HIGHWAY RESEARCH RECORD

Number 384

Traffic Records

4 reports
prepared for the
51st Annual Meeting

Subject Areas

51	Highway Safety
52	Road User Characteristics
53	Traffic Control and Operations

HIGHWAY RESEARCH BOARD

DIVISION OF ENGINEERING NATIONAL RESEARCH COUNCIL
NATIONAL ACADEMY OF SCIENCES—NATIONAL ACADEMY OF ENGINEERING

Washington, D.C. 1972

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ISBN 0-309-02054-9

Price: $2.00

Available from

Highway Research Board
National Academy of Sciences
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

CONTENTS

FOREWORD

Adequate traffic records are a vital key to planning and conducting effective accident prevention programs. The papers in this RECORD are concerned with the use of traffic records as a planning tool.

In the first paper, Baker describes the evaluation of the traffic conflicts technique that was originally developed by the General Motors Research Laboratories. The technique was developed primarily as a tool for measuring the traffic accident potential at intersections. In order to field test the technique, the Federal Highway Administration in cooperation with the state highway departments in Washington, Ohio, and Virginia conducted field evaluation studies. The data compiled from the studies tend to support the finding that conflicts and accidents are associated.

Burke and Leland outline the concept of a compatibility file to serve as an interface between transportation planning network files and other data files that are referenced by route number and cumulative mileage. Historically, the physical inventory phase of a transportation study has presented the planner with a costly task in terms of time and money.

The compatibility file, as an information retrieval system, may be used to relate computerized transportation study traffic networks to the actual physical inventory files maintained by the specialty divisions (traffic, highway needs, planning, and accident prevention) in a department of highway or transportation.

A study to examine and compare the speed profiles of traffic approaching a traffic signal, under six different signal displays, is the subject of the paper by Bleyl. The study was addressed to a number of questions related to the speeds and profiles of traffic approaching traffic signals: How do drivers respond to different signal displays? When do they speed up? When do they slow down? When do they maintain their speed? How far in advance of the signal do drivers respond?

Drivers at the study location entered the intersection more cautiously with a green traffic signal indication or with a flashing yellow indication than they did with no signal installed. When approaching a red signal indication, drivers did not begin to slow down until they were approximately 500 ft from the signal. These and other speed profiles are noted in the paper.

Near-miss traffic events have been considered but not adopted as a traffic safety tool because of the high degree of subjectivity involved with their identification. Hayward describes a scale of danger that can be applied to a traffic event to facilitate objective measurement and subsequent detection in near-miss situations.

The unit proposed for the danger scale is the time measured until collision between two vehicles involved in an unsafe event. The measure, computed from films taken with the Federal Highway Administration's Traffic Sensing and Surveillance System at an urban intersection, is suggested as an adequate unit to rate the danger of almost any traffic event.

AN EVALUATION OF THE TRAFFIC CONFLICTS TECHNIQUE

William T. Baker, Federal Highway Administration

The traffic conflicts technique, as developed by General Motors Research Laboratories, was evaluated by the Federal Highway Administration in cooperation with the state highway departments of Washington, Ohio, and Virginia. In addition to a field test of the technique, an attempt was made to find whether there is a statistical relation between traffic accidents and traffic conflicts. Conflicts were counted at 392 intersections before improvements were made and 173 intersections after construction of the improvements. It appears that those characteristics of intersections that contribute to accident causation can be more readily exposed by using conflicts than by using conventional accident analysis techniques. This may be especially true at low-volume rural intersections. Because of this ability to provide more precise information, lower cost remedial actions should result. Correlation coefficients were calculated for bivariate populations of number of conflicts and number of corresponding accidents. The compiled data tend to support a finding that conflicts and accidents are associated.

• THE traffic conflicts technique was developed by the General Motors Research Laboratories in 1968 (1). The advantages of a tool such as this for use in the traffic accident analysis field were obvious; however, because only limited field testing had been done, more extensive testing was needed to determine the correlation between actual traffic accidents and the measurements derived by using this technique. Field testing also was necessary to prove the worth of this technique as a means of gathering data usable in the design process. The Federal Highway Administration in cooperation with three state highway departments set up studies to carry out the necessary field testing.

TRAFFIC CONFLICTS TECHNIQUE

The technique was developed primarily as a tool for measuring traffic accident potential at intersections. A traffic conflict occurs when one driver takes evasive action by braking or weaving to avoid what he believes to be an impending collision with another vehicle. The objective evidence of a traffic conflict is a brake-light indication or a lane change effected by the offended driver. The brake-light indication or the lane change, as well as the offending vehicle, must be observed before a conflict can be recorded. Figure 1 shows four common types of conflict situations. In each case, vehicle No. 3 is the observation vehicle, No. 1 is the offending vehicle, and No. 2 is the offended vehicle. Criteria have been defined for over 20 specific conflict situations at intersections, details of which can be found elsewhere (1).

When a traffic conflicts count is taken, observations from two opposite intersection approach legs are recorded in 1 day by a two-man team using a single vehicle. One observer is responsible for counting conflicts, while the other is responsible for recording volume data. Fifteen-minute data samples are taken alternately on each intersection approach leg from the observation vehicle, which is parked on the side of the roadway about 100 to 300 ft back from the intersection. The team is allowed 15 minutes

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after each sample count to record the data and to move to the opposite approach. The team alternately surveys the two approach legs throughout the day.

PROCEDURE

In June 1969, the Federal Highway Administration contracted with the Washington Department of Highways to conduct a traffic conflicts study. Subsequently, contracts were negotiated with the Ohio Department of Highways and the Virginia Department of Highways to conduct similar studies. The contracts provided funds for the counting of conflicts at a minimum of 400 intersection approach legs in each state. The counts were to be made both before and after a "spot improvement" type of change had been made, if possible. Two engineers trained a supervisor and two crews in each state to ensure that the technique was applied in the same manner in all three states.

The states' role in the overall evaluation of the traffic conflicts technique was to determine whether conflicts data provided the kind of information from which the need for safety improvements could be determined. They were to make the counts, compare them to the actual accident data, and determine whether the traffic conflicts technique was advantageous. After each location was analyzed, the conflicts data were to be sent to the Federal Highway Administration, Office of Traffic Operations, for statistical analysis.

The primary objective of the statistical effort was to determine whether there is a significant correlation between conflicts and accidents. The results of this determination are given in Table 1.

It was beyond the scope of the studies to require that the improvements be made only on the basis of conflicts counts because this would probably have necessitated the funding of the improvements themselves. Instead, each state was to make counts at intersections that were already scheduled for improvement as the result of analyses based on accident experience. Because the traffic conflicts technique was developed as a tool for measuring traffic accident potential, it was hoped that the conflicts counts would point up the same safety deficiencies as did the routine accident analyses.

RESULTS

A total of 392 intersections was counted before improvements were made, and 173 intersections were counted after construction of the improvements. In terms of intersection approaches, 886 were counted before and 420 after improvements were made. At least 1 month was allowed after completion of construction before the after counts were taken.

Field Evaluation

Each of the three states reported that the technique provided the kind of information needed as a basis for design of safety improvements. It was reported that, in most cases where there was an adequate history of accident experience, the conflicts counts not only verified the accident analyses but often provided more insight as to the existing hazardous conditions. One example is that of an intersection approach where the cause of a number of ran-off-the-road and rear-end accidents could not be determined. The conflicts counts supported the probability that some drivers chose to leave the roadway rather than become involved in a rear-end accident. Thus, a significant amount of evidence was compiled that indicated that there was the potential for a rear-end accident grouping that was not apparent from the accident collision diagram. The ran-off-the-road accidents were, then, likely results of the same hazardous condition that produced the rear-end accidents.

One of the states found that with slight modification the conflicts technique could be applied to locations other than intersections. The technique is mainly oriented to conflicts between vehicles; therefore, for applications other than intersections, conflicts of single vehicles with highway geometrics have to be defined. For example, the various maneuvers that drivers perform when confronted with a complex gore area may include several types of single-vehicle conflicts not now included in the traffic conflicts technique.

Figure 1. Four common traffic conflict situations.

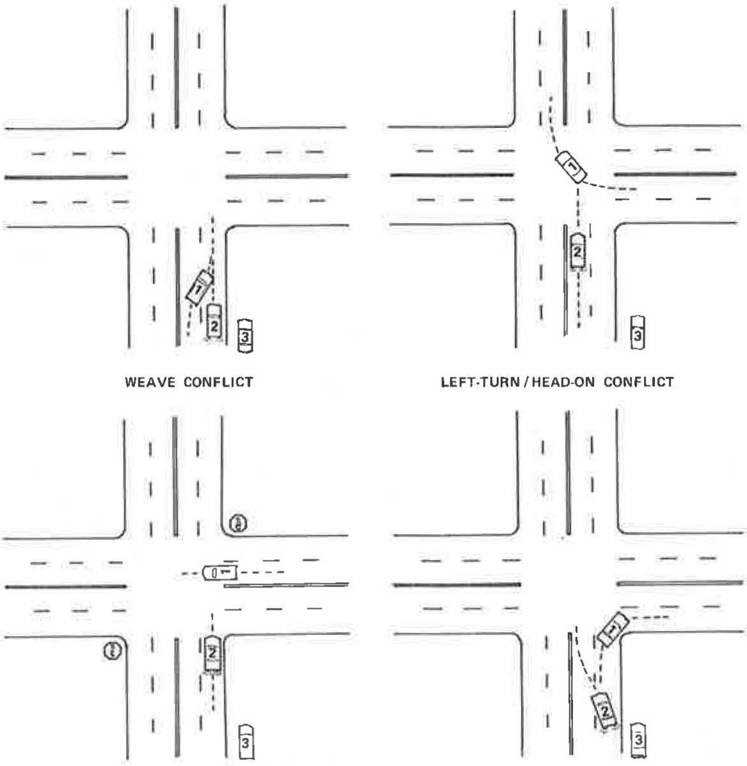


Table 1. Correlation coefficients (r) for T and 4-legged right-angle intersections.

Intersection	Conflict-Accident Situation: Weave	Left-Turn Head-On	Cross-Traffic	Rear-End	All Maneuvers	Critical r	Sample Size
Signalized							
T	-0.207	-0.128	-0.170	0.075	-0.172	±0.532	14
4-legged right-angle	0.360[a]	0.661[a]	0.209[a]	-0.018	0.410[a]	±0.179	122
All[b]	0.402[a]	0.615[a]	0.136	-0.017	0.326[a]	±0.160	157
Nonsignalized							
T	0.294[a]	0.432[a]	0.830[a]	0.410[a]	0.837[a]	±0.205	94
4-legged right-angle	0.159	0.459[a]	0.602[a]	0.213[a]	0.653[a]	±0.192	106
All[b]	0.276[a]	0.453[a]	0.665[a]	0.295[a]	0.671[a]	±0.130	235
All combined[c]	0.356[a]	0.546[a]	0.429[a]	0.154[a]	0.458[a]	±0.100	392

[a]Indicates statistical significance at the 5 percent level.
[b]Includes other intersection types such as skewed and multileg as well as T and 4-legged right-angle.
[c]Composed of all signalized and nonsignalized intersections.

Table 2. Conflict-opportunity and accident-conflict ratios.

Intersection	Conflict-Accident Situation: Weave	Left-Turn Head-On	Cross-Traffic	Rear-End
Conflicts per 1,000 opportunities				
Signalized 4-legged right-angle	51	28	15	29
Nonsignalized 4-legged right-angle	64	28	26	26
All combined[a]	65	28	25	25
Accidents per 100,000 conflicts				
Signalized 4-legged right-angle	7	20	56	3
Nonsignalized 4-legged right-angle	7	9	16	1
All combined[a]	6	15	20	3

[a]Includes other intersection types such as skewed, multileg, and T as well as signalized and nonsignalized 4-legged right-angle.

It was reported that the traffic conflicts technique seems especially applicable to low-volume rural intersections where the accident reporting level is usually low. Collision diagrams prepared for these locations are not very revealing, although a hazardous condition may exist that would be evident through a conflicts count.

Because analysts are often better able to pinpoint hazardous conditions from conflicts data than from collision diagrams, the remedial action taken may be lower in cost than that suggested by collision diagrams. One state told of situations where conflicts counts pointed to low-cost improvements in the $200 to $500 range that were not well received by officials in small incorporated areas. Without the conflicts data, it would have been difficult to show that these low-cost and relatively undramatic improvements would probably be more effective than more costly work such as the installation of traffic signals.

All of the states encouraged their counters to record unusual maneuvers, events, or situations that might affect the safe operation of the intersection. These comments and diagrams proved to be very valuable in some instances. One example given was that of a signalized intersection at which a significant percentage of the volume on one approach was cutting across an abandoned gas station on the red signal, thereby producing conflicts with the through traffic. This situation was not evident from the few available accident reports and was not suspected until actually observed during the conduct of a conflicts count.

Correlation Coefficients

One of the study objectives was to test the hypothesis that traffic conflicts are associated statistically with accident frequency. It was hoped that significant correlation coefficients might be found so that future corrective action might be taken at intersections selected on the basis of conflicts counts. Bivariate populations were described, with the number of conflicts in each category used as one variable and the number of corresponding accidents used as the other variable. [See the Appendix (2) for a description of statistical technique.]

In this analysis, the null hypothesis that there is no correlation between numbers of accidents and numbers of conflicts was tested. Correlation coefficients were calculated for the rear-end situation and for a number of maneuver situations, among which are the weave, left-turn, head-on, and cross-traffic categories given in Table 1. These situations were designated as maneuver situations because each involved one vehicle making a special movement or maneuver. The rear-end situation does not fall into the maneuver classification because it does not involve a vehicle changing its path.

When conflict data are recorded in the field, maneuver conflicts and rear-end conflicts are separated because the exposure for each classification is entirely different. For example, the total number of weaving maneuvers and the total number of weave conflicts that result from the weave maneuvers are recorded. In this case, the total number of weaving maneuvers is considered to be the weave exposure. For rear-end conflicts, the situation is different; the exposure associated with this situation is considered to be the total volume of traffic on the intersection approach that is being counted minus the total maneuver volume.

Coefficients were computed for approaches as well as intersections. In spite of the smaller sample sizes for intersections (each intersection consisted of at least two counted approaches), the general character of the correlations by intersection was very much the same as the correlations by approach; therefore, only the correlations by intersection are given in Table 1. Where the coefficients are significant, the hypothesis of no correlation is rejected and the hypothesis of correlation is accepted.

Overall, there appears to be a stronger case for rejection of the null hypothesis with the nonsignalized intersections than with the signalized intersections. A high percentage of the conflicts at signalized intersections are of the rear-end type. It is one of the most difficult types to observe, especially at signalized intersections where there is much braking that is unrelated to conflicts.

It is characteristic of the coefficients for the rear-end situation throughout the stratifications to be either not significant or somewhat close to the critical value.

Both the signalized and the nonsignalized intersections are further broken down into T and 4-legged right-angle types because these two types were most frequently counted. Data for the other types are not shown because of their small sample sizes.

Based on the data submitted by the three states, Table 2 gives the number of conflicts per 1,000 opportunities and the number of expected accidents per 100,000 conflicts for 4-legged right-angle intersections as well as all intersections combined. As might be expected, these numbers vary by type of situation. It must be remembered that the accident information used in this study represents the reported accidents compiled in the three states and therefore does not represent all the accidents that actually occurred.

It can be seen from Table 2 that the ratio of conflicts to conflict opportunities is higher for weaves than for other types of conflicts. Also, the ratio of accidents to conflicts is relatively low for weave and rear-end situations. These two situations are produced in traffic that is moving in the same direction. Same-direction-type accidents tend to be less severe and are therefore less likely to be reported. It may be reasonable to speculate that, because the same-direction-type conflicts usually result in less severe accidents, drivers may make less effort to avoid them.

Before and After Tests

As previously mentioned, conflicts counts were taken both before and after a spot-improvement type of change for 173 intersections (420 approaches). Although the improvements were not based directly on the conflicts counts, they were influenced by them to some extent as the states gained more confidence in the conflicts technique. Again, the conflicts analyses did generally support the analyses performed using accident experience.

Table 3 gives the improvements that produced a significant reduction in calculated danger indexes for T, 4-legged right-angle, and all intersections combined when t-values were computed from paired sets of before and after data.

The danger index for a particular intersection is found by dividing, for counted approaches only, the total number of types of conflicts by the total volume.

It can be seen from the table that, although danger indexes calculated for rear-end conflicts were not significantly reduced for the new signal improvement type, there were significant reductions in the maneuver indexes. Also, the overall effectiveness of signal upgrading and flashing signal installations appears doubtful from a conflicts standpoint.

By widening intersections (including adding turning lanes) the danger indexes for all cases were significantly reduced; however, widening together with signal improvements did not significantly reduce the danger indexes for the small number of T intersections where this type of improvement was made.

Volume Relationships

Because 4-legged nonsignalized intersections were by far the most predominant type of intersection counted, this type was chosen for the investigation of possible volume-traffic conflicts relationships. Figure 2 shows the number of conflicts per 1,000 opportunities by hourly approach volumes. It can be seen that 1-lane approaches experienced more conflicts per 1,000 opportunities than did the 2-lane or 3-or-more-lane approaches. The 3-or-more-lane approaches had the fewest number of conflicts per 1,000 opportunities possibly because drivers had the option of changing lanes to avoid left- or right-turning vehicles.

CONCLUSIONS

The following conclusions are drawn on the basis of the reported experience of the three states and the results of the statistical analysis:

1. The data compiled in this study tend to support the hypothesis that conflicts and accidents are associated;

Table 3. Before and after danger indexes using t-test.

Intersection	Improvement: New Signal	Upgrade Signal	Flashing Signal	Widening	Widening and Signal	Other
Maneuver conflicts danger index						
T	S	NS[a]	NS[a]	S	NS[a]	NS
4-legged right-angle	S	S	NS	S	S	S
All combined[b]	S	S	NS	S	S	NS
Rear-end conflicts danger index						
T	NS	NS	NS	S	NS	NS
4-legged right-angle	NS	NS	NS	S	S	NS
All combined	NS	NS	NS	S	S	NS
All conflicts danger index						
T	S	NS[a]	NS[a]	S	NS[a]	NS
4-legged right-angle	S	NS	NS	S	S	S
All combined	S	NS	NS	S	S	NS

Note: S = significant at 0.05 percent level; NS = not significant at 0.05 percent level.

[a]Sample size 5 or less. [b]Includes other intersection types such as skewed and multileg as well as T and 4-legged right-angle.

Figure 2. Conflicts for 106, 4-legged nonsignalized intersections.

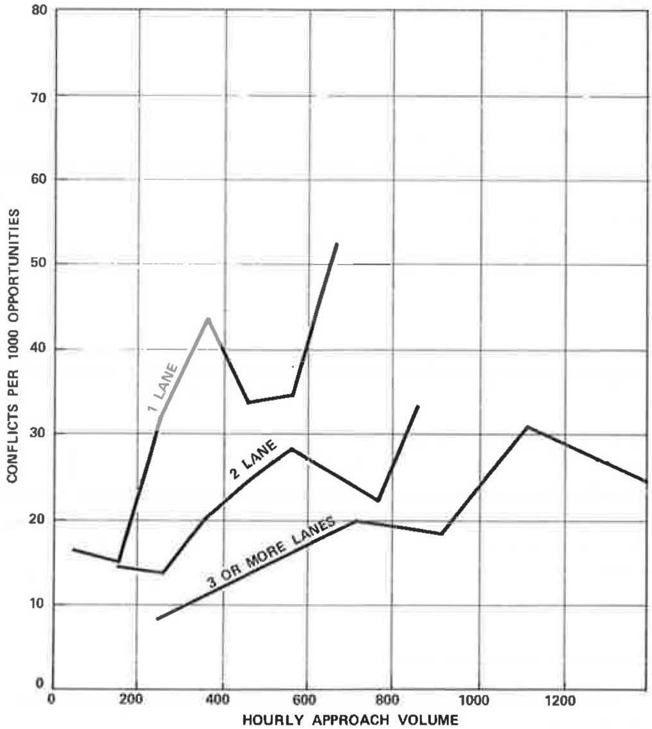
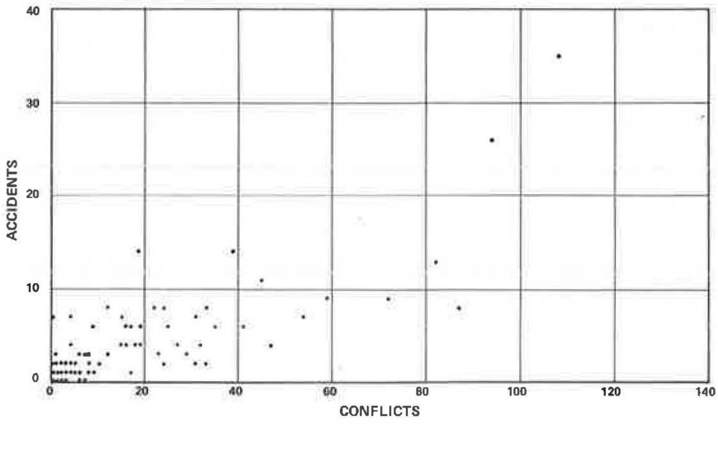


Figure 3. Maneuver conflicts versus maneuver accidents for 94 nonsignalized, 4-legged right-angle intersections.



2. On the basis of the experience of the three states, it appears that safety deficiencies at intersections can be pinpointed more quickly and reliably by using the traffic conflicts technique than by using conventional methods;

3. The traffic conflicts technique may be particularly valuable at low-volume rural intersections where the accident reporting level is low;

4. The traffic conflicts technique, because of its usefulness in pinpointing intersection problems precisely, should lead to lower cost remedial actions;

5. The traffic conflicts technique can be applied with minor modification to locations other than intersections;

6. The effect of intersection improvements may be demonstrated from conflicts counts taken shortly after completion of a spot-improvement type of change; and

7. The general surveillance information obtained during the conduct of conflicts counts may be valuable in improving the overall operations of intersections.

REFERENCES

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2. Snedecor, G. W. Statistical Methods. The Iowa State College Press, Ames, 1946.

APPENDIX

THE APPLICATION OF PEARSON'S CORRELATION COEFFICIENT

Where bivariate populations are concerned, the mutual relation between measures of each variable may be examined. One may make some evaluation of this relationship without thinking of one variate as dependent on the other. Each data point (X_1, Y_1; and X_2, Y_2) may be plotted to see whether there is a tendency for the points to plot in a band or whether they are scattered randomly in a shotgun pattern. When evaluation of this tendency (or correlation) is required, the correlation coefficient of Pearson, universally symbolized by r, is employed.

An r computed to equal +1 signifies perfect correlation, where the band of points plots in a straight line from lower left to upper right; an r equal to -1 signifies perfect negative or inverse association, where the band plots from upper left to lower right. Each area of investigation has its own range of values for the coefficients, and any judgment concerning a correlation should be made with reference to the size of similar correlations in the same area, with little reference to the theoretical limits of r (±1). In the case of accidents, conflicts, and other such phenomena, where many variables may affect the results, the coefficients are more apt to be small, that is, nearer in value to zero. Figure 3 shows a plot of accidents and all maneuver conflicts for non-signalized, 4-legged right-angle intersections. It can be seen that there is a tendency for the points to plot in a band. The correlation coefficient for this situation is 0.653. (Table 1).

The formula for the computation of the Pearson r is as follows:

$$r = \frac{N\Sigma XY - (\Sigma X)(\Sigma Y)}{\sqrt{[N\Sigma X^2 - (\Sigma X)^2][N\Sigma Y^2 - (\Sigma Y)^2]}}$$

An important consideration as to the meaning of a particular value for a correlation coefficient is the size of the sample from which the coefficient was computed because sample r's from a bivariate population are quite variable if the sample is small. For each sample size there is a probabilistically determined cutoff point for the value of the correlation coefficient below which it is not significant, that is, too small to be thought of as other than the result of a random scattering of data points. This critical point corresponds to a percentile (5 percent or 1 percent) of the probability distribution values of the correlation coefficient for a particular sample size. When the value of the correlation coefficient is found to be greater than this critical point, it is referred to as significant at a specific level of significance. Actually a test of significance is being performed when the value of a particular correlation coefficient is thus examined.

A few words might be helpful to explain the implication of the word test. In statistics, one tests a hypothesis. The hypothesis to be tested is proposed on reasonable grounds and defended on statistical grounds. The statement of any hypothesis, H_0, is necessarily tied to a statement of an alternate hypothesis, H_1, which would have to be rejected when the hypothesis H_0 is shown to be acceptable and which is acceptable when the hypothesis H_0 is rejected on the basis of the observed evidence.

Often a null or negative hypothesis is chosen as the appropriate hypothesis to be tested because of convenience, that is, according to the existence or absence of appropriate probability distribution tables. When a null hypothesis is rejected, this becomes the means of focusing attention on the acceptability of the hypothesis stated as its alternate. The correlation hypothesis is tested as a null hypothesis. The null hypothesis indicates that there is no correlation, no tendency for the plotted points of the bivariate population to form a band, and no difference between the plotted points and a random scattering of points. If we can reject this hypothesis at a small risk of being wrong, e.g., a 5 percent risk (5 percent level of significance), the alternate hypothesis is thereby shown to be acceptable. The alternate hypothesis would then indicate there is a correlation. Normally distributed bivariate populations of conflicts and accidents as well as random selection of the sample are assumed.

If the observed data substantiate a rejection of the null hypothesis, we can compute a correlation coefficient by using the preceding formula. If the evidence produces a coefficient larger than the critical value for that sample size, the hypothesis of no correlation is rejected, and the alternate hypothesis of correlation is accepted. The term significant is used to indicate an r-value large enough to reject the hypothesis, and a significant r implies a significant correlation.

When we accept the alternate hypothesis, we mean that we may behave as if it were true. We do not imply that it is actually true, only that, so far as available evidence is concerned, at a given level of significance, we have no reason for concluding that it is not true. This kind of statistical evidence does not constitute proof, and no claim of cause and effect may be had through such a statistical test. An important working hypothesis may have been successfully defended, however, until evidence to the contrary is found.

COMPATIBILITY FILE: AN INFORMATION RETRIEVAL SYSTEM

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The compatibility file serves as an interface between the transportation planning network files and other data files that are referenced by route number and cumulative mileage. The fundamental purpose of the file is to identify, by route number, each of the highway segments contained in the transportation study's link file (historical record) and also to identify by route number and cumulative mileage each node or highway intersection. The compatibility file would then be used to relate the computerized transportation study traffic networks to the physical inventory files maintained by the specialty divisions (traffic, highway needs, program planning, and accident) in a department of highway or transportation. The completed file not only provides a means of data exchange but, when used in conjunction with digitized networks and a data plotter, represents an excellent means of file editing and data presentation.

• HISTORICALLY, the physical inventory phase of a transportation study has presented the planner with a very costly task in terms of time and money. The data obtained during this important phase of a study are applied to the development of a historical record or link file that in turn serves as the base for computerized transportation networks.

Experience indicates that many portions of this inventory have been previously developed and are available but in a format that is not compatible with a transportation study's methods of storage and retrieval.

In the larger studies, such as those conducted by a department of highways or transportation, physical inventory data may be available from any one of several agencies that maintain separate inventories. Throughout the years, these agencies have been responsible for the development of portions of the existing transportation system and, therefore, data collection pertinent to the various specialty areas (such as traffic and needs) represents a normal function.

In recent years, the Transportation Planning Division in the Office of Planning, Bureau of Planning and Research of the Connecticut Department of Transportation, has been involved in several studies that require extensive use of data contained in the inventory files of other units. During this time, careful consideration was given to the possibility of developing a system that would allow the transportation planning data files to be compatible with the computerized inventory records of other units.

The Connecticut Department of Transportation has several such units or divisions that maintain inventory records of the existing highway system. Although the inventories have basic similarities, each is separately maintained and each serves separate functions. Typical data that may be obtained from the files are items such as number of lanes, pavement and shoulder width, geometric characteristics, and physical condition.

Several applications of the inventories were visualized. The data, however, were available only in printout form and could not be used in conjunction with the computerized networks except through manual conversion. To take full advantage of the data and,

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where possible, to eliminate the manual process, we initiated an effort toward the development of the compatibility file.

TRANSPORTATION PLANNING NETWORKS

The Transportation Planning Division of the Office of Planning maintains a set of historical records (link files) that contain data relating to approximately 60 percent of Connecticut's total roadway system. These physical inventory data are used almost exclusively as input to the computerized networks. The largest of these networks describes some 9,100 miles of roadway, which are represented by approximately 11,700 links or segments. (For the purpose of this paper, the term roadway refers to all types of roads regardless of function or class, whereas the term highway is used to describe primary and secondary systems.) In addition to a town-node description, the file contains a length, operating speed, lane, and roadway class record for each segment.

A smaller network contains similar data but represents fewer total miles. This system has some 5,800 links that represent 6,900 miles of roadway. Aside from the roadway mileage, the major difference between the two systems is the number and, consequently, the size of traffic zones. Each zone system represents the same land area, and in each of the files the total state highway system is represented. The variation in represented miles is attributable to nonstate local and collector roadways that are recorded in the system because of their significance in the traffic assignment process.

The roadway segments described in these files are coded for use with computers. As such, there is no identification (route names or numbers) associated with the roadway segments.

The roadway segments (links) are identified at each end by a three-letter town designation and a three-digit intersection (node) number. For example, assume that a given segment of highway between towns ABC and XYZ was to be identified by this file. The roadway would be listed by the town designator and the intersection number at each end of the segment. The intersections are numbered separately for each town, from 101 to 999. The link between towns ABC and XYZ might therefore be described as ABC101—XYZ101. In the event that the roadway segment existed entirely within town ABC, the appropriate description might be ABC101—ABC102. For the transportation planning networks, no other identification is required. The relation of a particular link segment to the existing system is established manually through a visual map comparison.

COMPATIBILITY FILE PRINCIPLE AND DATA FILES

The basic concept of the compatibility file is to identify, by route number, each of the highway segments and also to identify by route number and cumulative mileage each highway intersection. The compatibility file then could be used to relate the transportation planning networks to the physical inventory files maintained by the Records and Inventory, Traffic, and Program Planning Sections. Each of the aforementioned sections maintains an inventory of the highway system and catalogs data by route and cumulative mileage.

Unlike the files maintained by all other units, the network files contain the length of the roadway segments to the hundredth of a mile. Each of the other inventory files involved in this project contains the length of a section to the nearest thousandth of a mile. A brief description of these files follows.

The state road inventory master file (RM) is a tape file that is maintained by the Records and Inventory Section of the Office of Planning. It contains the largest number of records pertaining to the existing highway system. The roadway length recorded in this file is the result of a field inventory utilizing a "fifth wheel." The file lists a highway mileage, for each route, to the nearest thousandth of a mile and indicates a cumulative mileage point for each intersecting roadway. Various inventory data are recorded for each highway segment designated by the cumulative mileage points. The intersections are not identified by route number or word description in this file. They are identified in a companion file, the highway master log, that serves primarily as a mileage record and was utilized (in this study) only to identify the intersecting roadways at the cumulative mile points.

The highway needs inventory file is a tape record maintained by the Program and Scheduling Division of the Bureau of Administration. The file contains extensive inventory data that are used to develop a set of priorities for funding the needs of the highway system. The file also logs each route by cumulative mileage, but it does not list intersecting roads as thoroughly as does the RM file. The highway sections between cumulative mile points are identified by a route and a sequence number. The sequence numbers are consecutive, but space is allowed for breaking existing sections into smaller sequences. The physical inventory and related needs data are recorded for each sequence.

The accident master file is a tape record of reported accidents that is maintained by the Traffic Division of the Bureau of Highways. The file records each accident by type and utilizes route number and cumulative mileage to identify locations. In both the highway needs inventory and the accident master files, the cumulative mile values are obtained indirectly from the master log file. Other data files, such as the straight line diagram, that are inventoried and prepared by the Division of Traffic are being converted to computer format and also will be compatible with the transportation planning traffic networks.

FILE DEVELOPMENT

All of the aforementioned files have much in common; however, each has its own route number and cumulative mileage record. Although the transportation files do not contain a cumulative mile record, they do have a progressive summation of the link distances that creates the potential for establishing that item.

The first step in the development of the compatibility file was the identification of the network links, which represent the state-numbered road system. This manual process required a visual comparison of networks and the state road system.

The coding format (Fig. 1) was developed for use in the link-route identification phase. Application of this format allowed the related items to be collected and recorded during a single review of the system. The data contained on this coding form represent the base of the compatibility file and are discussed in the following.

The individual nodes that represent state routes are identified on the coding form along with the corresponding route number. This is accomplished on a route-by-route basis with a link distance and route sequence number recorded for each node. The link distance is taken from the link file. It represents the distance from the node being coded to the next node having the same route number and the next sequence number. The sequence number is used in forming node strings, which are a computer representation of the various routes. The node sequence numbers, starting at the zero cumulative mile point, are assigned in consecutive multiples of five (5, 10, 15, etc.) (In Connecticut, the zero or starting cumulative mile point for any highway is considered to be the most southerly or westerly end.) This procedure is followed to allow coding space for future expansion of the road system.

The overlap route numbers serve the function of identifying multinumbered or overlap routes, e.g., US-6/US-202. This eliminates double records for single roadway or link segments. The lowest numbered route is always referenced during data retrieval.

Word description is the only nonfunctional item contained in the file. Its purposes are to allow identification of link segments that cross town boundaries and to provide any desired comment relating to a particular segment. There is no restriction on the description format because it serves only informational purposes.

The intersecting route number and cumulative mile records represent the major control in the file. The intersecting route number is employed when a node is determined to represent a major route intersection. (The junction of two or more state-numbered routes is considered to be a major route intersection.) Where this occurs, a cumulative mile value, which is obtained from the master log file, is assigned to that node. It is important to note that this cumulative mile record is retained as a control value in the node string and is referred to elsewhere in the text as the checkpoint cumulative mileage.

Figure 2 shows the steps followed in arriving at the completed compatibility file. The more important aspects of this procedure are presented in the following text.

The node strings are formed by using a computer program that joins or strings the nodes into route sections. The program, as written, requires identical route numbers for nodes being added to the string and also checks the sequence number for compliance with the requirement that the sequence number of the node being processed be greater than the preceding sequence number.

The node string file was developed manually by transferring data from assorted maps and printouts. Because of this, the initial file contained many coding errors. The first consideration, therefore, in the editing process was to ensure that node strings do, in fact, represent valid links. This was accomplished by creating links from the node strings and matching these with a current link file that had been employed in the initial coding process.

The preliminary edited file is used to sum the link distances and to compare them with the checkpoint cumulative mileage. The summed link distance and the actual recorded distance, as computed from the control record cumulative mileage, must fall within a specified tolerance or the entire string is rejected. As each node string is processed, the preceding cumulative mile checkpoint becomes the zero or starting point for that string. During the editing of the string, only the difference between the newly designated starting point and next checkpoint and the summed link distances is compared. In this study's application, a 10 percent difference was allowed because the checkpoints rarely exceeded 3 miles and were generally less than 1 mile; therefore, any influence from compensating errors was minimal.

The rejected node string is recycled through a manual editing to determine the error and to correct discrepancies. It was found that rejections of node strings from the distance editing were due primarily to errors in coding and in transferring link distances. The distance editing also pointed out distance errors on links in the base network that had never been discovered because of the lack of an adequate editing program.

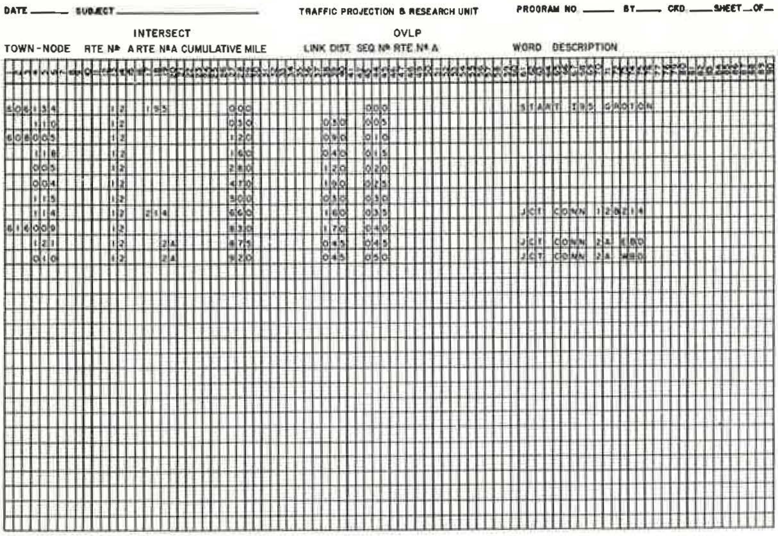
The final phase of the editing procedure was the compilation of the compatibility file. After it was established that the difference in cumulative mileage checkpoints and the sum of the link distances was no more than 10 percent, a cumulative mileage was computed for each node. This was accomplished through the use of a computer program that adjusts the link distances based on the error in the string segment, i.e., when a difference in distance existed between cumulative mile value and the summed link distances, and that difference was within the specified tolerance, the link distances were adjusted on a prorated basis to compensate for that difference. Then, by using the adjusted link distances, we computed cumulative mileages and assigned them to each node. The computed cumulative mileages then were added to the file, which completed the compatibility file.

IMPLEMENTATION

The actual data exchange is accomplished by using a generalized computer program written for the specific purpose of implementing the compatibility file concept. This program reads the compatibility file plus a data tape referenced by route and cumulative mileage and, according to user-specified parameters, performs certain data manipulation. The program uses the compatibility file as an interface between the networks and inventory files and produces a data tape referenced for use with the appropriate planning files.

The user may specify a maximum of seven data fields for manipulation. Allowable arithmetic operations include finding the minimum value, the minimum nonzero value, the maximum value and the mean, and the sum of the applicable input data. The user must supply the location of the data field on the input and the location for the output. He must also supply the tape blocking factor, the item size, and the field location of route and cumulative mileage for all files. The following example will best illustrate the computer program process. Let us assume that a user wants to update a highway

Figure 1. File coding format.



TOWN-NODE	RTE Nº	INTERSECT RTE NºA	CUMULATIVE MILE	LINK DIST.	SEQ Nº	WORD DESCRIPTION
506134	12	195	000		000	START I95 GROTON
110	12		030	030	005	
608005	12		120	090	010	
118	12		160	040	015	
005	12		280	120	020	
004	12		470	190	025	
115	12		500	030	030	
114	12	214	660	160	035	JCT CONN 12&214
616009	12		830	170	040	
121	12	2A	875	045	045	JCT CONN 2A EBD
010	12	2A	920	045	050	JCT CONN 2A WBD

Figure 2. Development of compatibility file.

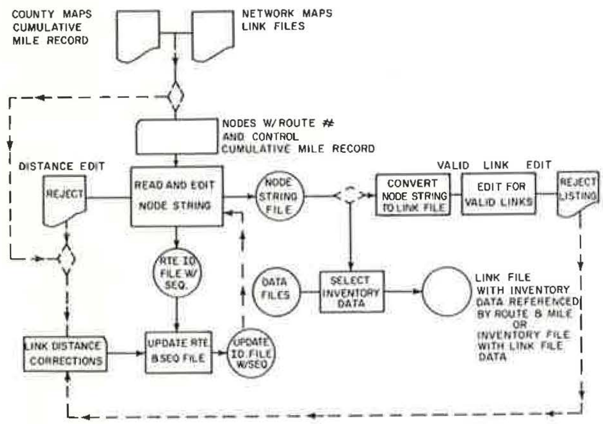
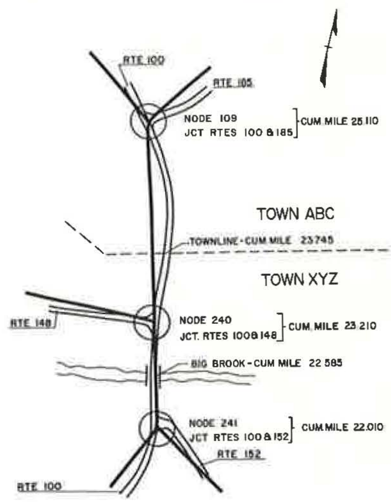


Figure 3. Data overlap section.



network link file with accurate street widths and that he wants the minimum street width for the length of each link recorded.

The computer program, after initial validation of the input parameters and tape files, reads the data tape and stores the applicable data (street width and route and cumulative mileage) for each inventoried segment contained in the data master file. The compatibility file is processed next. As each link is read, the data array is referenced by using route and cumulative mileage. If any part of an inventory segment is found to overlap the link in question, it is included in the development of data for that link. Any link for which there are inadequate data on the data master file is reported in an on-line printed output and included on the tape output with partial or no data.

The primary output is a data tape referenced by route, cumulative mileage, and link number that is used to update the file.

PROBLEM AREAS AND LIMITATIONS

The overlap of data segments is a problem that is difficult to eliminate from the data exchange system.

Because each unit that maintains an inventory file collects and records data primarily for use in its own specialty area, the beginning and ending points of data sections do not necessarily coincide. This is particularly true of transportation planning networks and needs files. Because the networks emphasize traffic projection and simulation techniques, they terminate links at nodes or intersections. The needs inventory file is concerned with the actual physical condition and consequent needs of the various facilities. In this file, therefore, data sections are referenced to needs projects and may begin or end at any point along the highway.

When a cumulative mile value is assigned to the nodes, the link sections may overlap the data sections represented by the other files and vice versa. Where these conditions occur, more than one data set may be available for a specified route segment (link).

This situation is less likely to exist in relating the networks to the RM file because the length of data sections contained in the RM is relatively short.

It is not a serious problem so long as the user is aware of the potential situation, and the degree to which any selected output is affected is dependent on the data involved.

A typical overlap situation is shown in Figure 3. If such a condition did exist, several sets of data might be available for that segment of Route 100 described as link ABC 109—XYZ 240. Whether a data retrieval problem would exist for this segment would depend on the data requested. If the user parameters specified a minimum or maximum value, then a single record, such as average daily traffic (ADT) or width, would be selected from the several records available.

A problem may occur when a listing of segments falling within a certain value range is requested. Under these conditions, and depending on the tolerances specified, a roadway segment may be identified as possessing the characteristics of an adjoining roadway section. In these instances, both sections would appear in the listing.

Assume that the user parameters specified a listing of all two-lane facilities carrying an ADT of 13,000 or greater. If the segment of Route 100 (Fig. 3) between cumulative miles 22.585 and 23.745, which overlaps link segments ABC 109—XYZ 240 and XYZ 241, has a recorded ADT equal to or greater than the specified minimum (13,000), then both link segments would appear on the listing. This, of course, assumes that the file being used records the sequence between the aforementioned cumulative mile points as one section. Some files, such as the RM file, might very well consider the same sequence as being several smaller sections, in which case the link ABC 109—XYZ 240 might represent two data sections. Those data sections would be from cumulative mile points 23.210 to 23.745 and 23.745 to 25.110. In the event that either of those sections recorded a value within the user's specified tolerance range, the link would be entered on the listing.

ADAPTATION TO OTHER SYSTEMS

Although the system described in this paper was designed for the inventory referencing scheme and transportation planning networks that are used in Connecticut, it is felt that the basic elements of the compatibility system can be used by every state and most transportation planning study groups. It is recognized by the authors that a majority of these planning study groups utilize the available Federal Highway Administration Urban Transportation Planning Program battery. The historical record format (Appendix), represents the basic inventory file of the computerized network used in this system. It contains items such as street width, parking, pavement type, and roadway classifications. These inventory items are allotted space for a rigorous analysis of network adequacy and, therefore, they should be included in the file. The problem, as was true in Connecticut, is that these data must be transferred to the historical record manually.

Before we discuss the system's applicability to other jurisdictions, let us review the system elements. First, a detailed transportation planning highway network must be available; this file should include route number (or similar identification of the link segments) and a record of the distance between nodes. If automatic data plotting is desired, geographic coordinates must also be available. Second, a compatibility file is prepared by using this coded network. The mandatory items contained in this file include route number, node number, and a cumulative mileage value for each node. The third and final consideration is the availability of data files referenced by route and cumulative mileage.

All states are required by law to maintain an inventory of the physical aspects of the state-maintained highway system. In addition, most states have accident information, needs, traffic control, and traffic count files. Usually, a computerized system does not exist to transfer the data from one file to another or from a file to the historical record. In most states, the files are referenced by route and cumulative mileage, and a few are referenced from mileage posts or some other variation of the two. Regardless of the referencing scheme, the compatibility system described in this paper can be applied.

The historical record format used in the FHWA Urban Transportation Planning Program battery readily lends itself to the development of a compatibility file. All of the previously mentioned items, node number, route number, and distance between nodes, are found in the historical record. The application of the techniques of progressive link distance summation combined with proper editing procedures will yield a cumulative mileage value for each node that can be further adjusted to the user's tolerances.

The development of the compatibility file provides the user with a new and powerful data-processing tool that, as the following application descriptions relate, allows previously inaccessible data to be transferred, sorted, plotted, and edited.

APPLICATION

The first data exchange accomplished through the use of a compatibility file was directed at the highway needs inventory files.

In this application, a program was written to utilize the geometric inventory data contained in the needs file as input to highway capacity programs. The resulting capacity values, based on procedures given in the 1965 Highway Capacity Manual were computed for each highway section listed in the needs file. Through use of the compatibility file, the highway sections and corresponding link segments were identified and the capacity values were transferred to the network files.

The capacity values also were used to add volume-capacity ratios to the needs file, which provided some measure of traffic demand. This was accomplished by utilizing the ADT value recorded in the needs file to obtain a service volume that was then compared with the computed capacity value to obtain the ratios. It is also possible to obtain a service volume by using projected traffic assignments to measure future traffic demand on existing systems.

Perhaps the single application that has proved most beneficial to the transportation study is the ability to easily relate links to routes and to add current ADT values to the link files.

The Connecticut Department of Transportation produces an annual ADT log. As this tape file (ADT log master file) is updated, it is a simple mechanical operation to transfer the current data to the link files. A recent application of this type has resulted in a 20 percent increase in the total number of ADT links in the network. This is a tremendous asset to the network calibration process because it represents a substantial increase in the number of links with ADTs and also updates existing records with recent counts.

Although initial implementation of the compatibility file was intended as a transportation planning tool, its benefits are easily extended to each of the agencies whose files are involved. Data editing and manipulations may be performed by working from the network and compatibility files. Many programs that are developed primarily for a transportation study may be used to edit or update files of other agencies. This type of application has recently been used to transfer current ADTs from the ADT master log to the RM file. Prior to this application, such a process was a time-consuming and lengthy manual project that created a difference of up to 2 years in the recorded ADT values.

One outstanding example of multiple-agency use in applying the file is in file editing. The transportation planning networks have been digitized for use with a data plotter, and, therefore, all data compatible with the network files may be plotted. Experience has shown that plotting data in this manner is an excellent form of file editing. It allows a rapid visual examination and very quickly presents a broad overall view of the data.

By using available computer programs in conjunction with the plotter, various combinations of data may be sorted, grouped, and presented for visual review. Data may be selected from individual files and combined and plotted to present relationships such as accident areas versus needs projects or accidents versus safety projects. Other items such as needs, railroad crossings, bridge repair projects, capacities, and ADTs may be plotted by individual route or area as well as on a statewide basis.

Recent applications have included the plotting of a statewide traffic flow map by using current ADTs. Plots of this type are also serving as drafting aids that provide the cartographer with scale values for use in producing published traffic flow maps.

SUMMARY

A procedure has been presented that provides the transportation planner with the capability to easily access typical inventory files such as highway needs, physical inventory, and accident and traffic files.

By using as an interface a node-to-node description of the state highway system, a planner can transfer data between transportation planning network files and inventory files. Connecticut Department of Transportation planners can perform the following by using this system:

1. Analyze the effects of the physical characteristics of the existing highway network in conjunction with developing a future transportation system;
2. Increase the data base of the transportation planning study into areas such as accident analysis, highway needs, and traffic inventory that heretofore were inaccessible, except on a small scale, because of the difficulty in manually transferring the data;
3. Graphically present any aspect of the transportation network and inventory files by using a data plotter; and
4. Aid other departmental units by making available to those units that maintain inventory files the various sorting, editing, and data plotter computer programs that have been written for transportation studies.

The system has been implemented to transfer current ADTs to the transportation planning network files. Also, capacities that were calculated by using procedures outlined elsewhere (1) and physical inventory data have been transferred to all state road links in the network files. Volume-to-capacity ratios have been calculated and transferred to the highway needs inventory file. All of the preceding items plus many aspects of the highway needs file have been plotted. Pictorial representation of many of these data is available for the first time.

It is felt that this procedure represents a real asset to the transportation planner by adding further to his ability to provide an effective transportation plan.

SPEED PROFILES APPROACHING A TRAFFIC SIGNAL

Robert L. Bleyl*, The University of New Mexico

The objective of this study was to examine and compare the speed profiles of traffic approaching a traffic signal under six different signal displays. Detector loops were installed along one approach to a rural traffic signal installation. Detector actuations, signal indications, and timing information were recorded at a remote observation point by using a 20-pen operation recorder. Observations were made of lone vehicles approaching the traffic signal location. The speed profiles observed under each signal display were summarized and compared with the speed profiles under the other signal displays. Drivers at the study location entered the intersection more cautiously with a green traffic signal indication or a flashing yellow indication than they did with no signal installed. They did not speed up when signal control was changed from regular stop-and-go operation to flashing operation. Approaching a red signal indication, drivers did not begin to slow down until they were approximately 500 ft from the signal. Under all other signal displays, drivers generally maintained their speed as they approached the signal location and entered the intersection.

•FOR efficient design and operation of safe traffic signal installations we must understand the responses of drivers to various traffic signal indications. Although numerous studies dealing with certain traffic response characteristics, such as starting delays and headways, have been conducted, few studies have attempted to examine the speed profiles of vehicles approaching a traffic signal under various signal displays.

Important questions related to the speeds and speed profiles of vehicles approaching traffic signals include the following: How do drivers respond to different signal displays? When do they speed up? When do they slow down? When do they maintain their speed? How far in advance of the signal do drivers respond to the signal? Do drivers speed up when signal control is changed from stop-and-go operation to flashing operation? Does the installation and operation of a flashing beacon cause drivers to approach the location more cautiously; that is, do drivers slow down when approaching a flashing beacon?

Past studies of these questions are considered to be inconclusive by the author for the following reasons: Some of the studies failed to provide adequate control over the variations in speed with time; some studies employed coarse data collection methods and permitted the observer to unconsciously influence the recorded speed measurements; other studies compared speeds at only one or two specific points on the approach rather than determining the speed profile over the length of the approach.

FIELD STUDY

The site selected for this research was one approach to a rural, right-angle, four-way intersection in central Pennsylvania. The study approach carried one lane of traffic in each direction. The speed limit along the test approach was 55 mph. The average daily traffic volume at the test site was approximately 1,200 vehicles. Visibility of the traffic signal installed at the intersection was restricted to 1,200 ft by a change in grade.

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*When the research in this paper was performed, Mr. Bleyl was associated with the Bureau of Highway Traffic, Pennsylvania State University.

Sponsored by Committee on Traffic Control Devices.

The land use on all four corners at the intersection was agricultural; this land use provided good visibility of other traffic near the intersection. The signal installation consisted of dual 8-in. signal indications; the installation conformed to state and national standards.

A series of 14 detector loops were installed in grooves cut in the pavement along the approach. The first loop was located 1,800 ft in advance of the intersection. Loops were spaced at 150-ft intervals with the last loop located 150 ft beyond the intersection. None of the loops was noticeable by approaching drivers. Detector actuations were transmitted by wire to a remote point of observation. Figures 1 and 2 show the plan and profile of the test approach.

A 20-pen operation recorder was employed to make a master record of signal indications, vehicle detections, timing pulses, and identification codes. Figure 3 shows the chart record produced during a demonstration run. The identification of each chart marking has been added to the illustrated record. The accuracy of the chart record and supplementary chart processing equipment was evaluated; the measured trap times were found to be accurate within $\frac{1}{20}$ sec 95 percent of the time. Therefore, by using this method of speed determination, the speed of a vehicle traveling at 50 mph could be determined to an accuracy of ± 0.8 mph 95 percent of the time. Also, speeds could be determined without being influenced by human limitations. This method also permitted the entire speed profile of any given vehicle traveling along the approach to be determined.

Six specific signal displays were selected for this study. Four of these displays are shown in Figure 4 and are described as follows:

1. A green signal indication from the moment the signal first became visible until the vehicle reached a point approximately 900 ft in advance of the signal, at which point a red signal indication was given (preceded by a yellow clearance period), referred to as the green-red display;
2. A red signal indication until the signal was reached, referred to as the red display;
3. A red signal indication from the moment the signal first became visible until the vehicle reached a point approximately 900 ft in advance of the signal, at which point a green signal indication was given, called the red-green display; and
4. A green signal indication during the entire approach, called the green display.

The timing of the signal controller was synchronized with the approach of each vehicle selected for observation. This synchronization was accomplished by using the offset circuit to brake the cycle unit drum at the advance setting from the desired arrival point. As the observed vehicle passed loop 2, the brake circuit on the cycle unit was automatically released, thereby establishing the desired relationship between the signal timing and the approaching vehicle.

The fifth signal display consisted of flashing operation with traffic on the test approach receiving a flashing yellow signal indication, referred to as flashing yellow. The sixth display consisted of no signals at all. For the no signal display, the signal heads, span wire, and cables were removed from the site. Traffic control at the intersection reverted to two-way stop control with the test approach located on the through street. Observations with this display were made 1 month after the signals were removed to allow drivers time to adjust to the new intersection control.

For all six signal displays, observations were made on lone vehicles that did not turn at the intersection. Vehicles were selected randomly from the traffic stream. A vehicle was considered lone if it was separated from every other vehicle traveling in its direction by at least 600 ft (approximately a 10-sec headway preceding and following the observed vehicle). Observations for all six signal displays were made on weekdays during the daytime when the weather was clear or cloudy and the pavement was dry.

To control the variations in traffic speeds with time and to provide an equivalent basis for comparing the speed profiles observed with each of the six signal displays, we selected for inclusion in the study only those vehicles that had an initial speed of from 40 to 45 mph, as measured between loops 2 and 3 (trap 2). This qualifying speed was measured before the drivers could see the traffic signal indication at the intersection ahead.

Figure 1. Test site.

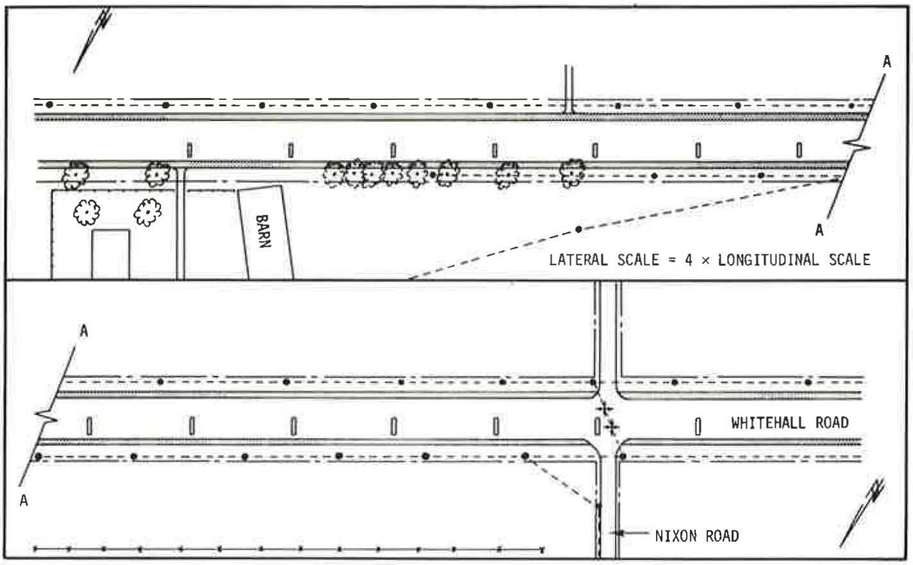


Figure 2. Test approach.

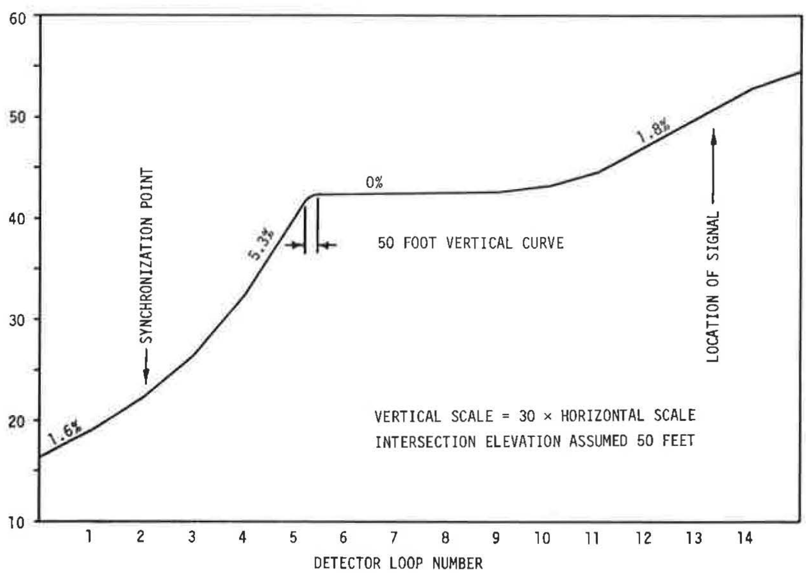
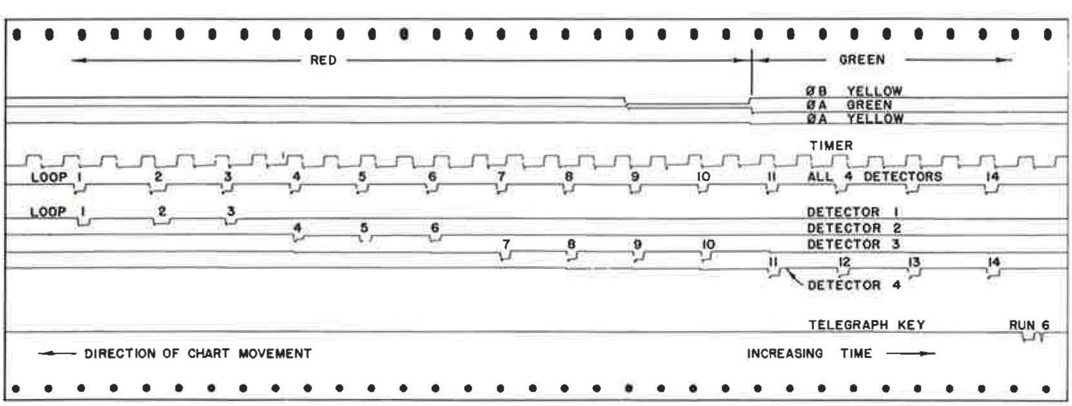


Figure 3. Operation recorder record of a demonstration run.



ANALYSIS

The markings recorded on the operation recorder charts for each observation were converted to coordinates and punched into data-processing cards. A Benson-Lehner model Oscar F film-chart reader and digital converter were used to automatically process the operation recorder records. In using this equipment, one merely positions a movable cross hair directly over the desired point on the chart and presses a button. The equipment then automatically determines the numerical coordinate of that point and punches the coordinate into a data-processing card.

An IBM 360 computer was used to edit the cards. The editing process included a check for errors in keypunching and a check for irregularities and inconsistencies in data speed and travel characteristics.

The time required to traverse the 150-ft distance between successive loops was converted to a speed that was assumed to be the actual spot speed of the vehicle at a point midway between the two loops. A single observation consisted of 13 successive trap speeds for one vehicle.

The individual speed profiles within each signal display condition were observed to be similar to each other. Accordingly, an average speed profile was computed for each of the six signal displays. Each point of the average speed profile was determined by averaging the individual trap speeds for each trap.

FINDINGS

Figure 5 shows the six average speed profiles, and Table 1 gives a summary of the average speeds and standard deviations. The following are specific findings based on Figure 5 and Table 1:

1. All six average speed profiles prior to trap 5 were essentially the same. The maximum difference between any two speed profiles in this area was 1.2 mph, which is not statistically significant. This finding indicates that the initial speeds were the same for all six sets of data.
2. All six average speed profiles exhibited a statistically significant reduction in speed at trap 5. This decrease was expected, inasmuch as trap 5 corresponds to the location of an abrupt vertical curve (Fig. 2). Prior to reaching trap 5, approaching drivers could not see the signal indications.
3. For the four signal displays that did not require approaching traffic to stop, red-green, green, flashing yellow, and no signal, the average speeds immediately prior to entering the intersection were slightly lower than the immediately preceding speeds. For the green display, the magnitude of this speed reduction, 2.4 mph over a 450-ft distance, was statistically significant. For the other three average speed profiles, the magnitude of the speed reduction was not statistically significant.
4. For the preceding four signal displays, the no signal display had a significantly higher intersection speed than the other three displays. This finding indicates that the operation of a flashing beacon or a traffic signal at this location caused drivers to approach the intersection more cautiously; that is, the drivers did slow down. The reduction in speed was between 3 and 4 mph.
5. A comparison of the average speed profile for the green display against the average speed profile for the flashing yellow display indicated that the two profiles were at no point significantly different. This finding indicates that drivers did not speed up when signal control was changed from stop-and-go operation to flashing operation.
6. Traffic approaching the red display at a speed of about 40 mph began slowing down approximately 150 ft after the red signal indication first became visible. The magnitude of this speed reduction was slight (less than 2 mph over a 500-ft distance) until a point approximately 500 ft in advance of the signal was reached, at which point the rate of deceleration increased significantly.

DISCUSSION OF FINDINGS

It is generally believed that the installation and operation of a traffic signal or flashing beacon will cause drivers to slow down and approach a location more cautiously than

Figure 4. Four signal displays studied.

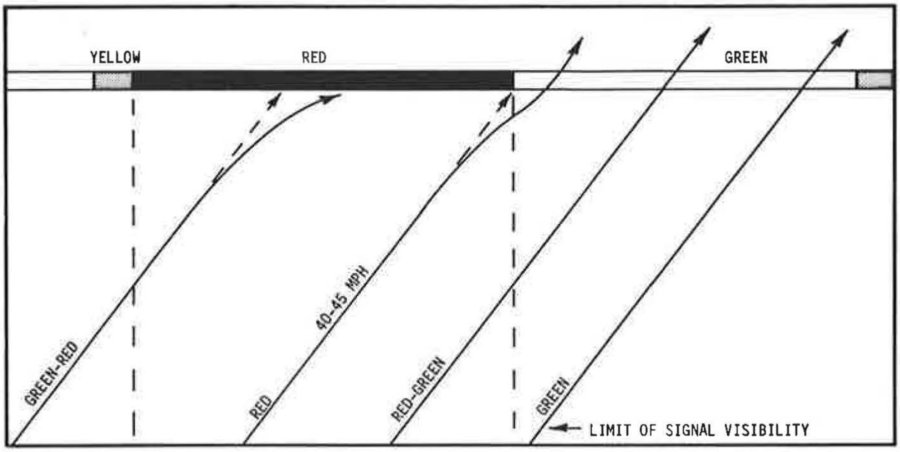
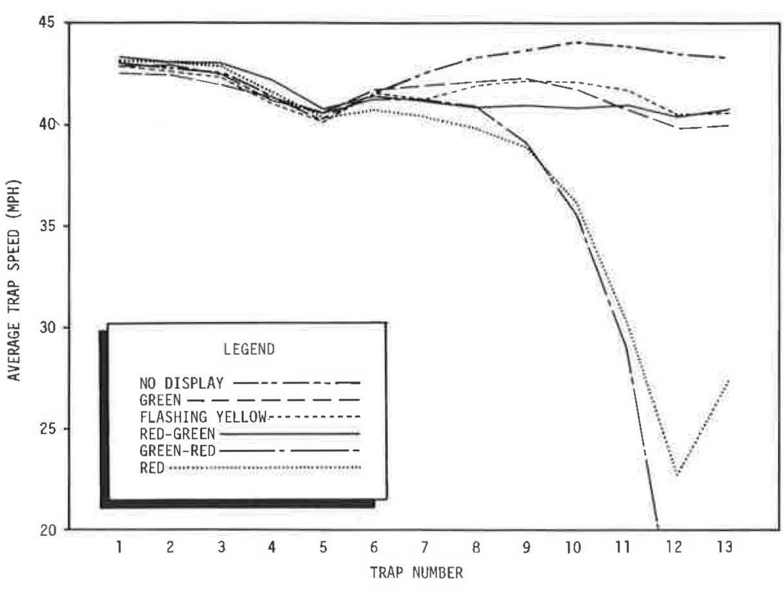


Table 1. Average speed profile characteristics.

Signal Display	Number Observed	Trap Number 1	2	3	4	5	6	7	8	9	10	11	12	13
Average speed (mph)														
Green-red	22	42.8	42.9	42.4	41.1	40.6	41.3	41.2	40.9	39.1	35.6	29.1	—	—
Red	17	43.2	43.0	42.9	41.6	40.3	40.7	40.4	39.8	38.9	36.1	30.3	22.7	27.3
Red-green	13	43.0	43.0	43.0	42.2	40.7	41.4	41.1	40.8	40.9	40.8	40.9	40.3	40.7
Green	18	42.5	42.4	41.9	41.3	40.6	41.7	41.9	42.1	42.2	41.7	40.7	39.8	40.0
Flashing yellow	17	42.8	42.6	42.3	41.0	40.1	41.5	41.2	41.9	42.1	42.1	41.7	40.5	40.5
No display	28	42.7	42.6	42.5	41.2	40.3	41.5	42.6	43.4	43.7	44.1	43.9	43.5	43.3
Standard deviation														
Green-red	—	1.3	1.3	1.4	1.4	1.8	2.1	2.1	3.0	4.1	5.4	6.8	—	—
Red	—	1.3	1.4	1.8	1.6	1.7	2.5	3.3	4.2	4.8	4.6	3.4	4.7	4.9
Red-green	—	1.7	1.8	2.0	2.4	2.6	2.7	3.6	4.2	4.6	3.8	3.6	4.3	5.3
Green	—	1.6	1.5	1.4	2.3	2.9	3.0	3.2	3.3	3.3	3.6	3.8	3.7	3.5
Flashing yellow	—	1.6	1.8	2.1	2.1	2.5	2.4	2.6	3.2	3.2	3.0	3.6	5.1	5.5
No display	—	1.4	1.5	1.6	1.7	2.1	2.4	2.4	2.7	3.2	3.7	3.4	3.5	3.6

Figure 5. Average speed profiles of traffic having initial speeds between 40 and 45 mph.



they would without such devices. Although the findings of this study support that conclusion with statistically significant data, the magnitude of the speed reduction, from 3 to 4 mph, may be considered to be of little practical significance. The significance in this finding is not in the magnitude of the speed reduction but in the increased degree of driver alertness and caution created by the device, as suggested by the fact that there was a decrease in speed.

It may seem unusual that with the four nonstop signal displays, green, red-green, flashing yellow, and no display, drivers generally did not slow down immediately in advance of the signalized intersection. The maximum speed reduction observed in all four average speed profiles was 2.4 mph over a 450-ft distance. This speed reduction is equivalent to a uniform deceleration rate of approximately $\frac{1}{100}$ ft/sec^2. Because the ground in the vicinity of the intersection was essentially flat, drivers approaching the signal could see across the corners of the intersection and observe that there was no conflicting traffic on the cross street, as was almost universally the case. Under these conditions, drivers apparently saw no need for slowing down in advance of the intersection. It is believed that a substantial speed reduction might have occurred had there been more cross traffic.

In approaching the red signal display, drivers could first see the red signal from a distance of approximately 1,200 ft. That they saw the signal at this distance is indicated by the fact that the red display speed profile significantly deviates from the no display speed profile soon after this point is reached. However, drivers did not begin to slow down for the red signal until they reached a point much closer to the intersection. They may have expected the signal indication to change to green before they reached the signal. In any case, they tended to hold their speed until reaching a point at which they were forced to decelerate. The distance at which forced deceleration begins varies with speed; in this case, the speed was approximately 40 mph, and forced deceleration began at approximately 500 ft in advance of the intersection. The resulting deceleration was equivalent to a uniform rate of approximately 4 ft/sec^2. The same deceleration rate was also observed for the green-red display.

This finding suggests that drivers considered a deceleration rate of approximately 4 ft/sec^2 to be the most comfortable rate. If they had preferred a more gradual rate, they would have begun forced deceleration earlier. If they had preferred a more abrupt rate, they would have waited longer before decelerating.

It has been claimed that traffic signals should not be placed on flashing operation when traffic volumes get low because flashing operation encourages drivers to drive faster (1). The average speed of all traffic approaching a flashing yellow signal would obviously be higher than the average speed of all traffic approaching the same signal with the stop-and-go operation of regular signal control; however, do drivers really approach a flashing yellow indication faster than they approach a green indication?

The findings of this study indicate that there was no significant difference in the speed profiles of traffic approaching a flashing yellow signal indication as compared to a green signal indication at any point along the 1,950 ft of roadway studied. It is not known whether these findings would apply to other locations. The major point of this finding is that serious consideration might advantageously be given to a careful study of the relative advantages of both regular and flashing signal control.

CONCLUSIONS

As a result of this study, the following conclusions were reached for the study location:

1. The installation and operation of a flashing beacon caused drivers to approach the intersection more cautiously than they did without the flashing beacon.
2. The installation and operation of a traffic signal caused drivers to approach the intersection more cautiously than they did without the signal.
3. Drivers did not approach a flashing yellow signal any differently than they approached a green signal. Drivers did not speed up when signal control was changed from stop-and-go operation to flashing operation.

4. Drivers approaching a red signal indication at a speed of approximately 40 mph did not substantially begin to slow down until they were 500 ft from the signal. The resulting deceleration was equivalent to a uniform rate of approximately 4 ft/sec^2.

5. When approaching this intersection with a green, flashing yellow, or no signal display, drivers generally did not slow down but tended to maintain their speed as they entered the intersection.

ACKNOWLEDGMENTS

The author gratefully acknowledges a generous grant of traffic signal equipment from the Eagle Signal Company, which helped make this study possible. The use of other equipment belonging to the Automatic Signal Company and to the City of Philadelphia is also appreciated.

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NEAR-MISS DETERMINATION THROUGH USE OF A SCALE OF DANGER

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Near-miss traffic events have been considered but not adopted as a traffic safety tool because of the high degree of subjectivity involved with their identification. A scale of danger may be applied to a traffic event to facilitate objective measurement and subsequent detection of near-miss situations. The unit proposed here for this danger scale is the time measured until collision between two vehicles involved in the unsafe event. This measure, computed from films taken with the Traffic Sensing and Surveillance System of the Federal Highway Administration at an urban intersection, is an adequate unit to rate the danger of almost any traffic event. It may be used to standardize human observer judgment of dangerous maneuvers and, therefore, make near-miss monitoring a viable alternative to traffic safety determination.

•NEAR-MISS traffic events have been considered for use as predictors of accident rate characteristics at roadway locations. The near miss, loosely defined, is a traffic event that produces more than an ordinary amount of danger to the drivers and passengers involved. Near misses would appear to be closely related to the accident pattern witnessed at a location and, therefore, could become an attractive alternative measure to accident-based safety determination.

Although the use of near misses seems appealing for safety monitoring, near misses have never been seriously considered as accident predictors because their detection and classification involves a great deal of judgment on the part of the observer. An event that looks dangerous to an observer who is a conservative or inexperienced driver may appear commonplace to an observer who drives very aggressively. Consequently, counts of near-miss events could vary substantially because of the differences in the personalities and driving experiences of the observers. Because judgment of near misses requires a subjective judgment of danger, near-miss measurement has been rendered virtually useless as a traffic engineering tool.

Because almost every traffic event has a certain level of danger associated with it, there is a need for establishing some threshold level for use in distinguishing near misses from less dangerous events. The fixing of this danger level requires a scale of danger that is physically measurable for a traffic event. If the danger level for each event can be rated on a common scale, the events can be ranked in order of danger.

The objective of this research was to define in physical terms a measurement of the danger involved in a two-vehicle interaction. This measurement may be used to establish a limit that would distinguish a near miss from other dangerous traffic events. The study was intended to provide a measurable frame of reference for degree of danger that could be used for standardizing human judgment of near-accident traffic events.

The dangerous events considered in this study were confined to two-vehicle interactions at an urban intersection during off-peak volume periods. The concept developed, however, may be easily extended to single-vehicle-fixed-object interactions and to other highway environments.

Sponsored by Committee on Effectiveness of Operational Measures.

TRAFFIC SENSING AND SURVEILLANCE SYSTEM

The measurement of motion and positional parameters involved in a two-vehicle interaction must be done by analyzing motion picture film. Film provides an accurate time base and instantaneous position points from which vehicle velocities and accelerations may be calculated.

The Traffic Sensing and Surveillance System (TSSS), developed by the Federal Highway Administration (FHWA), was used to record the film required for this study. The system has a unique instant-replay capability that makes it ideal for filming selected traffic events that occur with no discernible regularity.

The TSS System used two television cameras positioned on opposite corners of the intersection of 14th and F Streets in Washington, D. C. The cameras monitored the intersection action and recorded it continuously onto a magnetic video disc. The disc had a 20-sec storage capacity; when the 20 sec was reached, the recording arm recycled and recorded the next 20 sec onto the disc and erased the previous recording as it proceeded. Upon command, however, the video disc could be played back and the 20 sec of action could be transferred onto 16-mm motion picture film for a permanent recording of the selected event. The system gave the observer the capability to record on film the intersection action that occurred in the 20 sec prior to the activation instant.

The 16-mm film presented a sufficiently clear and precise picture for vehicle identification and tracking. The field of vision included the intersection itself and approximately 200 ft of each intersection leg, which permitted the observation and recording of near misses occurring both at the junction and in the approaches.

FILM SITE

The location chosen by the FHWA for installation of their TSS System was the intersection of 14th and F Streets located in the central business district of Washington, D. C. Its selection was based on site requirements for the TSSS hardware, but it proved to be an excellent site for the research reported here. The intersection is typical of intersections of surface streets in any large metropolitan area. It is signalized and handles high volumes of both pedestrian and vehicular traffic. Figure 1 shows the intersection's geometric configuration; about 35 to 40 accidents per year are reported at this location.

FILMING PROCEDURE

Sequences of dangerous traffic events were selected for filming by observing real-time television monitors in the TSSS control room during the data collection days of April 8 and 9, 1970. Each television camera relayed a picture of the intersection back to the control room for display on a monitor. Each camera view covered approximately one-half of the intersection with a certain amount of overlap at the center. The two television monitors were arranged to simulate an overall aerial view of the entire location. It was a simple matter to track a vehicle from one camera view to another through this arrangement. After a short adjustment period, the observer had a very good orientation for viewing the entire intersection simultaneously, as if suspended high above the center of it.

Additional information on traffic conditions was provided through an audio connection with the intersection. A ground-level microphone transmitted sounds from the street to the control room. This provided the observer an audio cue to the real-time events occurring at the intersection; the sounds of horns blowing and tires squealing were clearly distinguishable from the regular traffic noise. These two specific noises seemed to be indicative of a dangerous situation and were very useful taking films of dangerous traffic events.

The general plan of film collection was to observe the entire intersection through the monitors to detect near misses. Because only two intersection approaches were in motion at once, it was fairly simple to monitor all vehicular action simultaneously. When two vehicles appeared to be in a dangerous situation, the magnetic disc was operated in the playback mode. The 20 sec of action stored there were projected on a third monitor for filming by the motion picture cameras. The criterion used for activation was completely subjective and followed the general definition that a near miss is an event that produces more than an average amount of danger.

The time of observation for near misses was restricted to periods from 9:30 a.m. to 1:00 p.m. and 2:00 p.m. to 4:00 p.m. on both days of data collection because of TSSS equipment considerations. The afternoon period of the second day was not utilized because the amount of film provided by the FHWA had been exhausted, and it was felt that a sufficient number of sequences had been recorded. A total of 90, 20-sec segments was recorded for study.

DATA REDUCTION

The filmed events were reduced to tabulations of motion parameters to analyze the sequences quantitatively. The data reduction technique employed a Benson Lehner Oscar Model F film reading device to reduce the points of interest in each frame to coordinate points punched onto a computer card. These coordinates were transformed from film-reader Cartesian coordinate points to ground-level coordinates by using a series of regression equations. The resulting points were used, with the frame speed as a time base, for motion calculations for the two vehicles involved. These calculations were performed by a computer program modified from the original analysis program for the TSS System written at Cornell Aeronautical Laboratories.

The final result of the data reduction technique was the computer printout of the motion and spacing parameters for each vehicle at each frame point in the analyzed sequence. These parameters were presented in both a tabular listing and a graphic representation to ensure that an accurate and recognizable representation of the interaction dynamics was given. Velocity, acceleration, coordinate positions, spacing, and time to collision were presented for analysis.

NEAR-MISS MEASUREMENT

The initial approach to the problem of near-miss definition centered on the derivation of separate definitions for each type of encounter. The near misses were grouped by type categories such as the type of accident that would have occurred if a corrective maneuver had not been made and then different threshold levels of velocities and spacings were assigned as the near-miss definition. It was anticipated that this threshold level would be based on calculated theoretical values.

For example, in the case of a vehicle quickly changing lanes into the path of a second vehicle, a calculation to determine a near miss could be based on the stopping distances of the two cars involved. If, for some reason, the first car had to make a panic stop sometime after pulling into the path of the second car, the distance required to bring the first vehicle to halt could be calculated by using an assumed friction factor between the roadway and the tires. The second car would also be required to stop suddenly which would be a function of that vehicle's speed and coefficient of friction plus the distance traveled during the reaction time of the driver. Given the spacing of the vehicles, it could be calculated whether a rear-end collision actually would have occurred if the first car had suddenly stopped.

These situations where accidents would have taken place, given certain conditions, would seem to describe near-miss situations for the specific given condition. Unfortunately, there are too many conditions that control the actions and reactions of drivers to make all required classifications possible. If, for instance, in the preceding example the trailing car had swerved out of the collision path, the near-miss criterion measure would not hold. The numerous possibilities of accident instigation and avoidance that could occur in a two-vehicle interaction at an urban intersection made this type of approach impossible. Each near miss seemed to have its own set of conditions, which made the calculations of each event unique.

TIME TO COLLISION

Effort was directed toward development of an objective measure that would apply to all types of near-miss situations. This resulted in the development of the parameter recommended here, the time-measured-to-collision (TMTC) measure of danger. It was observed on the films that the traffic flow at the intersection seemed to be smooth until a perturbation was introduced. When a driver would make an error and cause a dangerous condition, the affected drivers would compensate to avoid collision, and the flow would return to a stable condition. Thus, the danger seemed to increase and then subside.

The TMTC measure was thought to reflect this subjective feel for the near-miss phenomenon in which danger peaks and then subsides. Very simply, the measure is the time required for two vehicles to collide if they continue at their present speeds and on the same path. It is a measure continuous with time; that is, the calculation may be performed at any instant within the sequence time frame.

Automobiles are frequently driven on paths of collision with other vehicles, pedestrians, or fixed objects. The reason collisions do not often occur is that drivers are constantly making the necessary speed and heading changes to avoid crashing. Therefore, almost all driving, except in the middle of a perfectly flat, deserted plane, involves a certain element of danger. Traffic events where corrections to evade collisions must be made in a very short time are what we intuitively call a near miss. It follows that the real degree of danger to drivers may be measured by calculating the time available to them to make the necessary correction to evade an accident. A near miss is nothing more than a traffic event with a low TMTC value associated with it.

A curve of the TMTC measure during a near-miss event plotted against time should be concaved upward, reflecting the increasing and then subsiding danger as a near miss passes. The theoretical shape of a simple near-miss curve of TMTC values versus time is shown in Figure 2.

A way of visualizing a real-life event that results in this curve is to consider the special case of a car-following situation with unequal speeds. If the lead vehicle is traveling at a slower pace than the following vehicle, there is a definite time to collision. As the vehicles draw nearer, the TMTC value will drop. The decrease will be linear as long as constant speeds of both vehicles are maintained. When the driver of the following vehicle senses the impending collision, he would slow his car and thus decrease the TMTC value. The following driver would continue to slow until the speed of his vehicle coincided with that of the lead car and a collision would not occur. If a collision cannot occur, the TMTC value is infinity.

The calculation of the TMTC value was added to the computer program, which produced the velocities and spacing determinations for each near-miss sequence. The method of the calculation was adopted from a navigation computation by which ships determine how close they will pass. One vehicle was considered stationary, and the second was considered to move with respect to the first. A collision was imminent when the relative velocity vector extending out from the moving vehicle passed through the stationary vehicle.

THEORETICAL BOUNDS OF TMTC

The maximum TMTC value of any two vehicles is infinity. Because drivers do not ordinarily drive on a collision course with other vehicles, it was expected that no-collision values were to be found in the output of the near-miss analysis program. The normal and safest value of time to collision for a given traffic event would be infinity.

The minimum value of a TMTC measure for a near miss would be the driver's perception plus reaction time. This time is the time required for the driver to perceive the imminent danger of collision and to decide a course of action and implement it plus the time needed for the vehicle to respond to the driver's command in order to avoid collision. If the TMTC value drops below this level, a crash will occur because there is not enough time for avoidance.

A numerical value of the minimum TMTC measure would be approximately ½ sec. This approximate value is estimated by using braking reaction time given elsewhere (1). There is difficulty in assigning a rigid value to the absolute minimum because all of the drivers involved in a near miss have an opportunity to attempt to avoid collision. The

Table 1. Minimum TMTC values.

Rank	Near-Miss Code Number	TMTC Minimum (sec)	Type
1	3-17	0.20	Rear-end
2	1-1	0.30	Lane-change
3	2-14	0.35	Right-of-way
4	2-6	0.40	Lane-change
5	1-3	0.45	Lane-change
6	1-4	0.55	Lane-change
7	1-2	0.60	Lane-change
8	1-23	0.65	Lane-change
9	2-5	0.70	Cutoff
10	1-14	0.80	Rear-end
11	3-18	0.80	Cutoff
12	3-19	0.80	Rear-end
13	2-1	0.90	Cutoff
14	2-2	0.90	Rear-end
15	3-13	0.95	Lane-change
16	4-1	1.15	Cutoff
17	4-7	1.15	Rear-end
18	4-5	1.20	Cutoff
19	1-13	1.25	Cutoff
20	1-19	1.30	Lane-change
21	3-10	1.35	Cutoff
22	4-10	1.40	Lane-change
23	4-13	1.45	Lane-change
24	2-11	1.50	Right-of-way
25	3-14	1.75	Cutoff
26	1-21	1.80	Cutoff
27	4-9	2.00	Lane-change
28	2-16	2.00	Rear-end
29	1-10	2.15	Rear-end
30	1-7	2.25	Lane-change
31	3-20	2.25	Broadside
32	2-4	2.35	Lane-change
33	4-3	2.40	Lane-change
34	4-8	2.55	Right-of-way
35	1-11	2.75	Rear-end
36	3-15	2.80	Lane-change
37	2-7	3.40	Lane-change
38	4-15	3.95	Cutoff

Figure 1. Study intersection.

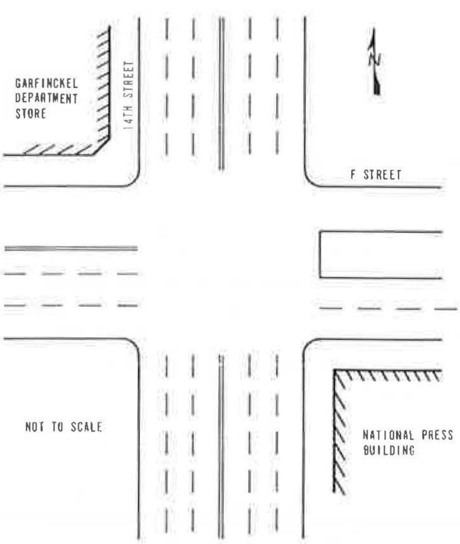
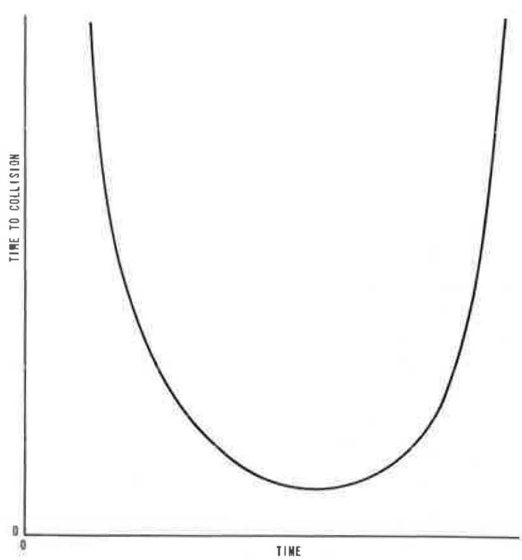


Figure 2. Theoretical TMTC curve.



vehicular response times would be a function of the vehicle itself and the maneuver that the driver directs it to perform. How much avoidance is necessary for one vehicle is a function of the avoidance action taken by the other driver. The ½-sec value represents the time required for one driver to apply his brakes; it does not include the time required to stop. Vehicular response times might be considered effectively zero because each driver can correct simultaneously.

RESULTS

The final results of the data collection and reduction were tabular and graphic computer printouts of each near-miss sequence. Of the 90 sequences that were filmed, 54 were analyzed. The reason for the large discrepancy in the number filmed and the number analyzed was that many sequences were too poor in quality to permit clear visibility on the film analyzer.

Of the 54 sequences analyzed 43 produced usable data. The 11 sequences that were dropped from consideration after they had been analyzed were deleted because the computer analysis method appeared to break down for their situations. The TMTC output was either too erratic or the minimum values were too low to be believable.

The erratic sequences were probably caused by invalid vehicle width assumptions. The left front fender of each vehicle was the only point that was analyzed and transformed for use in motion parameter calculations; but all other fender points were required for the TMTC calculation. The computer program solved for them by using an assumed vehicle length and width and a vehicle direction indicated by the slope of the velocity vector.

When two cars are in a side-by-side position, as they often are in a lane-change or swerving maneuver, the assumptions of width are critical. If the assumed widths are greater than the actual widths, the side-by-side vehicles would be unreasonably close. A small change in heading from parallel paths would cause them to have extremely low TMTC values. When the headings are parallel or divergent, the time to collision is infinite. Each of the four sequences that was disregarded because of the erratic behavior of the TMTC curve involved a parallel movement, a fact that served to validate the previous contention.

If the length of the vehicle is assumed to be larger than the true length, the entire TMTC curve would be moved toward zero. Unlike the width effect, the length effect would cause smooth curves that touch the zero TMTC value. A zero-time-to-collision value implies that the cars have collided, but the films showed this to be untrue. An increase in length increased the real distance between cars in the car-following situation so that as the rear car approached, the TMTC value reduced smoothly but down to an artificially low level. Of the seven sequences discarded for this reason, five involved car-following situtations. The remaining two sequences were concerned with very closely spaced vehicles where either length or width assumptions could have artificially lowered the TMTC curve.

TIME-TO-COLLISION ANALYSIS

Most of the remaining 43 sequences behaved according to the foregoing theory. The curves generally were of the concave upward shape, which the theory suggested to be the near-miss pattern. Figure 3 shows a typical TMTC curve based on data from this research. For five of the remaining 43 near-miss sequences no points on the TMTC curves fell below 999 sec. This indicated that they were never on a collision course. This was not a startling result because there was no preconceived notion of the TMTC measure at the time of data collection. The events were selected as potential near misses only on the basis of definition.

An ordered list of minimum TMTC values is given in Table 1. Each TMTC curve was evaluated for the minimum value in the near-miss zone of the curve. Often, after the TMTC curve had returned to the maximum value, stray points of low TMTC values would appear. This was caused by the width assumption explained previously. These points were not considered in the evaluation of minimum TMTC points, which ensured that the value selected for presentation in Table 1 represented the near-miss phenomenon and not the parallel vehicle inconsistency effects.

The near-miss types given in Table 1 are intended to generally describe the maneuver that was involved in the traffic event. They are as follows:

1. Rear-end—where a following vehicle was forced to stop suddenly to avoid an accident;
2. Lane-change—where a slow-moving car, by changing lanes into the path of a vehicle, caused the faster vehicle to either slow or swerve to avoid an accident;
3. Cutoff—where a turning movement across the path of a second vehicle caused it to alter its motion;
4. Broadside—where a driver passed into the intersection after the caution light had been activated and blocked the path of cross street traffic; and
5. Right-of-way—where two drivers proceeded to the same point and refused to grant a clear path to the other.

The mean TMTC minimum of all sequences that had vehicles on collision paths was 1.46 sec. The mean value was influenced by the large values at the lower end of the scale because the median TMTC value was 1.25 sec. Both values appear to agree with the theoretical values quite closely.

SUGGESTED MINIMUM FOR NEAR-MISS DEFINITION

On the basis of the absolute minimum TMTC values and the empirical values obtained from the TMTC measurements in the filmed sequences, it would appear that 1 sec would be a good threshold limit to impose on the measurement for a near miss. It is recommended that traffic events that display a minimum TMTC value of less than, or equal to, 1 sec should be designated as near misses and that events with greater values should not be counted. If this criterion were applied to the present study, 15 of the 43 sequences analyzed would be classified as near misses.

This level would certainly make the defined near miss an event that would occur frequently enough to satisfy the data-collection requirements of an alternative safety monitoring method. Data were collected at the Washington site for only 9 hours; 90 filmed sequences were made. If the same ratio of 15 defined near misses to 43 analyzed events can be applied to all the data collected, it would seem that near misses occur with a frequency of approximately 3.5 per hour at that site. Because the location shows an approximate accident rate 4- per year, the ratio of near misses to accidents would suggest that a number of near misses equivalent to 1 year's accident history could be collected in 1 day's observation.

SPECIAL CASES

The empirical curves derived from the filmed sequences suggest that a near-miss event is not quite as simple as was theorized. One disagreement between theory and actual data comes about through the existence of double-minimum points within the same TMTC curve. This means that the TMTC value decreases to a minimum and then arises again as predicted. Instead of continuing to rise to infinity, the time to collision drops a second time to a minimum and then goes to infinity. An example of this double-minimum curve is shown in Figure 4.

This result may be explained by assuming that one vehicle makes a second move that places the two cars in danger of colliding. If the first driver places his vehicle in danger of being hit by a second and the second driver acts to avoid the crash, the TMTC curve will dip downward and then begin to rise again. It appears that in double-minimum TMTC circumstances, the first driver senses the action of the second driver and elects to force the issue a second time by again placing his car onto a collision path. The avoidance of the second collision results in the second fall of the TMTC curve.

Another surprising result that deviates from the near-miss theory is the existence of a horizontal TMTC curve at approximately the 2- to 3-sec level of time to collision. It implies that some drivers choose to drive so aggressively that they are on the point of collision for relatively long periods of time. A plot of one of the curves exhibiting this trait is shown in Figure 5. The phenomenon was particularly evident in right-of-way types of sequences where each driver was reluctant to allow the other to proceed in his

Figure 3. Typical empirical TMTC curve.

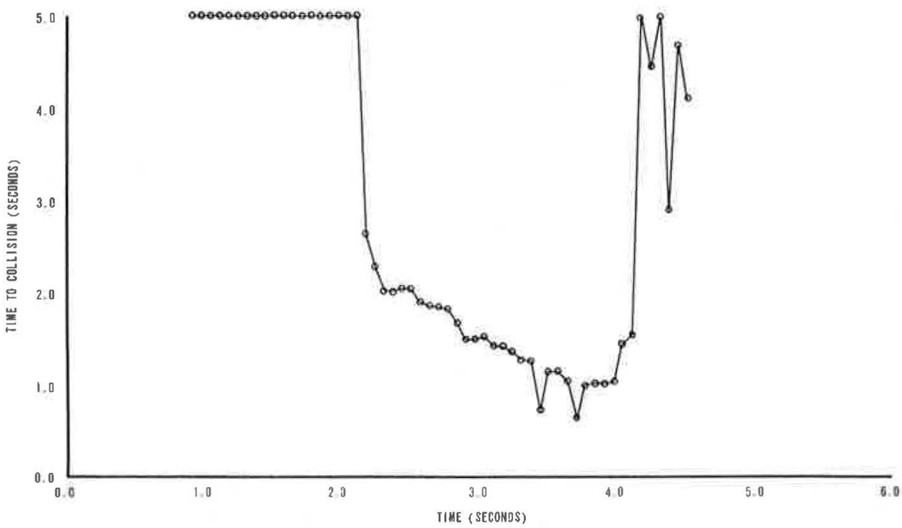


Figure 4. Double-minimum TMTC curve.

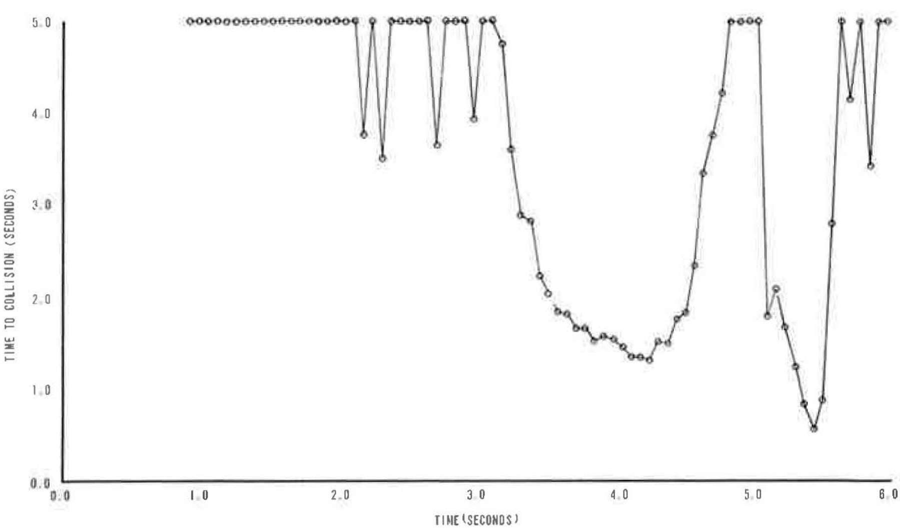
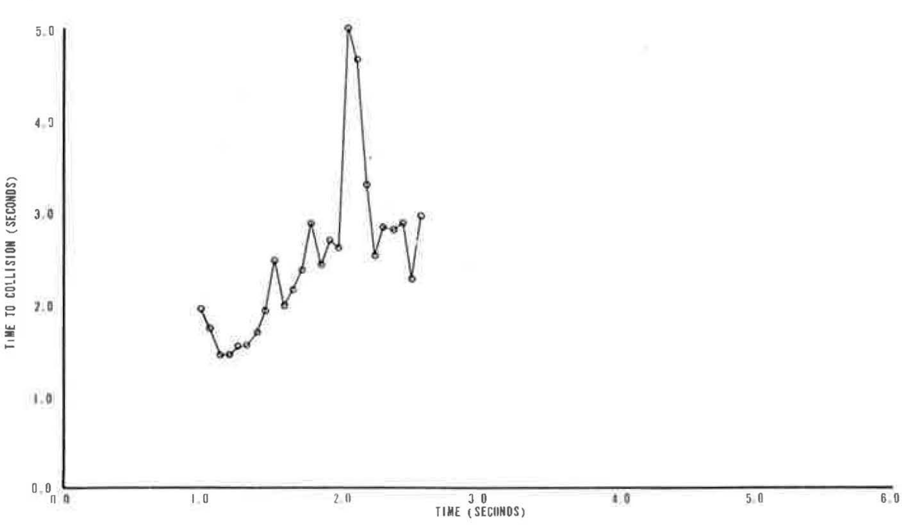


Figure 5. Horizontal TMTC curve.



desired direction. In congested urban locations this kind of excessively aggressive driving is not uncommon and could be expected to produce near misses.

HUMAN OBSERVER TRAINING

It is obvious that the elaborate equipment and data reduction methods employed in this study cannot be duplicated at every site where near-miss counts are desired. The near-miss method could never be justified if photographic data were a requirement for data collection. The following section is intended to present an outline of further long-range research of near misses that must be undertaken if the concept is to progress past the definition phase.

It would be desirable to investigate the possibility of teaching human observers to recognize near-miss situations as defined by the TMTC criterion. Although the inaccuracy of subjective judgment is the very thing that this paper intended to remedy, it is anticipated that with proper training the human judgment factor could be reduced to a tolerable level for data collection uniformity. Drivers are constantly required to judge the time to collision for their own vehicles while maneuvering in traffic, and they seem to do it relatively accurately. Discussions with Richard A. Olsen, a psychologist at the Pennsylvania Transportation and Traffic Safety Center, have indicated that observers could be trained to recognize a level of time to collision with the accuracy of recognition dependent principally on the intensity of their training.

One experimental approach to accomplish this goal would be to investigate several locations using observers and films simultaneously. A threshold level of 1 sec would be imposed on the TMTC curve to define near misses, and the observers would be instructed to record those incidents falling below that level. Continuous filming of the intersection would be performed during the observation period so that a visual record of all events could be made. The films should be of sufficient quality that frame-by-frame analysis could be carried out on those events where the TMTC value was thought to be near the threshold level.

The reduced films could be used as a checking, as well as a teaching, device for the human ground-level observers. Measurements of how many near misses (as defined by film analysis) the observers failed to detect as well as how many events recorded by them actually could not be classed as near misses could be made. The results could be presented to the observers in order that they might learn from their mistakes. Teaching methods could be established that would optimize the training procedure in order to produce qualified near-miss observers and observation techniques.

The outcome of this phase of near-miss implemention would be a measure of the accuracy and uniformity of human-observer near-miss detection. If observers cannot be trained to recognize the event specified here based on a TMTC criterion, then at this point very little more can be done in a safety measure direction. In this case, the only useful result of this phase of the implementation program might be in the extensive collection of filmed near-miss sequences. They could be used in a microscopic analysis of drivers' accident avoidance processes.

ACCIDENT CORRELATION

If the results of the observer training proved to be satisfactory, the next step toward implementation would be the correlation of near-miss statistics with accident statistics. It is necessary for this correlation to exist or the count of near misses would indicate nothing about the accident trends of a location.

This could be accomplished by using the trained human observers to investigate locations using the near-miss measure. The results of many observations would be compared to the accident histories of the sites studied, and correlation coefficients would be computed.

High relationships between accidents and near misses must exist for each type of accident. Possible near-miss classifications that might be used would be the type of accident that would have occurred if avoidance attempts had been unsuccessful. High correlations within types would suggest that a multiplier could be adopted to transfer from near miss to accident rates for each accident type.

The problems and expense that accompany this step would be greatly reduced by using observers instead of cameras. The only data required would be those that were collected by experienced observers. Data reduction would be minimal. Therefore, many sites could be investigated, which is a highly desirable situation for accurate correlation results.

PROGRAM ESTABLISHMENT

The final step needed to turn the use of near misses into a traffic engineering tool is to establish a monitoring program that uses them. Once it has been proved that near misses are good indicators of accident histories, an adequate program to periodically survey the locations within an area could be instituted. Near-miss counts could be taken as routinely as traffic volume counts, and summaries of the findings could be prepared for use in high accident location detection and subsequent safety improvements determination.

A continuing training program for near-miss observers should be set up to ensure that the uniformity of judgment that is so essential in near-miss counting persists. This might be accomplished in several ways, depending on the results of the original observer training phase. One method might be to set up a permanent training facility, possibly incorporated into the driver testing facilities that are becoming fairly common across the country. Also, a program of near-miss training might be accomplished by showing filmed near misses to the trainees and grading them on their interpretations of the films.

CONCLUSIONS

The near-miss definition as embodied by the TMTC measure is a valid indicator of danger for two-vehicle interactions. The results of this research show that the TMTC value provides a basis for ranking traffic events according to the danger that they generate. The theoretical TMTC curve can be shown to fit most of the empirically derived curves drawn from the films taken of near misses. The curves differ only in minimum TMTC values, from which it can be concluded that the danger involved can be quantitatively represented by that value.

A human observer is a good judge of the TMTC curve even though he may not be aware of the theory involved. The films of near misses analyzed in this research were taken by pure observation of the urban intersection. The only notion of a near miss was defined by the original definition of a traffic event that produces more than an ordinary amount of danger to the drivers involved. This loose definition, when applied in observation, resulted in very few sequences where the TMTC value failed to fall below 5 sec. The observation technique was to look at television monitors rather than at the actual site so that perhaps even higher observation accuracy would be attained by live viewing. The prospects of training observers to recognize rigid definitions such as 1-sec TMTC minimum value appear to be promising.

The minimum TMTC point in the near-miss sequence occurs before the minimum distance between vehicles is reached in the sequence. From examination of the near-miss curves, it is generally true that these two points are not the same on the time axis. The explanation of this effect can be seen if a near-miss event is pictured in one's mind. If a collision does not occur, the frantic maneuvering to avoid it is performed at the same time that the vehicles are closing in on one another. When the avoidance is completed, the vehicles become more under control with respect to each other so that they might pass closer to one another with more confidence. This is the case in a braking or swerving type of near miss where very close distances between vehicles are common even though the danger may be slight. Vehicles are nearest to each other in a swerving situation when they are side by side. Unless the vehicles are traveling on intersecting paths, however, they have little chance of colliding. The TMTC value reflects the idea that distance between vehicles is not the most dangerous point in the interaction sequence.

Some drivers appear to maintain a constant TMTC value of 3 or 4 sec throughout a sequence. A few sequences when plotted showed that the TMTC curve was nearly horizontal throughout the analyzed time. It seems obvious that in this situation both of the participants were driving very aggressively and were refusing to grant the right-of-way to the other. This seemed to be true in the dangerous case of two cars trying to change into the same lane simultaneously. Neither would give way to the other so that an impending collision was present for a relatively long period of time.

Double-minimum points were noted in some near misses. This is probably the result of a less severe case of the same type of driving behavior that produced a horizontal TMTC curve. Perhaps in this double-minimum condition there is one aggressive driver forcing the situation on a second defensive driver. If the aggressor puts himself onto a collision path and the defensive driver grants the right-of-way, the TMTC curve moves upward. The second minimum point is caused by a second aggressive action that requires a second defensive maneuver, which restores high TMTC values.

ACKNOWLEDGMENTS

This study was sponsored as an in-house research project by the Pennsylvania Transportation and Traffic Safety Center. Films were provided by the Federal Highway Administration through the cooperation of Merton J. Rosenbaum, Jr. James I. Taylor and Thomas B. Davinroy contributed many helpful suggestions in the planning and execution of the research program.

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