost every other trip previously not made by bicycle could be a potential bicycle trip due to the positive attitude of the respondents (this is similar to the demoscopic approach that is criticized in this paper), an individualization of general options shows that the present potential (i.e., without the implementation of any new policies) is actually much smaller (Figure 1).

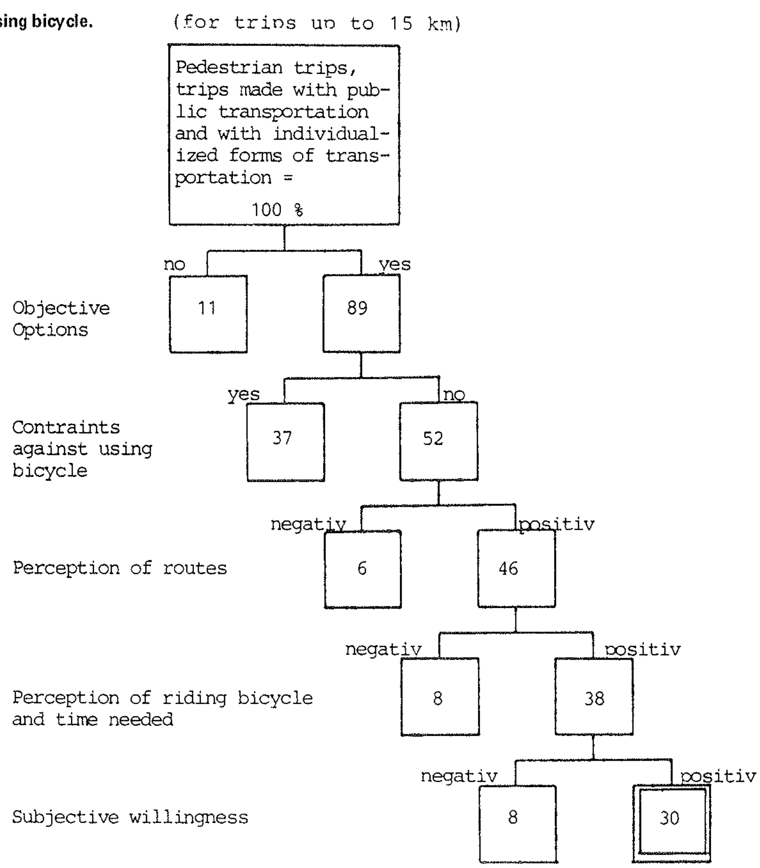
According to this categorization, which results from the individual combination of the five dimensions, one can differentiate six situational groups, which are characterized by Roman numerals. Groups I-V have a deterministic character given the (generalized) status quo conditions; only situational group VI has the unlimited option of using bicycles given existing conditions.

In this approach, individual (behavioral) situations are seen as factors that influence individual options and constraints. This approach is also based on the insight, however, that individual mode choice follows a unique, subjective logic that is frequently at odds with the researcher's, planner's, or politician's more or less externally imposed rationality. This does not mean that the individual's choice of mode is not rational but that it is subjectively rational. The regularities of this subjective rationality are naturally manifold and have not yet been studied comprehensively (10).

If one analyzes mode split by using the situational approach, then one can differentiate between three types of mode use:

1. Persons who cannot change to an alternative mode due to objective conditions or due to constraints that can be changed only with great difficulty (situational groups I and II);
2. Persons who are basically able to use another mode but do not do so due to inadequate information about the mode, poor perception of, or a negative attitude toward the mode; thus, the mode is not subjectively an option open to them (situational groups III, IV, and V); and

Figure 1. General options of using bicycle.



3. Persons who view the alternative as subjectively possible yet do not use it (situational group VI).

The sizes of these groups vary greatly according to mode and to the specific spatial and infrastructural conditions. However, the potentials presented in this paper give one an idea of the acceptance of measures to encourage the use of bicycles.

ESTIMATING DEGREE TO WHICH DIFFERENT MEASURES TO ENCOURAGE CYCLING ARE ACCEPTED

The situational groups also give a structural base with which to work out planning measures and to estimate the effects of these measures. This is possible because the value of all of the dimensions is determined for each trip. If, for example, the use of the bicycle is not possible for a specific trip that has been made by car because the household did not have a bicycle, an additional check was made to see if there were other constraints that also necessitated use of the car. This makes it possible to identify those car drivers whose transportation options would not have been increased even if bicycles had been available.

Measures directly affect only the external situation and can affect the resulting behavior only indirectly. An approach of this sort makes it possible to identify those situations for which a change is theoretically possible if certain measures are implemented--or to put it more concretely, which of the trips made by a specific situational group whose mode choice is of a deterministic character can be included in the group that has options. The step required here, the so-called dynamization (17), tells one the size of the maximum potential for reaction to a planning measure that is to be studied.

The inclusion of a trip in the group that has options only means, however, that a change of mode is possible, not that it will actually occur. In order to estimate the likely reactions to a measure, it is necessary to determine the given responsiveness for the trips included in the group that has options. This step of the study combines probabilistic elements in the basic model structure with the deterministic elements that have already been discussed. However, the probability that certain reactions will occur, which are determined with this approach, are based on the subjective rationality of the actors and cannot be compared with the utility function of an econometric approach, for instance.

The structuralization of the individual (decisionmaking) situations offers one the possibility to determine the likely potentials for different areas of measures and thus, the acceptance of measures in these areas. The different areas of measures are identical with the five dimensions and, in relation to possible measures, they can be roughly formulated as follows:

1. Objective option--basic availability of bicycles (e.g., making it possible to rent bicycles);
2. Constraints--only constraints that pertain to the bicycle itself (baggage transport needed, weather conditions) are referred to here, since other constraints (passengers, car needed at work, complex trip chains) cannot be dealt with by the measures discussed in this paper;
3. Routes--improvement of the bicycle infrastructure;
4. Riding bicycles--public relations work geared to clarifying misconceptions and incorrect perceptions; and
5. Subjective willingness--increase the number of persons willing to change to use of bicycles by creating a climate of opinion in the community that is favorably disposed to bicycles.

For each of these areas of measures, we can determine the maximum potential for change, which is the upper limit for possible reactions when all of the necessary measures have been adapted (from the point of view of the individuals affected) in this area (Table 2). However, this upper limit is a theoretical value that will never, in fact, be attained. Those reactions that are actually to be expected can be estimated by using the responsiveness coefficients that are determined interactively. This coefficient shows how high the percentage of likely mode change is from the given potential.

When determining the number of persons who react, we should not forget that the values in percentages of the given potentials only pertain to a limited group of travelers (i.e., nonbicycle trips of 15 km or less). For this reason, the potentials in Table 2 are also calculated for all trips (including bicycle trips); with the help of the responsiveness coefficients, the behavioral changes were determined (as a percentage of all trips).

This calculation, which is of great value for forecasting, serves here, however, as the basis for estimating the acceptance of measures. An acceptance index was established for this purpose. It relates the bicycle use that can be expected when a policy has been introduced to the present share of bicycle trips. The present share of 16.4 percent was set equal to 1.00. An acceptance index of 2.00 would then mean that twice as many persons would use bicycles, and an acceptance index of 1.00 would indicate that no change had taken place (Table 2).

The different acceptance indexes show that, even if no new measures are adapted, an increase in cycling would still occur. This is because the current climate of opinion in Germany is probicycle. Due to further developments since the study was done

Table 2. Maximum potential and acceptance of different measures.

	Potential A		Potential B		
Areas of Measure	In Relation to All Other Modes ≤15 km (%)	Responsiveness Coefficient[a]	In Relation to All Trips With All Modes (%)	Likely Reactions (%)	Acceptance Index[b]
Status quo[c]	30	0.03	18	0.5	1.03
Bicycle availability	32	0.13	19	2.5	1.15
Bicycle constraints	34	0.21	20	4.2	1.26
Infrastructure	34	0.14	20	2.8	1.17
Public relations work to clarify misconceptions	33	0.16	20	3.1	1.19
Community climate	38	0.14	23	3.2	1.20

[a]Percentage of likely responses from potential A.
[b]Likely reactions to potential B in relation to present bicycle share (16.4 percent = 1.00).
[c]Status quo included in the following areas of measure.

Table 3. Maximum potential and acceptance of different measures when climate of opinion in community is positive.

Areas of Measure in Combination with Positive Climate of Opinion[a]	Potential A: In Relation to All Other Modes ≤ 15 km (%)	Potential A: Responsiveness Coefficient[b]	Potential B: In Relation to All Trips With All Modes (%)	Potential B: Likely Reactions (%)	Potential B: Acceptance Index[c]
Bicycle availability	42	0.27	25	6.7	1.41
Bicycle constraints (baggage, weather)	56	0.25	33	8.3	1.51
Infrastructure	43	0.23	26	5.9	1.36
Public relations work to clarify misconceptions	46	0.30	27	8.2	1.50

[a]Measures assume that a positive climate of opinion exists. Status quo conditions are assumed in measures.
[b]Percentage of likely responses from potential A.
[c]Likely reactions to potential B in relation to the present bicycle share (16.4 percent = 1.00).

Table 4. Maximum potential and acceptance of different measures when their effects are viewed cumulatively.

Areas of Measure[a]	Potential A: In Relation to All Other Modes ≤ 15 km (%)	Potential A: Responsiveness Coefficient[b]	Potential B: In Relation to All Trips With All Modes (%)	Potential B: Likely Reactions (%)	Potential B: Acceptance Index[c]
Status quo	30	0.03	18	0.5	1.03
Climate of opinion in community	38	0.14	23	3.2	1.20
Public relations work to clarify misconceptions	46	0.30	27	8.2	1.50
Infrastructure	52	0.35	31	10.8	1.66
Bicycle constraints	64	0.33	38	12.6	1.77
Availability of bicycle	73	0.32	43	13.9	1.85
Long-term effects on all other persons	100	0.24	60	14.3	1.87

[a]Each measure is cumulative and assumes the use of all preceding measures.
[b]Percentage of likely responses from potential A.
[c]Likely reactions to potential B in relation to the present bicycle share (16.4 percent = 1.00).

(especially since the price of gasoline keeps going up), one can assume that the current general acceptance of the bicycle will increase even more in the future.

The goal of well-considered community planning can no longer be to implement specific policies in isolation. Thus, the goal of the town for cyclists project is also to improve the climate of opinion in the communities in favor of cycling in order to increase the effectiveness of other measures. This concept can be demonstrated by using the figures available. If one wishes to pinpoint the acceptance of specific types of measures when combining them with positively influencing the climate of opinion within communities, then the potential grows considerably and the responsiveness, and thus the acceptance indexes, grow cumulatively (Table 3). Measures that deal with the bicycle itself (e.g., baggage-loading facilities and weather protection) and public relations work (clarification of misconceptions and correction of negative perceptions) are more important than improvements to the infrastructure.

However, one should be careful not to come to a false conclusion here. The responsiveness and acceptance of measures presented here deal with the possible use of the bicycle (i.e., with mode change). Irrespective of the fact that other aspects are not taken into consideration here (e.g., improvement of neighborhood quality), this does not mean that an improvement in the bicycle infrastructure does not also play an important, if not decisive, role in stabilizing the new potential of those persons who change mode. This becomes clear if one considers the cumulative effect of all of the different areas of measures (Table 4). This evaluation also shows that, in the towns studied (whose populations range from 50 000 to 80 000), almost three-fourths of all trips made by the mobile population (for the most part between 20 and 60 years of age) could possibly be made by bicycle. The share of trips made by bicycle could, at least in the long run, almost double. These upper limits, of course, cannot be attained or can only be attained over a longer period of time, given conditions as they are now. Previous experiences with comparable studies (18) show that almost half of this maximum potential can be attained, when appropriate measure are taken, in the not too distant future. For a town for cyclists project, this means that the acceptance index will be approximately 1.40. This acceptance index only pertains to the willingness to change mode if measures are taken to encourage cycling. A number of studies, however, have shown that a comparable effect could result (perhaps even more quickly) if restrictive measures pertaining to car use are taken. In this case, even in larger cities that have good public transportation, the number of car drivers who would change over to bicycles is about twice as large as the number of car drivers who would change over to public transportation (15).

CONCLUSIONS

The results of the situational analysis, the determination of potential, and the estimate of acceptance lead to the following conclusions:

1. A potential for the increased use of the bicycle can be recognized. This potential can be attained by implementation of specific measures.
2. This potential exists for all five of the different areas of measures. The spectrum of those measures that can achieve success is thus larger than has frequently been assumed.
3. Bicycles can frequently not be used due to several simultaneous constraints; thus, combined measures are particularly effective.
4. Bicycles are rarely not used only because persons perceive the bicycle infrastructure to be

inferior. Thus, when the infrastructure is improved but no other measures are instituted, the effect is minimal. However, when other measures are taken and the bicycle network is also improved, a considerable potential can be attained.

5. For many trips, bicycles cannot be used due to constraints. Improvements, especially weather protection and baggage-transport facilities, could make it possible for more persons to use bicycles. Thus, it would be worthwhile to work on improving the bicycles themselves.

6. Options are rarely solely affected by the fact that persons are unwilling to use bicycles. This means that a public relations campaign, in itself, will be of only limited value. Only when combined with other measures will a public relations campaign prove to be effective.

7. The largest potential can be found among those persons subjectively willing to use bicycles given status quo conditions. This group is subjectively and objectively in favor of bicycles but does not use them. Among this group, measures that restrict car use would be more effective than measures to encourage bicycle use.

All in all, the approach described above to determine the potential for reactions also offers one a usable, understandable, and methodologically valid approach to estimate the acceptance of measures to encourage bicycle use. This naturally also pertains to similar topics, especially those measures to reduce traffic (such as automobile-restricted lanes, pedestrian areas, and other similar efforts) that are currently so popular in the Federal Republic of Germany.

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