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4 Reports

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Foreword

As transportation research strives desperately to keep up with the ever-increasing demands of a strong economy and a mobile population, more effective ways of evaluating contemplated improvements and reducing of complex tasks such as driving to mathematical comparison have been taking place. While many of these rather complex techniques still remain to be applied by the practitioner, they certainly have indicated their potentials in research and a plethora of new research is continually being evolved utilizing mathematical solutions.

The first paper in this RECORD, by two Franklin Institute researchers, is concerned with the development and evaluation of a simulation model concerned with the use of remedial devices thought to aid passing on two-lane rural roads. The model uses inputs of roadway data and vehicular data as well as traffic volumes and passing data. The output of the model (in statistical terms) is being used to determine the relative benefits of alternative sensing devices of the system elements in terms of safety and throughput.

A paper by two Purdue University researchers is concerned with the evaluation of conditions for which the construction, maintenance and interest costs for a median left-turn lane would be warranted. Vehicle delay times and accident rates were used as criteria, and it was found that delay times and accidents are reduced when median left-turn lanes are constructed under most circumstances.

The last paper, by a group of California traffic researchers, concerns the effects of minor traffic improvements on accidents and performance. Flashing beacons, safety lighting, delineation and guard installations were studied. It was found by before-and-after studies that flashing beacons and safety lighting were effective if used where warranted (and the research sets forth cost/benefit studies) but delineation and guardrail improvements, while effective in many instances, need more study and evaluation.

An abstract taken from work done on an NCHRP project which was presented in full at the 47th Annual Meeting of the Board is also presented. This work by Texas Transportation Institute researchers is concerned with the development of techniques of analyzing operation of major traffic interchanges.

This RECORD has pointed out some of the various ways mathematical models and skilled evaluation can be used to judge contemplated improvements or to resolve judging between alternates. It should be of considerable interest to those who must employ these techniques in arriving at solutions and of passing interest to those who must make decisions in transportation planning, construction, maintenance and operations.

Contents

A Simulation Model of a Two-Lane Rural Road

ARNO CASSEL and MICHAEL S. JANOFF, The Franklin Institute Research Laboratories.

A simulation model has been developed to evaluate traffic flow and safety benefits arising from use of remedial devices which would aid passing maneuvers on two-lane rural roads. Inputs to the model are arbitrary and consist of road configuration data, vehicle data, traffic volumes, and passing probability data. The output statistics can be used to determine the relative benefits of alternative remedial aid systems in terms of safety and throughput.

Initially the effects of no-passing zone configurations due to road geometry and knowledge of oncoming car speed on tangents were investigated. The results show that no-passing zones cause a marked decrease in throughput, while oncoming car speed information appears to have a beneficial effect on safety. Additional runs will be made to study the effects of other passing rules on traffic flow and safety.

•THE Franklin Institute Research Laboratories (FIRL) has recently completed a study for the Bureau of Public Roads on the conceptualization of the overtaking and passing maneuver on 2-lane rural highways (1) and is presently developing functional specifications for remedial aids to assist drivers in solving discriminatory and judgmental problems associated with overtaking and passing. This study is in support of the Bureau of Public Roads program to minimize rear-end and head-on accidents and to increase the service volume on 2-lane rural highways.

The first study has shown that passing performance can be significantly improved if drivers are given additional information such as oncoming car speed or closing rate.

The present study is concerned with the development of functional specifications for cost effective remedial aids which would provide the driver with this additional information. This study uses the results of the completed program and other related research.

To evaluate the traffic flow and safety benefits of alternative remedial aids a computer simulation model has been developed. This model and some initial results are described in this paper.

TRAFFIC FLOW MODEL

General Description

The traffic flow model is the primary means for evaluating the effectiveness of alternative remedial aids in terms of both traffic flow and traffic safety. The model can simulate the movement of vehicle traffic on 2-lane rural roads with various road geometries and traffic volumes. During the simulation, which spans a specified interval of time, vehicles will, under certain conditions, attempt and execute passing maneuvers in order to attain and maintain their individual desired speeds. These conditions generally depend on the relative position and speed of each of the vehicles on the length of the road at a particular point in time. Although this road is assumed to be straight and

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level, the restrictions placed on the traffic flow by a more general configuration of road geometry are achieved in this model by specifying "no-passing" zones and sight distance restrictions for each direction of travel. Additionally, slow-down factors can be used for curves and grades if desired.

The elements of traffic, the vehicles in each lane, are introduced into the model from both ends of the road. Speed distributions and headway distributions are predetermined so that the traffic configuration will simulate some prescribed volume. The method is completely arbitrary to the extent that the time of entry and speeds of all introduced vehicles are at the discretion of the user. These data, together with other data associated with each of the vehicles entering the traffic pattern at any time (such as its desired speed, its actual speed, and its state of maneuvering), need only reflect plausible physical conditions, and are otherwise unrestricted.

The dynamic operation of the model consists of determining at any time the future picture of the traffic flow at some appropriately selected incremented time based on the present picture and on various estimates and decisions of the drivers of the vehicles. A maneuver by any vehicle to pass one or more (up to a maximum of 5) leading vehicles in its lane will be attempted, if certain conditions are satisfied. Some of the more important ones are as follows:

1. The vehicle is currently constrained to travel at a speed less than its desired speed, because of the speed of the leading vehicle (i.e., the vehicle immediately in front of it).
2. After overtaking 1 to 5 vehicles in a passing maneuver, there is a sufficiently large gap to allow a safe return to the traffic flow.
3. The vehicle is in a passing zone and will remain there until completion of the pass.
4. The probability of passing, as a function of the oncoming gap and lead car speed yields a number which is greater than the next random number.

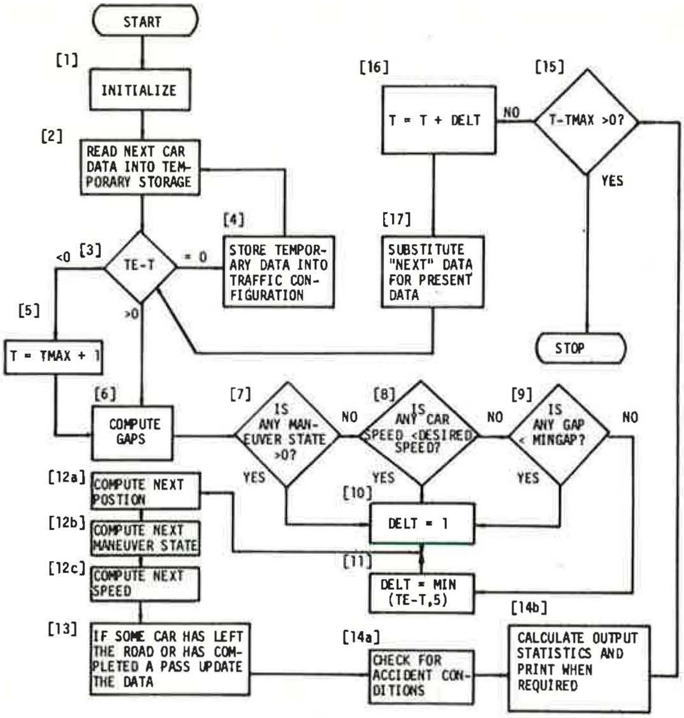
At any point in time during the simulation each vehicle on the roadway under observation will be in any one of four possible maneuver states indicating the following four different phases of maneuvering utilized in this model:

Maneuver State	Phase of Maneuvering
0	Traveling in its normal lane.
1	Decision to initiate a passing maneuver.
2	Traveling in the oncoming lane while passing one or more cars in its own lane.
3	Terminating the passing maneuver by re-entering its normal lane.

If, during maneuver state 2, a driver changes his mind about going through with the pass, because it becomes necessary to complete or abort the pass sooner than originally anticipated, certain latitudes of action are available by which the involved vehicle can accelerate or decelerate in the proper time to the speed required to re-enter its normal lane in order to avoid an accident. Only accelerative type passes are being considered in the model.

Programs

The traffic model (Fig. 1) can be most easily described in four sections:



*Classification of Symbols: T = Present time; DELT = Time increment; TMAX = Total Simulation time; TE = Time of entry of next vehicle.

Figure 1. Flow chart of the traffic model.

Main routine controls all input, updatings due to position changes, and printing of output statistics. This is steps 1-12a, 13, 14b-17 of the flow chart (Fig. 1).

Subroutine maneuver calculates the "next" maneuver state of each vehicle. This is step 12b.

Subroutine speed calculates the "next" speed of each vehicle. This is step 12c.

Subroutine accident investigates possible accident conditions and decides what type of correction can be made in the passing maneuver to avoid an accident. This is step 14a.

Main Routine—The most important function of the main routine is to calculate output statistics and to print results when necessary. Printing of results can occur in various ways, such as,

1. At every time increment;
2. At every time increment at which a vehicle is attempting to pass;
3. At every time increment at which a vehicle is actually passing;
4. Only when a possible accident condition occurs;
5. At equal time intervals;
6. At termination only.

The last two seem to be the preferred methods, since they yield enough output statistics and do not increase computer time to any great degree. Other functions of the main routine are as follows:

1. Computation of gaps;
2. Computation of positions;
3. Computation of the time increment;
4. Updating due to position changes (one vehicle passing another vehicle); and
5. Computation of speed changes.

Subroutine Maneuver—This is the heart of the model since it calculates exactly what each vehicle is going to do at every time increment. If the conditions mentioned earlier, along with several other conditions, are satisfied, then the final decision to pass is calculated as follows:

> As a result of observational studies on tangent sections of 2-lane public highways previously conducted by the Franklin Institute Research Laboratories for the Bureau of Public Roads (1) it was found that the decisions made by drivers whether to attempt to pass or not are based on many factors, but that the decisions, once made, closely resemble probability distribution curves with oncoming gap and lead car speed as independent variables. Because of this, it was decided to incorporate these probability curves into the model and to rely on them for the final decision to pass. These curves were derived for day-light conditions only.
>
> Given on oncoming gap and a lead car speed, the probability of passing is determined by linear interpolation of the probability curve in the appropriate region.

These calculations are done vehicle by vehicle, and are repeated whenever the conditions are such that a vehicle is in a potential passing situation. If a vehicle does not pass, then it remains in maneuver state 0 until the next opportunity. If a vehicle does attempt to pass, a check is made at each time increment to decide if the pass will be completed safely or if other action must be taken.

Subroutine Speed—Subroutine speed merely calculates a vehicle's next speed as a function of its present and next maneuver state.

Subroutine Accident—Subroutine accident checks each passing maneuver to decide if corrective action such as acceleration or deceleration is needed to complete the pass sooner or abort it. This is done by projecting what the oncoming gap will be when the pass is completed and then testing the effects of increasing the speed to complete the pass in less time or decelerating and returning to the normal lane. No vehicle other than the passing vehicle is altered. Projected accidents where no avoidance procedures can be taken in the model to deter a possible accident are also identified.

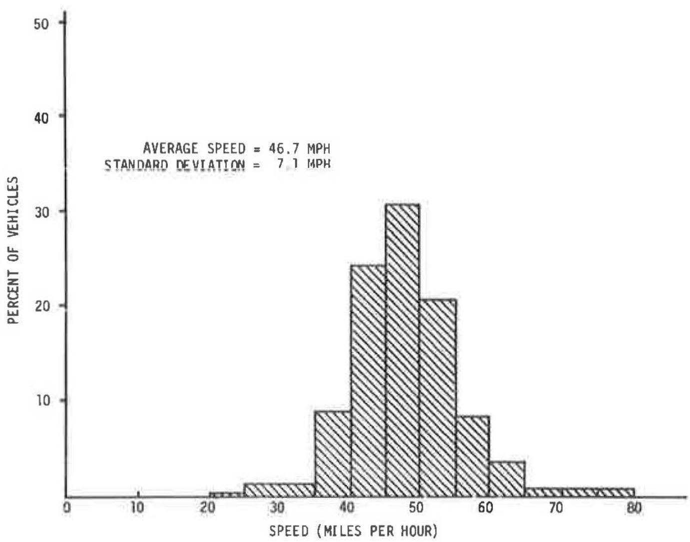


Figure 2. Speed distribution.

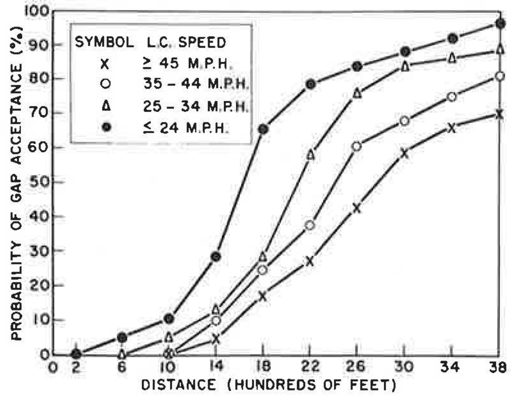


Figure 3. Gap acceptance without knowledge of oncoming car speed.

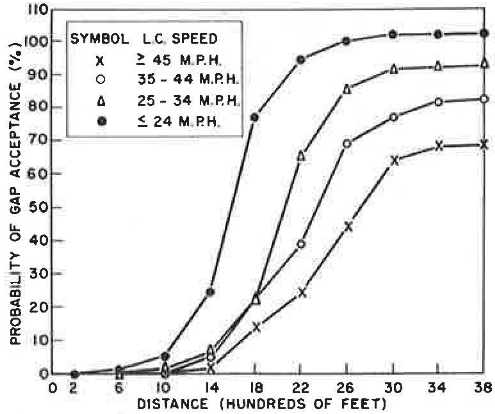


Figure 4. Gap acceptance with knowledge of oncoming car speed.

Input Data

The inputs to the model consist of road configuration data, vehicle data, and passing probability data.

Road Data—The road data consist of road length, no-passing zone configurations, sight distance restrictions, and the maximum simulation time to be used. These numbers are all read into the program and do not change during a given simulation.

They may be varied in order to obtain various roadway geometries with arbitrary lengths in order to simulate the effect of remedial aids on traffic flow and safety under different conditions.

Vehicle Data—The vehicle data consist of desired speed (which equals actual speeds upon entering), maximum speeds, headway time gaps (which determine a vehicle's time of entry), and maneuver state (which is 0 upon entering). Each vehicle's desired and maximum speed do not change throughout the simulation, but this could be done if desired. An acceleration rate of 5 ft/sec^2, and a maximum emergency deceleration rate of 20 ft/sec^2 are used—these are average vehicle data from the Traffic Engineering Handbook (2). These also do not change throughout the simulation. The speed distributions used in the model were determined from observational studies completed previously (1) and are typical free speeds on 2-lane rural roads in southern New Jersey, and from the literature. The speed distribution in Figure 2 is approximately normally distributed with a mean of 46.7 mph and a standard deviation of 7.1 mph. The headway distributions have been taken directly from the Highway Capacity Manual (3) and are in the form of a modified Poisson distribution. Ranges from 100 to 800 vph have been used successfully in the model. Other headway and speed distributions can be used in the model as desired.

The desired speeds and headway time gaps are randomly assigned to the vehicles by a separate data preparation program before each simulation begins.

Passing Probability Data—The passing probability data (Fig. 3) consist of four probability curves obtained from the observational studies on public highways mentioned previously. The curves show the percent of passing opportunities accepted as a function of lead car speed and oncoming gap, and are based on passing behavior without giving drivers information on oncoming car speed.

Any remedial aid would change the passing behavior of the drivers, hence also changing the probability distribution curves which are now being used in the model. Several experimental studies have been conducted on closed roads as part of the previous program to determine the effect on passing behavior of providing drivers with information such as oncoming car speed or closing rate as compared to no knowledge conditions. These controlled road tests yield new probability distributions describing passing behavior if drivers were given information on oncoming car speed or closing rate.

Additional experimental studies can be run if it is desired to test the effect of giving drivers other information such as distance to an oncoming car or to the end of a passing zone. These distributions have been corrected so that they would simulate actual passing behavior of drivers using a remedial aid providing oncoming car speed (OCS). This was accomplished by noting that in the controlled tests, providing drivers with OCS caused a 50 percent reduction in the variance of passing gap acceptance without changing the mean passing time. The real curves were then adjusted so as to reflect this 50 percent reduction in variance. The new curves (Fig. 4) are merely the original set (Fig. 3) with a 50 percent reduction in variance. The means of both sets are the same based on results of the experimental studies on closed roads.

Output Statistics

The output statistics, which can be used to determine the relative benefits in terms of safety and throughput, consist of the following:

- Volume;
- Average speed and standard deviation;
- Number of attempted and completed passes and aborts;
- Number of vehicles passed and percent of multiple passes;
- Amount of delay (seconds) suffered by the vehicles which leave the road;
- Number of possible accident conditions termed emergency indicators, when some type of evasive (i.e., acceleration or deceleration) action must be taken, during a passing maneuver;
- Number of projected accidents, when no evasive action can be taken in the model to deter a possible accident;
- Average safety margin (average time to meeting of oncoming car after completion of a pass); and,
- Number of speed change cycles.

An increase in the output volume in an equal time interval would indicate an increase in throughput.

An increase in average speed by use of a remedial aid would cause an increase in throughput on the road. If the standard deviation decreased it would signify that traffic is flowing more evenly.

An increase in the number of passes and number of vehicles passed taking place under identical traffic conditions would signify a better use of passing opportunities and hence should increase throughput.

A reduction in the amount of delay caused to a given group of vehicles would show that the remedial device has a beneficial effect on throughput, since vehicles would be traveling at speeds which differ less from their desired speeds.

An increase in the average safety margin and decrease in the number of emergency indicators would signify safer passing conditions on the road.

A reduction in the number of speed change cycles would signify a smoother flow of traffic.

The output statistics can be used to determine the relative benefits of alternative remedial aid systems in terms of safety and throughput. The prime benefit measures for each remedial aid are effect on road user costs including motor vehicle running costs, time costs, and accident costs. Changes in speeds, delay and other output statistics can be converted to dollars to develop estimates of annual savings associated with a given type of remedial aid system for all 2-lane rural roads or some roads which have more than a given ADT. By use of cost-benefit analysis techniques, optimal solutions for remedial aid systems can then be developed.

The program is written in FORTRAN IV and is presently being run on a UNIVAC 1107 at the Franklin Institute.

USES OF THE TRAFFIC FLOW MODEL

The primary use of the traffic flow model is to evaluate the effects of remedial aids for passing maneuvers on traffic flow and safety. Two basic applications have been considered:

1. Use of existing no-passing zones for passing maneuvers by providing drivers with information describing the opposing traffic (e.g., positions and speeds of oncoming cars).
2. Providing drivers with oncoming car speed (OCS) or closing rate on tangents.

A series of simulations was run for each of these applications.

No-Passing Zones

Each of the following series included four simulations: 0, 25, 50, and 70 percent no-passing zones. Each simulation was accomplished using a 30,000-ft road, a 50-50 traffic directional distribution, 10-15 percent heavy trucks, and the no-knowledge passing rule mentioned previously. Passing was allowed only on tangent sections.

Series Number	Volume (vph)	Speed (mph)
1	100	46.7
2	200	41.7
3	200	46.7
4	200	51.7
5	400	41.7
6	400	46.7
7	400	51.7
8	600	46.7

Knowledge of Oncoming Car Speed

The same eight series were re-run using the passing rule derived when knowledge of oncoming car speed was provided to the drivers. For both applications, other directional distributions and road lengths were also simulated to test the sensitivity of the model. The results of some of these simulations are described in the following sections.

RESULTS OF THE TRAFFIC FLOW MODEL

Effect of No-Passing Zones

The first use of the model was to study the effects on traffic flow of various geometric configurations on 2-lane rural roads. Representative roads were studied in flat (southern New Jersey), rolling (foothills of Virginia), and mountainous (Vermont, New Hampshire, Maine, and Virginia's Skyline Drive) terrain to collect data describing various road configurations which were used to generate typical configurations for the model. It was determined from these data that 25, 50, and 70 percent no-passing zones approximated the configurations found on these three types of roads, respectively. A method developed by Stanley R. Byington of the Bureau of Public Roads was then used to derive the arrangement and spacing of the no-passing zones.

Table 1 gives the output of the 16 runs. The most significant overall results on traffic flow with increasing traffic volume were the following:

1. A decrease in average speed and its standard deviation.
2. An increase in delay.
3. An increase in the number of passes and aborts.
4. An increase in the number of vehicles passed.
5. An increase in the number of possible accident conditions (emergency indicators).
6. A decrease in the average safety margin.
7. An increase in the number of speed change cycles.

The following comparisons were made with Normann's data (4, 5, 6) for 0 percent no-passing zones: (a) total number of passes per hour per mile, and (b) number of passes per vehicle per hour per mile. The results are shown in Figure 5.

TABLE 1

EFFECT OF NO-PASSING ZONES WITHOUT KNOWLEDGE OF ONCOMING CAR SPEED

Operation	Run Number															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Input:																
Volume, vph	100	100	100	100	200	200	200	200	400	400	400	400	600	600	600	600
Speed, ft/sec	68.5	63.5	68.5	68.5	68.5	68.5	68.5	68.5	68.5	68.5	68.5	68.5	68.5	68.5	68.5	68.5
Standard deviation, ft/sec	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4
% No-passing zones	0	25	50	70	0	25	50	70	0	25	50	70	0	25	50	70
Output:																
Volume, vph	113	113	113	113	237	237	237	237	371	371	371	371	546	543	541	539
Speed, ft/sec	67.6	65.9	65.4	63.8	67.3	66.0	63.9	62.0	66.4	64.1	61.8	57.1	63.6	61.4	59.0	54.8
Standard deviation, ft/sec	9.5	3.8	8.2	9.6	8.0	7.0	7.3	7.6	7.9	7.1	7.4	9.8	6.7	6.4	6.4	8.6
Delay/hour/mile, sec	25	106	296	621	196	501	1177	1828	784	1826	3051	6348	2643	4232	6195	10646
Delay/vehicle/mile, sec	0.2	0.9	2.6	5.5	0.8	2.1	5.0	7.7	2.1	4.9	8.2	17.1	4.8	7.8	11.5	19.8
No. attempted pass/hour/mile	11	9	7	3	39	27	19	8	88	59	35	18	151	99	58	29
No. passes/hour/mile	11	9	7	3	38	26	18	7	85	56	32	16	138	87	51	23
No. pass/vehicle/hour/mile	0.10	0.08	0.06	0.03	0.16	0.11	0.08	0.03	0.23	0.15	0.09	0.04	0.25	0.16	0.09	0.04
No. aborts/hour/mile	0	0	0	0	1	1	1	1	3	3	3	2	13	12	7	6
No vehicles passed/hour/mile	12	10	7	5	40	28	22	9	93	69	41	24	162	103	65	32
Multiple passes, %	5	14	5	37	6	7	22	21	10	23	29	49	16	18	26	39
No. em. ind/hour/mile	1	1	1	0	6	5	3	2	20	16	6	2	53	29	13	5
Average safety margin, sec	33.8	35.9	38.6	31.9	15.7	14.4	14.9	13.7	13.3	12.0	12.8	19.0	8.3	9.0	11.0	14.0
No. speed ch/hour/mile	22	20	17	12	83	69	70	51	213	188	158	150	433	399	296	274
No. speed ch/vehicle/hour/mile	0.2	0.2	0.2	0.1	0.4	0.3	0.3	0.2	0.6	0.5	0.4	0.4	0.8	0.7	0.5	0.5

TABLE 2

EFFECT OF NO-PASSING ZONES WITH KNOWLEDGE OF ONCOMING CAR SPEED

Operation	Run Number															
	17	18	19	20	21	22	23	24	25	26	27	26	29	30	31	32
Input:																
Volume, vph	100	100	100	100	200	200	200	200	400	400	400	400	600	600	600	600
Speed, ft/sec	68.5	68.5	68.5	68.5	68.5	68.5	68.5	68.5	68.5	68.5	68.5	68.5	68.5	68.5	68.5	68.5
Standard deviation, ft/sec	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4
% No passing zones	0	25	50	70	0	25	50	70	0	25	50	70	0	25	50	70
Output:																
Volume, vph	113	113	113	113	237	237	237	237	371	371	371	371	544	542	540	539
Speed, ft/sec	67.6	66.6	65.0	63.0	-67.1	65.7	63.4	61.3	65.9	63.2	60.6	55.6	62.9	60.5	58.1	54.1
Standard Deviation, ft/sec	9.5	8.6	8.3	9.8	7.9	7.1	7.4	8.3	7.9	7.6	8.1	11.1	7.0	6.7	7.1	8.6
Delay/hour/mile, sec	29	138	359	751	233	600	1348	2140	1013	2347	3820	7853	3218	5025	7046	11242
Delay/vehicle/mile, sec	0.3	1.2	3.2	6.6	1.0	2.5	5.7	9.0	2.7	6.3	10.3	21.2	5.9	9.3	13.0	20.9
No. attempted pass/hour/mile	11	8	5	1	37	24	15	6	82	50	26	16	133	84	42	17
No. passes/hour/mile	11	8	5	1	37	24	15	6	80	46	24	14	124	77	39	14
No. pass/vehicle/hour/mile	0.10	0.07	0.04	0.01	0.16	0.10	0.06	0.03	0.22	0.12	0.06	0.04	0.23	0.14	0.07	0.03
No. aborts/hour/mile	0	0	0	0	0	0	0	0	2	4	2	2	9	7	3	3
No. vehicles passed/hour/mile	11	9	6	2	39	27	16	7	91	55	29	21	151	99	49	36
Multiple passes, %	5	14	13	38	7	10	10	35	13	19	21	48	21	28	26	(≈75)
No. em. ind/hour/mile	1	1	0	0	4	5	2	1	14	7	4	2	37	19	5	4
Average safety margin, sec	36.8	39.1	48.6	24.0	17.2	15.1	17.8	22.2	14.4	13.9	15.8	19.1	10.3	10.7	18.0	18.3
No. speed ch/hour/mile	21	18	16	11	79	63	61	47	205	173	143	131	414	385	248	247
No. speed ch/vehicle/hour/mile	0.2	0.2	0.1	0.1	0.3	0.3	0.3	0.2	0.6	0.5	0.4	0.4	0.8	0.7	0.5	0.5

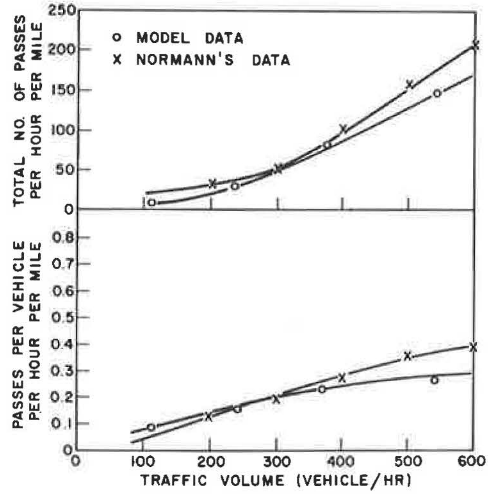


Figure 5. Comparison of model data with Normann's data.

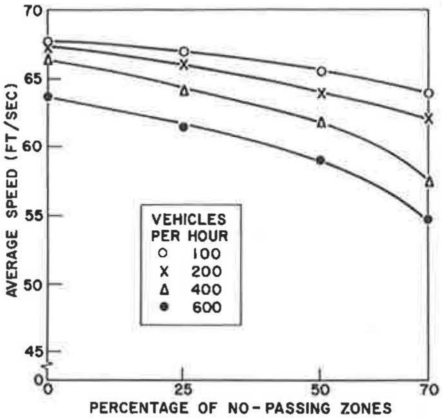


Figure 6. Effect of no-passing zones on average speed.

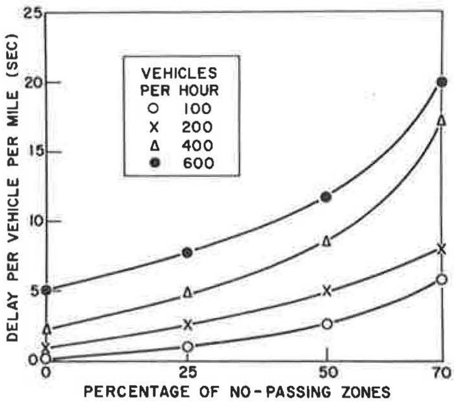


Figure 7. Effect of no-passing zones on time delay.

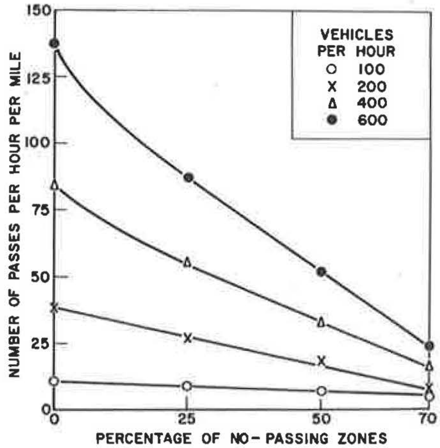


Figure 8. Effect of no-passing zones on passing maneuvers.

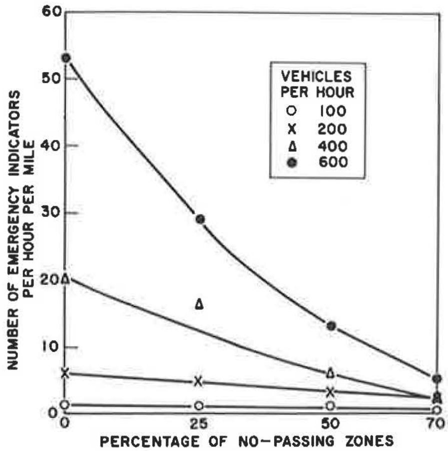


Figure 9. Effect of no-passing zones on emergency indicators.

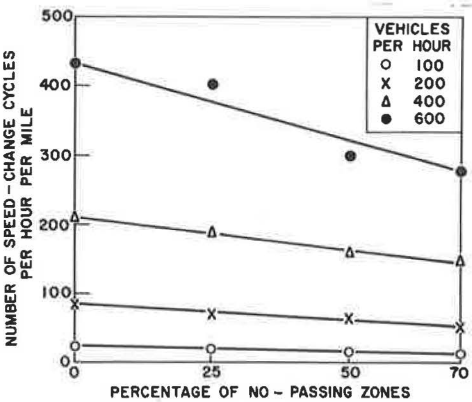


Figure 10. Effect of no-passing zones on speed change cycles.

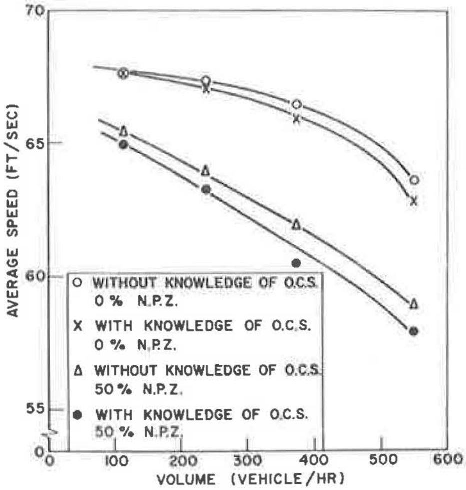


Figure 11. Effect of knowledge of OCS on average speed.

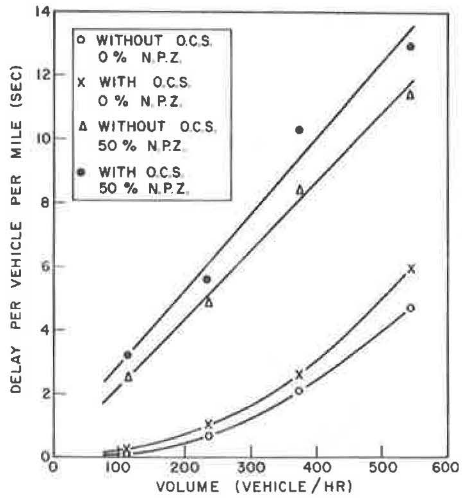


Figure 12. Effect of OCS on time delay.

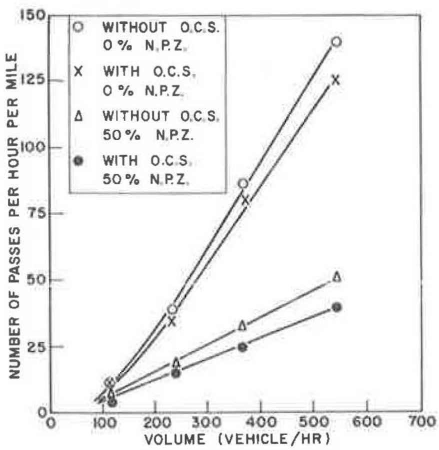


Figure 13. Effect of knowledge of OCS on passing maneuvers.

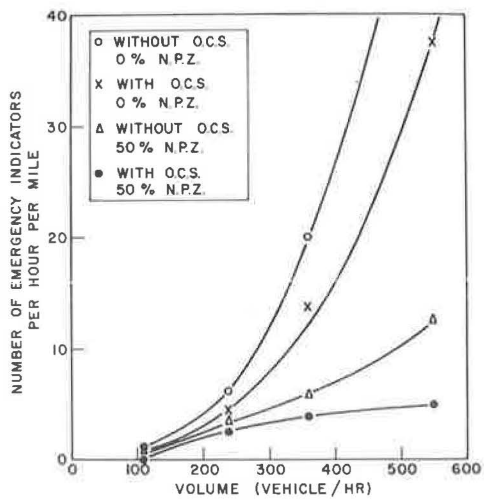


Figure 14. Effect of knowledge of OCS on emergency indicators.

In both instances there seems to be good agreement, at least between the traffic volumes of 100 to 600 vph. Any differences in results could be attributed to the fact that we used a 50-50 directional distribution while in Normann's data a ⅔-⅓ is used. All of the preceding help to verify the reliability of the model to describe actual road conditions at different traffic volumes.

The following effects were due to no-passing zone configurations.

1. A decrease in average speed as the percentage of no-passing zones increased (Fig. 6).
2. An increase in delay as percentage of no-passing zones increased (Fig. 7).
3. A decrease in the number of passes and number of vehicles passed but an increase in the percentage of multiple passes as the percentage of no-passing zones in-

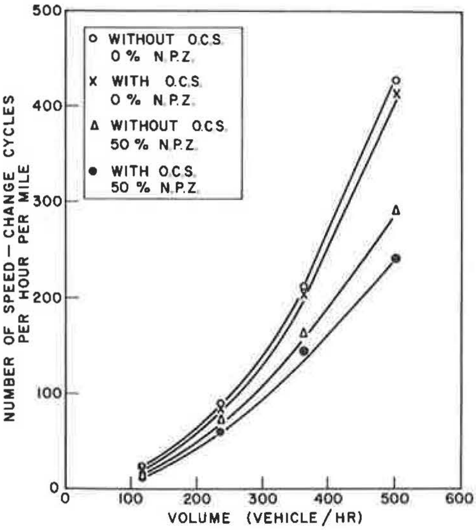


Figure 15. Effect of knowledge of OCS on speed change cycles.

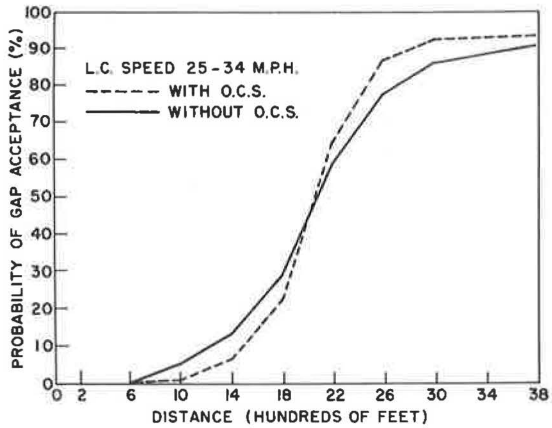


Figure 16. Comparison of passing probabilities with and without knowledge of oncoming car speed.

creased. Figure 8 shows the number of passes per hour per mile as a function of volume and the percentage of no-passing zones.

4. A decrease in the number of possible accident conditions (emergency indicators) as the percentage of no-passing zones increased (Fig. 9).

5. A decrease in the number of speed change cycles as the percentage of no-passing zones increased (Fig. 10).

There was little difference in results when a longer road, or different directional distributions were used while input speed was inversely related to the number of passes.

Effect of Knowledge of Oncoming Car Speed

The traffic flow model was run under the conditions mentioned previously to disclose the effects of providing drivers with knowledge of oncoming car speed. The results of the 16 simulations are given in Table 2.

The effects of providing drivers with knowledge of oncoming car speed as compared to no-knowledge (the previous 16 simulations) were the following:

1. A decrease in the average speed (Fig. 11).
2. An increase in delay (Fig. 12).
3. A decrease in the number of passes (Fig. 13), number of aborts, and number of vehicles passed.
4. A decrease in the number of possible accident conditions. Figure 14 shows the number of emergency indicators with and without knowledge of oncoming car speed for different traffic volumes with 0 and 50 percent no-passing.
5. An increase in safety margin ranging from 9 percent at 100 vph to 24 percent at 600 vph for 0 percent no-passing.
6. A decrease in the number of speed change cycles (Fig. 15).

These results can be partly explained by Figure 16. This is merely one of the curves of Figure 3 and the same curve, with a 50 percent reduction in variance, as shown in Figure 4. The lower part of the curve has been depressed, whereas the upper part has been elevated by the 50 percent reduction in variance.

These changes indicate the following:

1. Unsafe passes or passes with small oncoming gaps have a much lower probability of acceptance, hence occur less often.

2. Large gaps, though accepted now with greater probability, are not as significantly changed as are small gap acceptance probabilities, i.e., they had a high probability of occurrence before and after.

From item 1, a reduction in unsafe conditions, passes and aborts has been caused, hence also a reduction in throughput, since drivers are forced to wait for larger gaps. This would also increase the average safety margin. Also since the probability of accepting gaps $\geq$ 2,500 feet is so high already, the slight increase in performance at the upper end of the curve does not offset the loss in throughput suffered by depressing the lower end. This seems to verify the results which were calculated in the simulation model for the 16 runs.

The reduction in the number of speed change cycles is a consequence of both a lower standard deviation from the mean speed, and the decrease in the number of passes.

Similar results were also obtained with a 60,000-foot road, input speeds of 42 and 52 mph, or a 60-40 directional distribution.

SUMMARY AND CONCLUSIONS

From the results in the two previous sections the following conclusions can be drawn:

1. When drivers are given knowledge of oncoming car speed on tangents, there appears to be an increase in safety, as shown by the reduction in the number of emergency indicators and increase in safety margin, but the average speed is reduced so that a significant loss in time occurs.
2. As the percentage of no-passing zones increases, there is a marked decrease in throughput as indicated by average speed, time delay, and number of passing maneuvers. The safety on the road, as determined by the emergency indicators, seems to increase slightly (even though the average safety margin oscillates).

These results apply only to the situation where no-passing zones can be removed by road reconstruction or remarking. However, the costs involved to reconstruct roads from, say, 70 percent no-passing to 50 or 25 percent no-passing, is high. Alternately, passing rules arising out of use of remedial aids, which provide drivers with information on oncoming car speed and distance or time headway, could be used to permit passing in no-passing zones. Such remedial aids may provide substantial benefits in throughput and safety at considerably less cost than road reconstruction.

Additional sets of simulations using different passing rules will be run to further investigate the possible use of no-passing zones for passing maneuvers.

The time required for a simulation is directly related to both the traffic volume, road length, and percentage of no-passing zones. At 100 vph using a 30,000-ft road and 0 percent no-passing zones, 1 min of computer time is required for 1 hr of simulation; but at 600 vph using 25 percent no-passing zones and a 60,000-ft road, 30 min of computer time was required for 1 hr of simulation.

ACKNOWLEDGMENTS

This paper is based on research conducted under Contract CPR-11-4193, "Development of Functional Specifications for Remedial Aids in Relation to Motor Vehicle Traffic Flow and Safety," sponsored by the Office of Research and Development, Bureau of Public Roads.

The authors are indebted to the many individuals, organizations and state highway departments who contributed information for use in this study. Particular thanks are due to Stanley R. Byington for his many helpful suggestions and contributions.

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Discussion

STANLEY R. BYINGTON, U. S. Bureau of Public Roads—To recognize the value of the value of the simulation model (SIMMOD) developed by Cassel and Janoff, one must consider that simulation is an intermediate step between mathematical analysis and experimental testing and normally an essential step in the design of a large-scale system. This consideration should include seeking answers to the following questions: (a) what large-scale system design is dependent on SIMMOD; (b) what mathematical analysis has been performed, what experimental testing is contemplated, and how does such analysis and testing relate to SIMMOD; and (c) what are the limitations of SIMMOD?

Following is an examination of the first two questions as they pertain to the Bureau of Public Roads Research and Development program to minimize rear-end and head-on accidents and to increase service volume on two-lane rural highways. Questions pertaining to other possible applications of SIMMOD are then raised, the answers to which are left to Cassel and Janoff. Such answers should give some indication as to the limitations of SIMMOD and where minor modifications can be made to the model to make it more applicable.

SIMMOD and Large-Scale System Design

Can SIMMOD be employed in the design of a large-scale system? This can best be answered by studying the role of SIMMOD in the development of a real world system; namely, a passing aid system called PAS. PAS is an electronic system which will inform motorists of oncoming vehicles beyond their line of sight and judgment capability, as along tangents, and will provide the motorists with information on the adequacy of available passing distance as derived from vehicle closing velocities and existing headways.

The development of PAS is part of an overall program to develop systems and/or procedures for aiding drivers in solving discrimination, judgment, information and vehicle control problems on 2-lane rural highways so as to raise their present level of service. The procedure being followed in developing the aforementioned systems and procedures, including PAS, consists of (a) studying the driver's task to uncover those limiting factors in driver performance (judgmental and operational) that are amenable to improvement; (b) identifying possible remedial aids and screening them through cost effectiveness analysis, experimental study of driver acceptance and use, and review of pertinent existing legal statutes and regulations; (c) developing functional specifications for the most promising remedial devices, procedures or systems; (d) designing and testing a "bread board" prototype of the functionally defined remediation devices; and (e) building and analyzing a field hardened version of the remediation devices.

The role of SIMMOD in developing PAS utilizing the aforementioned procedures is best studied by reviewing the history of PAS. In some cases, past studies and experience have already defined a limiting factor in driver performance (3). For example, the problem of driving on high-volume and/or winding 2-lane rural highways is well known by all those who drive on such roadways. Restricted sight distances, oncoming traffic and adverse environmental conditions make it difficult or impossible to pass slower moving vehicles and, as a result, motorists realize markedly increased travel times and inconvenience. Geometrically restricted opportunities to pass also encourage unsafe passing attempts (7) and may encourage unsafe following conditions, such as tailgating.

In the example, the limiting factor or problem is an insufficient number of passing zones for the frequency of passing maneuvers required by drivers to maintain desired speeds. Remediation for this problem can take two forms: removing the need for adequate passing sight distance or passing zones, and/or providing adequate passing sight distance to establish additional passing zones. Remediations of the first type consist of providing additional traffic lanes at selected locations, such as climbing lanes on grades, or along the entire length of an existing 2-lane rural road through conversion to a 4-lane facility. Another option, but far less desirable, is the exercising of control over the entry of traffic to a roadway and/or the speed at which vehicles are permitted to operate. Either control results in less maneuver freedom for drivers which is really the problem attempting to be solved. Remediation to provide adequate passing sight distance consists of realigning segments of a highway or providing the required sight distance electronically through a system such as PAS.

Screening of the proposed PAS remediation measure initially consisted of mathematically analyzing the costs and benefits of a PAS production model making certain assumptions as to how such a system would eventually be designed and how and where it would operate. Although estimates of costs and benefits used in the analysis were crude, they did serve to point out that implementation of PAS was not outside the realm of economic feasibility. This preliminary benefit-cost analysis, together with subsequent experimental study on possible system use by drivers and examination of the system's legal aspects, indicated that further development of PAS was warranted. Still, a more detailed economic evaluation of PAS was needed to answer questions like the following:

- Under what traffic volume conditions should PAS be operated?
- How do geometric conditions, such as intersections and ratio of passing to no-passing zone mileage, affect the possible benefits of PAS?
- How are PAS benefits affected by the accuracy with which information is transmitted to drivers? (A wider spacing of sensors to detect the presence, direction and velocity of vehicles will result in lower system cost but will reduce system benefits through the need for larger safety factors.)
- How does the percentage of drivers who use the system affect the economic feasibility of PAS?

All of these questions can be initially studied using SIMMOD. Eventually, however, a system like PAS must be experimentally analyzed on a real highway with respect to its reliability, accuracy, maintainability and economic feasibility. Thus, it is planned that PAS will be installed on 50 miles of rural highway and analyzed under normal driving conditions, proceeding from the highly controlled situation using test subjects to the standard uncontrolled situation with normal traffic. In the meantime, though, a simulation model like SIMMOD can be effectively employed in determining the potential usefulness of such a system and in the actual design of the system.

Limitations of SIMMOD

The preceding discussion certainly lends evidence as to the possible usefulness of SIMMOD, but what are its limitations? Answers to the questions below should, in part, answer the primary question just stated.

1. Can the model handle the overtaking maneuver?
2. How is passing performance measured?
3. What kinds of remedial-aid devices can be evaluated using SIMMOD?
4. How would slow-down factors be introduced into the model?
5. How is SIMMOD different from other (ITTE model and Shumate and Dirkson's model) 2-lane road simulation models?
6. Can the model handle both types of passes, accelerative and flying?
7. What constitutes a speed change within the model?
8. Of what value is the measure of number of accidents within SIMMOD? (Accidents are such a rare event that even lengthy runs of the model would produce no accidents.)

Treatment of at least some of the above questions by Cassel and Janoff, in their closing statement, should serve to benefit those who are interested in making use of SIMMOD, or a modification thereof, for their own purpose.

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F. G. LEHMAN, Newark College of Engineering—This paper represents a good application of the digital simulation technique to a specific type of highway traffic problem. Whenever a suitable model is found, simulation is an efficient tool for determining the effects of parameter changes such as the authors have done for number of no-passing zones, traffic volumes, etc.

The objectives of the study are well conceived and very clearly stated. The authors should be commended on the carrying out of their objectives in a very direct manner. It is apparent that the work has been done in close relation with people in highway practice.

The heart of this study is the model of the passing maneuver, knowledge of which must come from human factors data. In this work, a simple, logical model is based primarily on a set of probability functions for gap acceptance and changes in these functions with the passing driver's knowledge of oncoming car speed. Data for these functions have been supplied from a recent study referenced in the paper. Because of the importance of these data for the present study, the validity of this previous study is crucial. Some background information establishing this previous work as authoritative would increase confidence in the validity of the model.

Because simulation studies are often suspected to be academic exercises, it is necessary to build up a strong case for validity. This the authors have attempted to do by carefully analyzing their results in the light of experience, reason, and real data. A case in point where a more acceptable check is desired is the comparison with Normann's data in Figure 5. On the basis of curve shape, the comparison is good, but the difference in values between model data and real data is rather significant. The authors explain this difference by attributing it "to the fact that we used a 50-50 directional distribution while in Normann's data a $\frac{2}{3}$-$\frac{1}{3}$ is used." Why did not the authors program for a set of runs using the same directional distribution? However, only initial results have been presented. It is expected that other model tests against real data are being planned to further demonstrate the reliability of the simulation results.

Before applying the results of a simulation study, it is necessary to observe the caution that such results can show the feasibility of certain types of remedies but do not prove that they will be effective. The importance of the human factors in the situation should receive strong recognition.

ARNO CASSEL and MICHAEL S. JANOFF, Closure—The authors would like to thank Mr. Byington and Dr. Lehman for their remarks concerning this paper. However, a few additional comments are warranted based on the questions raised.

In reply to Byington's questions:

1. Any overtaking maneuver which is not converted immediately to a potential passing maneuver is instead converted to a following condition at some safe distance.

2. Passing performance is measured directly both by the number of attempted passes, actual passes, aborts and emergency indicators, and indirectly by average speed and

delay. An increase in passing performance would be indicated by an increase in both the number of passes and the average speed and a decrease in aborts, emergency indicators, and delay.

3. The model can test any type of remedial aid that would input a rule (or set of rules) by which a pass-no-pass decision could be made as in the following two examples:

Example 1: Pass if the oncoming gap is greater than or equal to a fixed number of seconds (fixed number of feet); no-pass if less than this value.

Example 2: Passing rules determined by the probability curves presently used in the model.

4. Slow down factors are presently being used for trucks on uphill grades and could be extended to other vehicles. A uniform deceleration is used on the uphill grades with a uniform acceleration on the downhill. A crawl speed of 20 ft/sec is presently being used in the model.

5. Our model differs from other simulations in that the main feature, the passing maneuver, is treated superficially in other models. We can employ various passing rules which simulate specific types of remedial aids and then measure the benefits; other models have no more than one passing rule.

6. The model presently converts all possible passing situations into accelerative passes but with minor logistical changes flying and accelerative passes could be treated distinctly by deleting the condition mentioned earlier that vehicles must be traveling at a speed less than desired speed to enter a potential passing situation.

7. A speed change cycle, as calculated in the model, is a change in operating speed from and back to a given speed. These occur mainly when slowing down due to congestion or when performing passing maneuvers.

8. The calculation of the number of projected accidents is not used in any economic analysis but is merely a check on the validity of the model. To date, less than 6 projected accidents have appeared during the simulation runs.

In reply to Lehman's questions:

The reliability of the original empirical data which were used to derive the probability curves presently being used has been discussed in the final report of our first contract. But, basically these data were the result of 2000 passing observations made on a 2-lane rural road in southern New Jersey. It would be desirable to repeat these tests in different geographical areas and under different road configurations as verification but this has not yet been accomplished. Also, the median of the accepted distances (i.e., the accepted oncoming gaps) was theoretically appropriate for the given oncoming car speed distribution encountered.

The second question of basing the differences on a difference in directional distributions has been partially answered by additional simulations.

After running the model at a 60-40 distribution and noticing little difference in output, further simulations at other input speeds were accomplished. At higher operating speeds, typical of lower traffic volumes, a decrease in the number of passes per vehicle was obtained, while at lower operating speeds, coinciding with greater traffic volumes, an increase in the number of passes per vehicle was obtained. These changes caused a better fit of the model data to Normann's data in Figure 5.

Evaluation of Delays and Accidents at Intersections to Warrant Construction of a Median Lane

ROBERT B. SHAW, Pennsylvania Department of Highways, and
HAROLD L. MICHAEL, Joint Highway Research Project, Purdue University

The objective of this study was to evaluate the conditions under which the construction, maintenance, and interest costs for a median lane would be warranted at suburban and rural approaches to an intersection. Delay times and accident rates to through vehicles caused by left-turning vehicles were analyzed in depth at three right-angle intersections which had median lanes, and at eight right-angle intersections which did not.

Seconds of delay per hour to through vehicles caused by left-turning vehicles were determined for the major approaches to the eleven intersections during daylight-weekday hours; 6 a.m. to 6 p.m., Monday through Friday. The accidents caused by left-turning vehicles were collected for almost a 5-yr period and analyzed to determine accident rates for each major intersection approach. This study found a substantial reduction in the number of accidents attributed to left-turning vehicles and negligible delay times to through vehicles at the intersection approaches which had median lanes. The accident rates and delay times were analyzed by a multiple linear regression analysis.

Although this study is based only on daylight-weekday hours, the findings are of considerable value in planning the construction of median lanes. The reduction in accidents and delays estimated for a period of years resulting from the construction of a median lane is used to determine if the construction, maintenance, and interest costs of the median lane at an intersection approach are justified.

•THE increase in motor vehicle use during recent years throughout the United States has greatly affected highway operation. This increase has created an added demand on all components of the highway system and has resulted in increased operating costs to the motoring public. Intersections are an important component of this system and the increased travel volumes have created congestion at many approaches in the urban, suburban, and rural areas. This study investigated one possible technique for congestion relief at suburban and rural intersection approaches.

Congestion at approaches to intersections is a cause for many of the critical problems in highway traffic operations and control (8). Where the intersection is at grade, streams of turning and crossing vehicles must join and cross each other. The points within the intersectional area used in common by these intersecting streams are focal

Paper sponsored by Committee on Operational Effects of Geometrics.

points of accidents and delay. Delays result when vehicles in different streams wish to pass through these focal points at or nearly at the same time. Accidents often result when drivers make mistakes in judgment of the time and place that such intersecting movements will occur.

The time and place of conflicts at approaches to intersections may be altered by traffic controls or design. Channelization of intersections at grade has been defined (6) as the separation or regulation of conflicting traffic movements into definite paths of travel by pavement markings, raised islands or other suitable means to facilitate the safe and orderly movement of both vehicles and pedestrians. Channelization is, therefore, used to control the place of conflict between intersecting traffic streams and to influence the time element by separating the conflict points and controlling the speeds at which these conflicts occur.

The median lane is one form of channelization used to separate the conflict points between left-turning vehicles and through vehicles. It provides a temporary, protected storage location for vehicles waiting to make a left-turn movement. This is a report on the results of a research project concerned with warrants for such median lanes, performed by the Joint Highway Research Project of Purdue University.

The objective was to evaluate the conditions for which the construction, maintenance, and interest costs of a median lane would be warranted at suburban and rural approaches to an interesection. Delay times and accident rates to through vehicles caused by left-turning vehicles were analyzed in depth at three right-angle intersections which had median lanes and at eight right-angle intersections which did not. By evaluating the benefits from the reductions in delay times and accident rates realized from the presence of a median lane, a method was developed which can be used to determine when construction of a median lane is economically justified.

STUDY LOCATIONS

The eleven intersections are located within a 60-mile radius of Lafayette-West Lafayette, Ind. (Fig. 1). These intersections are located on highways near Lafayette-West Lafayette, Kokomo, and Indianapolis. The approximate 1965 populations of these urban areas were 65,000, 50,000 and 500,000, respectively. Each intersection had the following characteristics: (a) signal or stop control, (b) four approaches, (c) intersection at right-angle, (d) restricted parking, and (e) suburban or rural location.

A large percentage of the traffic using these intersections was through traffic destined for Chicago, Indianapolis, Fort Wayne, or South Bend. The 1965 major street weekday ADT's for the intersections ranged from 7,100 to 27,500. A summary of the characteristics for the study intersections is given in Tables 1 and 2.

PROCEDURE

Delay Date

The delay time incurred by a through vehicle caused by a left-turning vehicle was determined at the eleven study intersections during daylight-weekday hours; 6 a.m. to 6 p.m., Monday through Friday.

The method developed to collect the delay time data was designed to be simple, inexpensive, and easily adaptable for use by one or more observers. A typical field setup of the equipment used to study the delay time is shown in Figure 2. The equipment used in the collection of delay data consisted of traffic volume counters, 20-pen recorder, 12-volt battery, push-button box, junction box, pneumatic tubes, and electrical conducting wire.

The placement of the traffic counters A and B varied in the suburban and rural areas. Counter A was located prior to the point at which an approaching through vehicle was influenced by the presence of the intersection. Counter B was located beyond the intersection at a point where the through vehicle had resumed its initial approach speed. As the approach speed increased, therefore, the distance between counters A and B increased; this distance was designated as the "zone of influence" and varied from about 800 to 1300 ft.

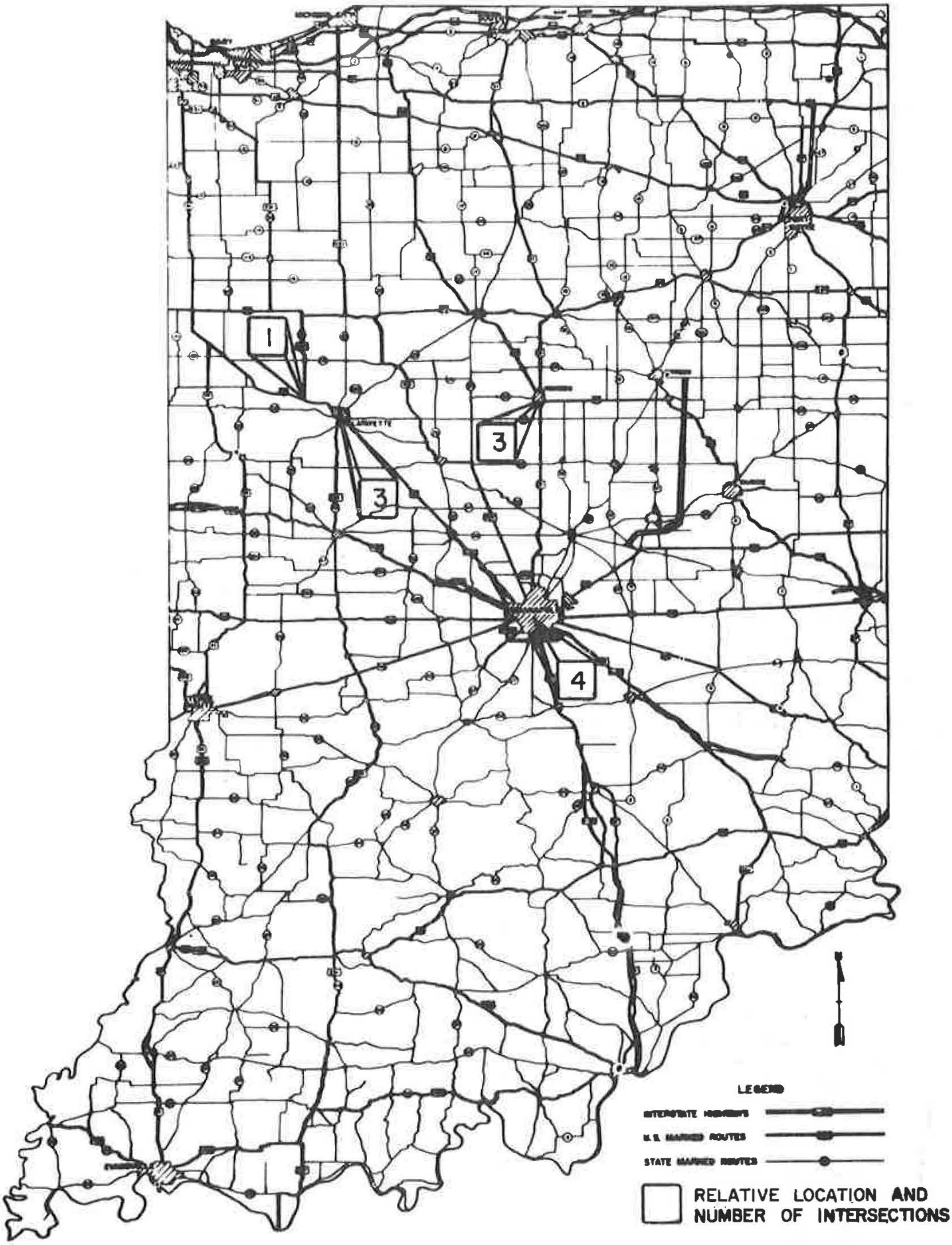

Figure 1. Relative locations of study intersections.

TABLE 1

SUMMARY CHARACTERISTICS OF STUDY INTERSECTIONS WITHOUT MEDIAN LANES

Intersection	Location	Type of Area	Type of Signalization	Weekday Approach[a] ADT Plus Weekday Opposing ADT
US 52 Bypass and Union St.	Lafayette	Suburban	Fixed time	17, 500
US 52 Bypass and SR 26	Lafayette	Suburban	Fixed time	18, 000
US 52 Bypass and Salisbury St.	Lafayette	Suburban	Semitraffic actuated	15, 800
US 52 and US 231	Lafayette	Rural	Stop sign controlled (flasher)	7, 100
SR 100 and 56th St.	Indianapolis	Rural	Fully traffic actuated	10, 500
SR 100 and Fall Creek Rd.	Indianapolis	Rural	Stop sign controlled (flasher)	7, 600
SR 100 and US 31	Indianapolis	Suburban	Fully traffic actuated	12, 900
US 35 (SR 22) and US 31 Bypass	Kokomo	Suburban	Fully traffic actuated	9, 500

[a]Weekday ADT's based on 1965 volume data.

TABLE 2

SUMMARY CHARACTERISTICS OF STUDY INTERSECTIONS WITH MEDIAN LANES

Intersection	Location	Type of Area	Type of Signalization	Weekday Approach[a] ADT Plus Weekday Opposing ADT
US 31 and US 35	Kokomo	Suburban	Fully traffic actuated	22, 000
US 31 and SR 26	Kokomo	Rural	Fully traffic actuated	15, 100
SR 100 and 30th St.	Indianapolis	Suburban	Fully traffic actuated	27, 500

[a]Weekday ADT's based on 1965 volume data.

Approach speed was the determining factor in indicating whether the intersection approach was considered to be located in a suburban or a rural area. Intersection approaches were classified as suburban when the approach speed was greater than 30 but less than 45 mph. Rural intersections were those locations where the approach

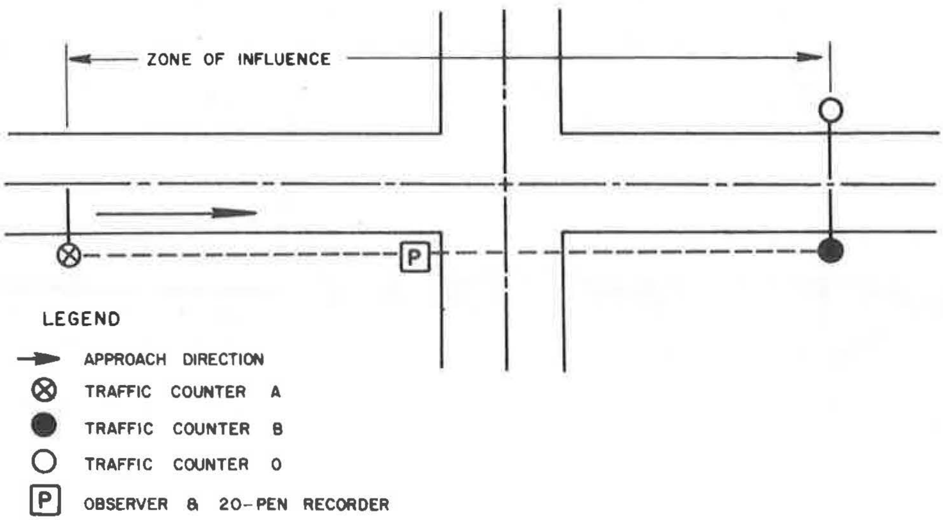

Figure 2. Typical field setup of equipment.

speed was greater than 45 mph. Much greater development of the adjacent land, of course, existed at the suburban intersections.

Traffic counters A and B were equipped with relay devices which actuated the 20-pen recorder whenever a vehicle axle crossed the pneumatic tubes connected to these two counters. Each axle actuation caused a pip on the recorder chart. An opposing traffic volume counter was located opposite counter B. Each observer had a push-button box which actuated six different pens of the 20-pen recorder, as follows:

Pen Number	Description
1	Cancel
2	Stopped time
3	Left-turn vehicular delay
4	Identification of study vehicle
5	Tube A
6	Tube B

Once the equipment was set up at the intersection, an observer selected the first approaching vehicle as a study vehicle. Each study vehicle was identified by pressing the identification button as the vehicle crossed tube A. If the study vehicle turned left or right before crossing tube B, the cancel button was pressed; if the vehicle was delayed by a left-turning vehicle at the intersection, the button signifying a left-turning vehicular delay was pressed; if the vehicle was stopped due to a traffic signal, the stopped time button was pressed both when the vehicle stopped and again when the vehicle started in motion; and finally, when the vehicle crossed tube B, the identification button was again pressed. When a study vehicle had been canceled or had passed through the zone of influence, the next succeeding vehicle to approach the intersection was selected as a study vehicle. This procedure was repeated for a 3-hr period on each approach studied at an intersection.

Additional notations were made on the recorder chart to indicate the classification of each study vehicle, and the number of stopped left-turning vehicles present in a queue. This number of stopped left-turning vehicles was later used to study adequate storage length for a possible median lane.

A study was conducted to verify whether or not the delay times incurred to through vehicles during the 3-hr study period were unique to that intersection approach for the particular time and day. The three suburban intersections in the Lafayette-West Lafayette area were selected for this purpose. Delay times for specific time periods and days of the week were measured on three successive weeks at the three intersections. It was found that the delay times for any particular time and day at a specified intersection approach were not significantly different at the 5 percent level of significance. As a result, it was concluded that adequate samples of delay time at an intersection approach could be obtained during any three consecutive hours for weekday-daylight hours.

The 20-pen recorder was operated at a rate of 6 in./min during the time each approach was studied. The elapsed time in seconds for a study vehicle to pass through the zone of influence was scaled from the recorder charts and recorded in one of the four following categories:

1. No delay;
2. Signal delay: (a) total time, (b) stopped time, (c) total time minus stopped time;
3. Left-turn vehicular delay; and
4. Left-turn vehicular delay and signal delay: (a) total time, (b) stopped time, (c) total time minus stopped time.

These data were used to determine averages of the hourly totals for each of the four categories and percentages of the vehicles delayed by a left-turning vehicle and of the vehicles delayed by a left-turning vehicle and a signal. Time differences were then determined for categories 1 and 3, and 2 and 4. The seconds of delay per hour caused by left-turning vehicles to the total volume of through vehicles per hour in the approach direction were calculated as follows:

$$Y_D = (V)(P_L)(T_L) + (V)(P_{LS})(T_{LS}) \tag{1}$$

where

Y_D = seconds of delay per hour caused by left-turning vehicles to the total volume of through vehicles per hour in the approach direction;
V = approach volume per hour of through traffic;
P_L = percent of through vehicles delayed by a left-turning vehicle;
T_L = difference, in sec, for the average hourly times of categories 1 and 3;
P_{LS} = percent of through vehicles delayed by a left-turning vehicle and a signal; and
T_{LS} = difference, in sec, for the average hourly times of categories 2 and 4.

An early conclusion from the field data was that the delay time experienced by a through vehicle was negligible at the three locations which had median lanes on the approaches to the intersection. Further analysis, therefore, was limited to the delay time experienced by a through vehicle at the approaches to the eight intersections which did not have median lanes.

Accident Data

An almost 5-yr study period was chosen in order that an adequate sample of accidents could be obtained. Accidents were collected for the daylight-weekday hours at the eleven study intersections for the period Jan. 1, 1961 through Aug. 31, 1965, and accident rates were calculated (Tables 3 and 4).

Data on accidents for the three intersections with median lanes clearly indicated the almost total absence of accidents caused by left-turning vehicles. As a result, it was concluded that a median lane substantially reduces accidents involving left-turning vehicles.

The accident analysis was limited to those accidents caused by left-turning vehicles which could have been prevented with the installation of a median lane. The types of accidents considered preventable were the following:

1. Accidents involving a left-turning vehicle with opposing traffic,
2. Sideswipe overtaking accidents involving a left-turning vehicle, and
3. Rear-end accidents that probably resulted from a left-turn movement.

The accident data were collected from the Accident Records Division of the Indiana State Police and local police records. Indiana state law requires that all accidents involving a personal injury, death or property damage of $50 or more be reported to the police.

In most instances the collision diagram and description of the accident from the investigating officer report form provided the necessary information to distinguish a preventable accident from a nonpreventable accident. It was concluded, however, that additional accidents probably were attributed to left-turning vehicles. A study was conducted, therefore, to determine additional rear-end collisions caused by left-turning vehicles which were not recorded as such on the investigating officer report forms. Accident rates for the other rear-end collisions were calculated for the eight intersections without median lanes and for the three intersections with median lanes (Tables 3 and 4). The difference in the averages of these two accident rates was then used as a basis to randomly assign additional rear-end accidents which could be considered preventable with the installation of a median lane.

TABLE 3

ACCIDENT RATES AT STUDY INTERSECTIONS WITHOUT MEDIAN LANES[a]

Intersection	Cause and Type of Accident[b]			
	Left-Turn		Other	
	Rear-End	Right-of-Way	Rear-End	Right-of-Way
US 52 Bypass and Union St.	0.151	0.490	0.151	0.075
US 52 Bypass and SR 26	0.183	0.366	0.440	0.073
US 52 Bypass and Salisbury St.		0.167	0.417	
US 52 and US 231 (SR 53)	0.186	0.279		0.466
SR 100 and 56th St.		0.126	0.315	0.126
SR 100 and Fall Creek Rd.	0.437	0.262		0.699
SR 100 and US 31	0.360	0.514		0.051
US 35 and US 31 Bypass	0.075	1.196	0.149	0.299
Average	0.278	0.604	0.361	0.405

[a]Accident rates are expressed as the number of accidents per million vehicles for the period Jan. 1, 1961 through Aug. 31, 1965.
[b]Accidents are classified according to cause: left-turn vehicle or other; and according to type: rear-end or right-of-way.

The accident data were analyzed on a yearly basis at each intersection approach to determine an accident rate, number of accidents per million vehicles caused by left-turning vehicles, at each of the eight intersections. No accidents involving a fatal injury were included because of the rarity of such accidents and the difficulty of establishing an economic loss.

Volume

In delay and accident studies, volume has correlated well with delay times and accident rates. This volume can be represented as an hourly volume or as the annual average weekday traffic (ADT). Both the hourly volumes and the weekday ADT were used in the analysis.

The traffic volume counters, used as part of the equipment to measure delay time, were employed simultaneously to obtain the approach and opposing volumes per hour for a given direction of travel. An observer was used to record the number of left-turning and right-turning vehicles, as well as the classification of vehicles entering the intersection approach. It was, therefore, possible to analyze volumes, turning movements, and commercial vehicles for the same period of time the delay data were collected.

The approach and opposing hourly volumes at the time the accident occurred and the weekday ADT's were correlated with the accident rate. Because volume counts were not available for the entire study period, these hourly volumes were estimated.

The traffic volumes obtained at the time the delay data were collected were supplemented by volume data from the Division of Planning, Indiana State Highway Commission. Factors were determined from the volume data collected, from records of the Commission, and from charts depicting the yearly, monthly, daily, and hourly variations in traffic volume during average conditions (12). Therefore, by knowing the location, year, month, day, and hour of an accident, the hourly volumes at the time an accident occurred were estimated by applying the appropriate factors to the volume counts taken at each intersection approach.

TABLE 4

ACCIDENT RATES AT STUDY INTERSECTIONS WITH MEDIAN LANES

Intersection	Type of Accident	
	Rear-End	Right-of-Way
US 31 and US 35	0.301	0.422
US 31 and SR 26	0.220	0.396
SR 100 and 30th St.	0.177	0.133
Average	0.240	0.354

Capacity

The practical capacity of each intersection was calculated by the method described in the Highway Capacity Manual (7).

Six of the signalized intersections had paved shoulders on the right side which allowed through vehicles to maneuver around a left-turning vehicle. These paved shoulders also acted as turning lanes but were not designated for this specific movement. To determine the effectiveness of the paved shoulders in increasing the practical capacities of these six intersections, reference was made to a study (9) which indicated that each paved shoulder carried approximately one-third the capacity of a properly constructed and signed turning lane.

The practical capacity was calculated for an extra turning lane if more than one lane existed for a direction of travel. This lane was assumed to be a left-turn only lane if the predominant turning movement at that approach was left, and assumed to be a right-turn only lane if the predominant turning movement at that approach was right. If the additional lane was only a paved shoulder not constructed, signed, or used exclusively as turning lane, only one-third of the turning lane capacity was added to the through lane capacity.

The two stop-controlled intersections were also protected with flashers. Although no precise method was available to evaluate the practical capacity of these two unsignalized intersections, it was assumed that the crossroad traffic interference caused a wave-like behavior to the through traffic which approached the behavior of traffic under signal control (1). As the crossroad traffic interference did not result in interrupted flow, the practical capacities of these intersections were computed as if the intersections had been operated under traffic control signals with a green time to cycle length ratio of one.

ANALYSIS OF DATA

Multiple Linear Regression

Many variables possibly affecting the delay and accident data were analyzed by multiple linear regression. This method provided expressions for predicting the seconds of delay per hour caused by left-turning vehicles to the volume of through vehicles per hour, and the number of accidents per million vehicles caused by left-turning vehicles at approaches to intersections in both the rural and suburban areas. The computer program used in this study for the multiple linear regression analysis was the BIMD-2R, Stepwise Regression (10).

Tests were conducted on the resulting delay time and accident rate prediction equations to determine whether each independent variable in each equation was significant. The purpose was to develop simplified equations which would in most instances adequately predict delay times and accident rates for both suburban and rural intersections by using a fewer number of independent variables. An option in the BIMD-2R program provided for a summary table listing the order each independent variable entered the multiple linear regression equation and the corresponding increase in the multiple coefficient of determination (R^2) associated with each new variable. The F-test (3) was used to determine the first independent variable which did not add significantly to the increase in the multiple R^2, given the other independent variable or variables already in the regression equation. For example, tests were conducted at a 5 percent level of significance to determine whether a significant increase resulted from the addition of a second independent variable given the first independent variable, or from the addition of a third independent variable given the first two independent variables already in the regression equation. The results of these tests are the basis for the formulation of simplified predictions equations for delay time and accident rates.

Delay Time

The variables in Table 5 represent the independent variables which were considered in the initial analysis for predicting the variability in delay times for both suburban and rural areas. The results from this initial regression analysis were examined for significance and duplication, and certain variables deleted. The final delay time prediction equations were based on the remaining independent variables.

TABLE 5

INDEPENDENT VARIABLES—SUBURBAN AND RURAL DELAY TIMES

Number	Variable
3	Type of area—suburban or rural
4	Flasher (stop) controlled
5	Fixed-time controlled signalization
6	Semitraffic-actuated controlled signalization
7	Fully traffic-actuated controlled signalization
8	Green time to cycle length ratio of through approach
9	Green time to cycle length ratio of left-turn phase
10	Grade of approach, %
11	Number of approach lanes
12	Width of approach roadway at the intersection, ft
13	Average speed through intersection for a nondelayed through vehicle, ft/sec
14	Ratio of width of access points to zone of influence length
15	Approach volume per hour, vph
16	Opposing volume per hour, vph
17	Number of left-turning vehicles in approach direction per hour
18	Number of right-turning vehicles in approach direction per hour
19	Number of commercial vehicles in approach direction per hour
20	Number of approaching through vehicles per hour delayed by a left-turning vehicle only
21	Number of approaching through vehicles per hour delayed by a left-turning vehicle and a signal
22	Ratio of approach volume per hour to capacity of approach direction
23	Ratio of opposing volume per hour to capacity of opposing direction
24	Average number of stopped left-turning vehicles in an approach queue per hour
25	Total volume per hour in approach and opposing directions, vph

Suburban Area—The prediction equation explaining the greatest amount of variability in suburban delay time (Y_{DS}) is the following:

$$Y_{DS} = 483.788 - 726.881\,X_8 - 33.292\,X_{10} - 338.278\,X_{11} - 4.157\,X_{13} + 4.347\,X_{17} - 3.635\,X_{19} - 1027.246\,X_{22} + 1.984\,X_{26} \quad (2)$$

The multiple correlation coefficient is 0.828. The variables explain approximately 69 percent (R^2) of the variation in the seconds of delay per hour caused by left-turning vehicles to the total volume of through vehicles per hour for a suburban intersection approach.

The most significant variable for suburban delay time is the total volume per hour in the approach and opposing direction (X_{26}). Other important variables are the green time to cycle length ratio for the through approach (X_8), the percent grade of the approach (X_{10}), the number of approach lanes (X_{11}), the average speed through the intersection for a nondelayed through vehicle (X_{13}), the number of left-turning vehicles per hour in the approach direction (X_{17}), the number of commercial vehicles per hour in the approach direction (X_{19}), and the ratio of the approach volume per hour to the capacity of the intersection approach (X_{22}).

The simplified prediction equation for suburban delay time is as follows:

$$Y_{DS} = -620.838 + 3.505\,X_{17} + 0.886\,X_{26} \quad (3)$$

The multiple correlation coefficient is 0.791. The variables explain approximately 63 percent (R^2) of the variation in the seconds of delay per hour caused by left-turning vehicles to the total volume of through vehicles per hour for a suburban intersection approach.

The most significant variable is the total volume per hour in the approach and opposing directions (X_{26}). The other independent variable is the number of left-turning vehicles per hour in the approach direction (X_{17}).

Rural Area—The prediction equation explaining the greatest amount of variability in rural delay time (Y_{DR}) is the following:

$$Y_{DR} = -\ 44.469 + 50.673\ X_{10} - 13.514\ X_{12} + 1.003\ X_{15} + 5.017\ X_{17} - 2.735\ X_{19} + 547.598\ X_{22} + 0.731\ X_{26} \quad (4)$$

The multiple correlation coefficient equals 0.986. The variables explain approximately 97 percent (R^2) of the variation in the seconds of delay per hour caused by left-turning vehicles to the total volume of through vehicles per hour for a rural intersection approach.

The most significant variable for rural delay time is the total volume per hour in the approach and opposing directions (X_{26}). Other important variables are the percent grade of the approach (X_{10}), the width of the approach roadway at the intersection (X_{12}), the approach volume per hour (X_{15}), the number of left-turning vehicles per hour in the approach direction (X_{17}), the number of commerical vehicles per hour in the approach direction (X_{19}), and the ratio of the approach volume per hour to the capacity of the intersection approach (X_{22}).

The simplified prediction equation for rural delay time is as follows:

$$Y_{DR} = -\ 242.880 - 9.119\ X_{19} + 1.669\ X_{26} \quad (5)$$

The multiple correlation coefficient is 0.958. The variables explain approximately 92 percent (R^2) of the variation in the seconds of delay per hour caused by left-turning vehicles to the total volume of through vehicles per hour for a rural intersection approach.

The most significant variable is the total volume per hour in the approach and opposing directions (X_{26}). The other independent variable is the number of commercial vehicles per hour in the approach direction (X_{19}).

During the collection of delay data, notations were made on the recorder chart indicating the number of stopped left-turning vehicles in each queue. It was possible, therefore, to determine an average number of stopped left-turning vehicles in a queue per hour. This average number could then be used to determine the adequate storage length for a proposed median lane.

The required length of the proposed median lane will vary at each intersection approach. The following factors, however, should be considered when determining the length of the proposed storage lane: (a) approach volume, (b) percent left-turning vehicles, (c) average approach speed, and (d) average number of stopped left-turn vehicles in a queue per hour.

Accident Rate

The variables in Table 6 represent the independent variables which were considered in the initial analysis for predicting the variability in accident rates for both suburban and rural areas. The results from this initial regression analysis were examined for significance and duplication, and certain variables deleted. The final accident rate prediction equations were based on the remaining independent variables.

Suburban Area—The prediction equation explaining the greatest amount of variability in the suburban accident rate (Y_{AS}) is the following:

$$Y_{AS} = 1.2411 - 1.0882\ X_7 + 0.0029\ X_{10} + 1.3094\ X_{12} - 0.8496\ X_{13} + 0.0824\ X_{14} - 1.6262\ X_{16} + 0.0443\ X_{17} \quad (6)$$

TABLE 6

INDEPENDENT VARIABLES—SUBURBAN AND RURAL ACCIDENT RATES

Number	Variable
2	Type of area, suburban or rural
3	Flasher (stop) controlled
4	Fixed-time controlled signalization
5	Semitraffic-actuated controlled signalization
6	Fully traffic-actuated controlled signalization
7	Number of approach lanes
8	Width of approach roadway at the intersection, ft
9	Width of opposing roadway at the intersection, ft
10	Approach volume per hour at time the accident occurred, vph
11	Opposing volume per hour at time the accident occurred, vph
12	Weekday approach, ADT, vpd
13	Weekday approach ADT plus weekday opposing ADT, vpd
14	Total intersection weekday ADT, vpd
15	Ratio of approach volume per hour to capacity of approach direction
16	Ratio of opposing volume per hour to capacity of opposing direction
17	Average speed through the intersection for a nondelayed through vehicle, ft/sec

The multiple correlation coefficient is 0.781. The variables explain approximately 61 percent (R^2) of the variation in the number of accidents per million vehicles caused by left-turning vehicles on a suburban intersection approach.

The most significant variable for suburban accident rate is the weekday approach ADT plus the weekday opposing ADT (X_{13}). Other important variables are the number of approach lanes (X_7), the approach volume per hour at the time the accident occurred (X_{10}), the weekday approach ADT (X_{12}), the total intersection weekday ADT (X_{14}), the ratio of the opposing volume per hour to the capacity of the opposing intersection approach (X_{16}), and the average speed through the intersection for a nondelayed through vehicle (X_{17}).

The simplified prediction equation for the suburban accident rate is as follows:

$$Y_{AS} = 3.6203 - 1.1407\, X_7 + 1.2446\, X_{12} - 0.7723\, X_{13} + 0.0371\, X_{14} \tag{7}$$

The multiple correlation coefficient is 0.743. The variables explain approximately 55 percent (R^2) of the variation in the number of accidents per million vehicles caused by left-turning vehicles on a suburban intersection approach.

The most significant variable in this simplified prediction equation is the weekday approach ADT plus the weekday opposing ADT (X_{13}). Other independent variables are the number of approach lanes (X_7), the weekday approach ADT (X_{12}), and the total intersection ADT (X_{14}).

Rural Area—The prediction equation explaining the greatest amount of variability in the rural accident rate (Y_{AR}) is the following:

$$Y_{AR} = 0.6411 - 0.2848\, X_7 - 0.0110\, X_8 + 0.0045\, X_{10} - 0.0077\, X_{11} + 0.8690\, X_{13} - 0.6018\, X_{14} - 2.9019\, X_{15} + 6.0704\, X_{16} \tag{8}$$

The multiple correlation coefficient is 0.825. The variables explain approximately 68 percent (R^2) of the variation in the number of accidents per million vehicles caused by left-turning vehicles on a rural intersection approach.

The most significant variable for rural accident rate is the total intersection weekday ADT (X_{14}). Other important variables are the number of approach lanes (X_7), the width of the approach roadway at the intersection (X_8), the approach volume per hour at the time the accident occurred (X_{10}), the opposing volume per hour at the time the accident occurred (X_{11}), the weekday approach ADT plus the weekday opposing ADT (X_{13}), the ratio of the approach volume per hour to the capacity of the approach direction (X_{15}), and the ratio of the opposing volume per hour to the capacity of the opposing direction (X_{16}).

The simplified prediction equation for the rural accident rate is as follows:

$$Y_{AR} = 1.1333 + 0.0015 X_{10} - 0.0497 X_{14} \tag{9}$$

The multiple correlation coefficient is 0.609. The variables explain approximately 37 percent (R^2) of the variation in the number of accidents per million vehicles caused by left-turning vehicles on a rural intersection approach.

The most significant variable for rural accident rate is the total intersection weekday ADT (X_{14}). The other independent variable is the approach volume per hour at the time the accident occurred (X_{10}). This simplified equation, however, does not adequately predict the accident rate at a rural intersection approach as indicated by the low multiple correlation coefficient. As a result the full prediction equation should be used.

APPLICATION OF PREDICTION EQUATIONS

General

The development of prediction equations for estimating the delay time and accident rate due to the absence of a median lane at rural and suburban intersections permits the evaluation of benefits to be expected from construction of such a lane. Application of these equations to evaluation is a simple process.

The application is limited to two extreme conditions under which median lanes might be proposed. It is assumed that a median lane is warranted when the costs of its construction are equal to or less than the economic benefits derived. Benefits are realized by reduced delays to through vehicles and a reduction in the number of accidents attributed to left-turning vehicles. The simplified prediction equations are used to determine such reductions in delay time and accident rates.

The first example considers the case where adequate right-of-way exists on both approaches of a two-lane highway to a signalized intersection in a suburban area. The existing pavement on one or both sides of the highway must be widened for a specified distance on both approaches so that median lanes can be constructed and new through lanes designated.

The second example considers the case where a median strip at least 16 ft wide is located between the major approaches to a signalized intersection of a four-lane divided highway in a suburban area. The left-turn lanes will be constructed within the existing median and no changes to the existing lanes are required.

The basic specifications and construction costs for median lanes were obtained from the Indiana State Highway Commission, Division of Traffic. Several contracts of intersection channelization projects were examined to obtain representative 1965 costs.

The actual cost of delay time was determined for the southbound approach to the intersection of US 52 bypass and SR 26 in Lafayette. The cost of delay for the average vehicle type was calculated to be $2.25 per hour of delay. This cost estimate includes time and fuel costs for deceleration, acceleration, and idling, and a time cost for comfort and convenience. The unit costs and rates used in the determination of the hourly estimate for delay costs are given in Table 7.

Average costs for an accident caused by a left-turning vehicle were determined from the accident report forms collected for the period Jan. 1, 1961 through Aug. 31, 1965. The average cost of each injury in 1965 was set at $1900 (11). The average accident costs, which included both property damage and injury costs, were calculated to be $710 in suburban areas and $1352 in rural areas.

A 6 percent interest rate was used to obtain the annual costs for construction and maintenance of the median lane based on 1965 unit costs.

TABLE 7

1965 UNIT COSTS AND RATES USED TO CALCULATE THE HOURLY DELAY COST[a]

Item	Passenger Vehicles	Commercial Vehicles
1. Fuel, \$/gal	0.32	0.28
2. Idling, gal/min	0.007	0.011
3. Time, \$/hr	1.55	2.80
4. Comfort and convenience, \$/veh-mi	0.01	0.01

[a]These unit costs and rates are average values (3, 8, and 9).

The prediction equation used to estimate the seconds of delay per hour and the number of accidents per million vehicles to through vehicles caused by left-turning vehicles are based on weekday-daylight hours. These predicted delay times and accident rates, therefore, include only 12 hours per day for 260 days of the year. For a second calculation, it was assumed that the delay times and accident rates for the weekend-daylight hours are the same or greater than the delay times and accident rates for the weekday-daylight hours. With this assumption, computations are based on the 12 hours per day for 365 days of the year. In the two examples to follow, annual cost estimates for delay times and accident rates are based on both 260 days and 365 days per year.

It is also assumed that all delays to through vehicles from the left-turn movement and all accidents involving left-turn vehicles will be eliminated by the construction of a median lane. Although this is not completely accurate, it is substantially correct. Furthermore, the prediction equations, by not considering the night hours, give conservative values for both delay and accidents.

Cost estimates for the installation of a median lane are based on construction costs at an existing intersection approach with no additional improvements at that intersection approach. Lower costs would result when additional improvements to an existing intersection are to be made in conjunction with the median lane or when a median lane is to be installed on the intersection approach of a completely new highway.

The two examples on the following pages may not be the best possible solutions to the chosen intersection approaches, and are only illustrative examples for the application of the simplified prediction equations.

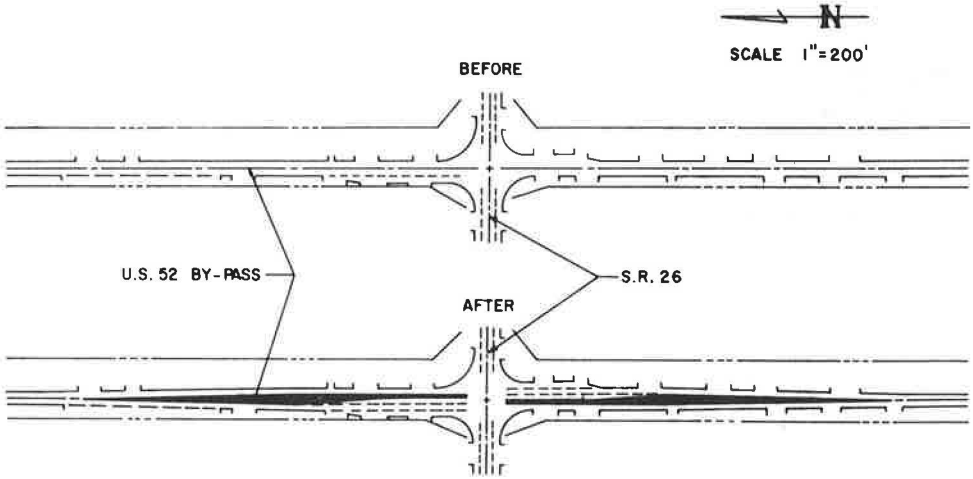

Figure 3. Conditions before and after construction of median lanes at US 52 bypass and SR 26.

Example 1

This example attempts to justify the construction of median lanes on both approaches to the intersection of US 52 bypass and SR 26. The bypass is a two-lane highway in a suburban area with adequate right-of-way for median lane construction on both approaches to the intersection. The conditions before and after construction of the median lanes are shown in Figure 3.

The annual construction, maintenance, and interest costs were determined based on 1965 unit construction costs. No attempt was made to improve the type of signalization nor to include any cost estimate for such improvement.

The number of daylight hours of delay per year attributed to left-turning vehicles was determined from Eq. 3, developed for suburban areas.

Variable	Northbound	Southbound
X_{17}	80	32
X_{26}	1107	1107

An annual increase in traffic of 3 percent was assumed to evaluate variables X_{17} and X_{26} for the succeeding 5 and 10-yr periods.

The number of accidents per year caused by left-turning vehicles during the daylight hours was determined from Eq. 7, developed for suburban areas.

Variable	Northbound	Southbound
X_7	1	1
X_{12}	8.80	9.20
X_{13}	18.0	18.0
X_{14}	26.3	26.3

TABLE 8

SUMMARY COST ESTIMATES FOR EXAMPLE 1

Description	Costs	Annual Cost in Dollars			
		1965-1969		1965-1974	
		260 (days/yr)	365 (days/yr)	260 (days/yr)	365 (days/yr)
Median lanes:					
Preparation	1, 462				
Construction	20, 822				
Finishing	100				
Signs and maintaining traffic	3, 000				
Total cost	25, 984				
Maintenance and miscellaneous (15.0%)	3, 898				
Total cost	29, 882				
Annual cost at 6.0% interest rate (C + M + I)		6, 078	6, 078	4, 061	4, 061
Cost reduction estimates:					
Delay time (C_{DS})		2, 450	3, 439	2, 838	3, 984
Accidents (C_{AS})		2, 284	3, 206	1, 894	2, 659
Total reduction cost (C_{DS} + C_{AS})		4, 734	6, 645	4, 732	6, 643
Difference [(C_{DS} + C_{AS}) - (C + M + I)]		-1, 344[a]	+ 567[b]	+ 671	+2, 582

[a]A negative difference indicates that the annual cost to install median lanes cannot be justified by the annual savings in delay and accidents to through vehicles.

[b]A positive difference indicates that the annual cost to install median lanes can be justified by the annual savings in delay and accidents to through vehicles.

An annual increase in traffic of 3 percent was also assumed to evaluate variables X_{12}, X_{13}, and X_{14} for the succeeding 5 and 10-yr periods.

A summary of the annual cost estimates determined for median lane construction and the resulting reduction in delay time and number of accidents is given in Table 8. The results indicate that the construction, maintenance, and interest costs for median lanes on both approaches to the intersection of US 52 bypass and SR 26 can be justified over a 5-yr period using 365 days per year.

Example 2

This example attempts to justify the construction of a median lane on the northbound approach to the intersection of US 31 bypass and Lincoln Road. The US 31 bypass is a four-lane divided highway in a suburban area with an existing median 40 ft wide. The southbound approach to the intersection already has a left-turn lane. The conditions before and after construction of the median lane are shown in Figure 4.

The annual construction, maintenance, and interest costs were again determined based on 1965 unit construction costs. No attempt was made to improve the type of signalization nor to include any cost estimate for such improvement.

The number of daylight hours of delay per year attributed to left-turning vehicles was determined from Eq. 3 for suburban areas.

Variable	Northbound
X_{17}	7
X_{26}	890

An annual increase in traffic of 3 percent was assumed to evaluate variables X_{17} and X_{26} for the succeeding 5 and 10-yr periods.

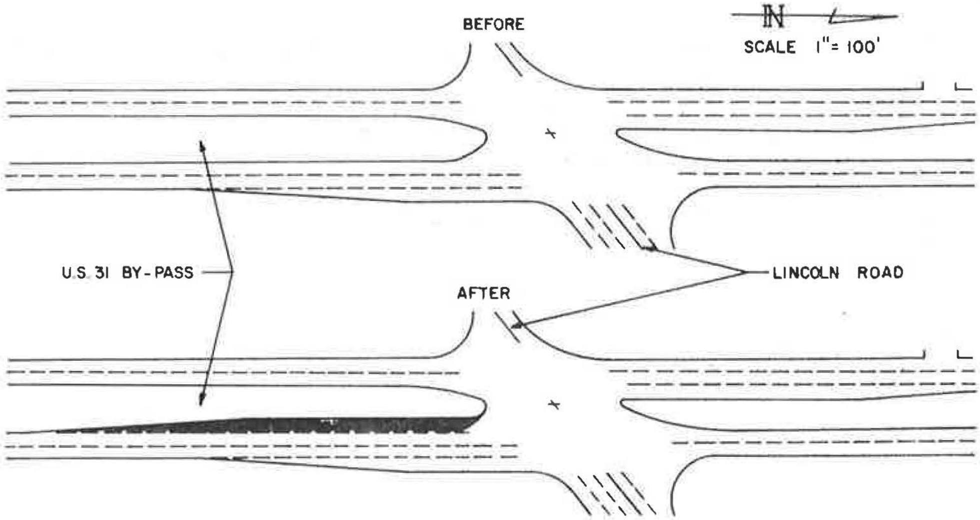

Figure 4. Conditions before and after construction of a median lane at US 31 bypass and Lincoln Road.

TABLE 9

SUMMARY COST ESTIMATES FOR EXAMPLE 2

Description	Costs	Annual Cost in Dollars			
		1965-1969		1965-1974	
		260 (days/yr)	365 (days/yr)	260 (days/yr)	365 (days/yr)
Median lane:					
Preparation	40				
Construction	3, 521				
Finishing	200				
Signs and maintaining traffic	1, 000				
Total cost	4, 761				
Maintenance and miscellaneous (15.0%)	714				
Total cost	5, 475				
Annual cost at 6.0% interest rate (C + M + I)		1, 114	1, 114	744	744
Cost reduction estimates:					
Delay time (C_{DS})		473	664	607	852
Accidents (C_{AS})		814	1, 427	717	1, 007
Total reduction cost ($C_{DS} + C_{AS}$)		1, 287	2, 091	1, 324	1, 859
Difference [($C_{DS} + C_{AS}$) - (C + M + I)]		+ 173[a]	+ 977	+ 580	+1, 115

[a]A positive difference indicates that the annual cost to install a median lane can be justified by the annual savings in delay and accidents to through vehicles.

The number of accidents per year caused by left-turning vehicles during the daylight hours was determined from Eq. 7 for suburban areas.

Variable	Northbound
X_7	2
X_{12}	9.5
X_{13}	17.4
X_{14}	20.6

An annual increase in traffic of 3 percent was also assumed to evaluate variables X_{12}, X_{13}, and X_{14} for the succeeding 5 and 10-yr periods.

A summary of the annual cost estimates determined for median lane construction and the resulting reduction in delay time and number of accidents is given in Table 9. The results indicate that the construction, maintenance, and interest costs for the median lane on the northbound approach to the intersections of US 31 bypass and Lincoln Road could be justified over both the 5 and the 10-yr periods using either 260 weekdays or 365 days per year.

RESULTS AND FINDINGS

The results and findings of this study are summarized in the following paragraphs.

1. The presence of a median lane substantially reduces the number of accidents and eliminates delay time to through vehicles resulting from left-turning vehicles.

2. A warrant for the construction of a median lane which relates the annual cost for construction and maintenance of a median lane to the total estimated benefits de-

rived from reductions in delay and in accidents for suburban and rural areas is as follows:

$$C_{DS} + C_{AS} \geq C + M + I \tag{10}$$

$$C_{DR} + C_{AR} \geq C + M + I \tag{11}$$

where

C_{DS} and C_{DR} = annual cost reduction estimates for delay time in the suburban and rural areas, respectively,

C_{AS} and C_{AR} = annual cost reduction estimates for accidents in the suburban and rural areas, respectively, and

$C + M + I$ = annual construction, maintenance, and interest costs for the median lane.

3. Equations were developed to predict delay times and accident rates for the weekday-daylight hours for through traffic at suburban and rural intersections that resulted from left-turning vehicles and the absence of median lanes.

4. Using a life of only five years, it was shown that median lanes were warranted at two example intersections. The benefits were found to be such that, when compared with the cost of a median lane, almost every intersection on a divided highway with a median of 16 ft or more and many intersections on other four and two-lane highways possess the warrants for construction of median lanes.

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Evaluation of Minor Improvements

THOMAS N. TAMBURRI, CHARLES J. HAMMER, JR., JOHN C. GLENNON, and ALAN LEW, California Division of Highways

These investigations evaluated the current effectiveness of: (a) center-suspended and advance warning flashing beacons in reducing accidents, (b) safety lighting installations in reducing night accidents, (c) various delineation devices, and (d) protective guardrail in reducing reported accidents. A before-and-after study method was used to evaluate 45 flashing beacons, 41 safety lighting projects, 32 delineation locations, and 14 guardrail locations. In addition, the current warrants for intersection flashing beacons and for safety lighting were compared with other possible warrants to determine if more effective criteria could be established. Two methods of predicting future accidents are also reviewed.

SUMMARY AND RECOMMENDED WARRANTS

Flashing Beacons

The study was conducted to evaluate the current effectiveness of flashing beacon installations in reducing accidents and to determine if more objective criteria could be established for flashing beacons.

The before-and-after study method was used to evaluate 52 flashing beacon projects. Of these, only 45 projects had sufficient information for a detailed comparative analysis. Approximately 75 percent of the projects had a reduction in accident rates although not all were statistically significant (see Fig. 1). It was concluded that percent reduction of accident rates alone is an unreliable indicator of the success of the improvement.

Flashing beacons as a whole have been quite effective in reducing accidents (34 percent reduction), with an 83 percent reduction at railroad crossings, 40 percent reduction at intersections and 21 percent reduction at advance warning beacon installations.

All of the projects evaluated in this study were sumarized (see Table 2). Presuming a 20-yr project life, the cost per accident reduced by flashing beacons averages \$38 and ranged from \$27 for beacons at intersections to \$56 for advance warning beacons to \$328 for No. 8 flashing beacons at railroad crossings.

Railroad flasher installations are costly; thus the cost per accident reduced is high for a flashing beacon installation. The accidents reduced by this treatment, however, are of a severe nature (car-train) and the cost may be of secondary consideration.

Seven trial accident warrants for red-yellow flashers at 4-leg intersections were compared with the present warrants. Present warrants allow flashing beacon installations where sight distance is extremely limited or where other conditions make it especially desirable to emphasize the need for stopping on one street and for proceeding with caution on the other; or when there has been a preponderance of broadside or crossing accidents. Four trial accident warrants for all red flashers at 4-leg intersections were compared with current practice. An analysis of the effect of these

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eleven warrants on total accident reduction suggests that the following warrants be adopted:

1. Flashing beacons should be considered at 4-leg intersection locations which have stop sign control and which experience four or more crossing (broadside) plus left-turn accidents in one year or six or more crossing plus left-turn accidents in two years.

If the above criterion has been met, the type of control at the intersection can be determined from the following: (a) if the minor to major entering volume ratio is 0.50 or less, red-yellow lens operation (2-way stop) should be considered; (b) if the minor to major entering volume ratio is greater than 0.50, 4-way red lens operation (4-way stop) should be considered.

2. Where stop signs are warranted, flashing beacons should be considered also (a) where approach speeds are high; (b) where visibility to stop sign is limited; or (c) where the intersection is hidden or unexpected.

Four-leg red-yellow intersection flashers (15 projects) had an accident rate reduction of 31 percent from 2.29 to 1.59 with the nine projects meeting the recommended warrants having a 51 percent accident rate reduction from 2.29 to 1.13. Four-leg red intersection flashers (eight projects) had a 68 percent reduction in accident rate from 2.77 to 0.88 with six of these projects meeting the recommended warrants having a 77 percent reduction in accident rate from 3.48 to 0.80.

Four-leg intersection data indicate that greater accident reductions can be expected by using a 12-in. lens (rather than 8-in.). The additional cost is small ($20 to $25 per flasher or approximately $100 per 4-leg intersection). Consequently, a small amount of additional money can effect a greater accident reduction. The numbers of projects were small and statistically inconclusive for 3-leg intersections. Indications were that the 8-in. lens size was adequate. However, in view of the experience of 4-leg intersections, a 12-in. lens is also recommended.

It is recommended that for red-yellow 4-leg intersections an average accident rate reduction of 50 percent or an average base rate of 1.1 be used to estimate the number of future accidents. For 4-way red flasher installations, an average rate reduction of 75 percent can be used or an after base rate of 0.8 accidents per million vehicles entering. The preferred method is the use of after base rate.

The flashers were effective in reducing the number of vehicles that run through a T-intersection from the minor road across the highway.

Advance warning beacons are effective in reducing single vehicle accidents of the "ran-off-the-road" variety. Greater accident reductions are realized at nighttime although daytime accidents were also reduced. Rear-end accident increases were noted with the school flashers, which had otherwise little or no change in the accident experience after the improvement.

The numbers of projects within each subcategory of advance warning beacons were considered insufficient for any analyses on the effectiveness of present warrants. Additional research should be considered for the following flashing beacon categories: (a) 3-leg or T-intersections, and (b) advance warning beacons used in conjunction with advance warning signs; i.e., SIGNAL AHEAD, STOP AHEAD, SCHOOL AHEAD.

Safety Lighting

The study was conducted to evaluate the current effectiveness of safety lighting installations in reducing the nighttime accident experience and to determine if more objective criteria could be established for installing safety lighting.

The before-and-after study method was employed to evaluate 41 project reports. These 41 reports were subdivided into various categories by type of location. Table 1 illustrates the percent reduction in the total accident rate and the night accident rate for the various categories.

Safety lighting as a whole has been quite effective in reducing the night accident rate (64 percent reduction) with a 65 percent reduction at intersections, 24 percent

TABLE 1

SUMMARY OF BEFORE AND AFTER ACCIDENT EXPERIENCE FOR SAFETY LIGHTING PROJECTS, CLASSIFIED BY TYPE OF LOCATION

LOCATION TYPE	NUMBER OF PROJECTS	BEFORE EXPERIENCE					AFTER EXPERIENCE					PERCENT CHANGE				
		ACCIDENTS			TOTAL ACCIDENT RATE	NIGHT a/ ACCIDENT RATE	ACCIDENTS			TOTAL ACCIDENT RATE	NIGHT a/ ACCIDENT RATE	ACCIDENTS			TOTAL ACCIDENT RATE	NIGHT a/ ACCIDENT RATE
		DAY	NIGHT	TOTAL			DAY	NIGHT	TOTAL			DAY	NIGHT	TOTAL		
Three-Leg Intersection	10	39	75	114	2.24	4.41	39	27 S	66 S	1.10	1.34	0	-64	-42	-51	-70
Four-Leg Intersection, 2-lanes on Major Leg	7	31	25	56	2.23	2.98	34	11 S	45 S	1.61	1.18	+10	-56	-20	-28	-60
Four-Leg Intersection, 4-lanes on Major Leg	9	31	33	64	1.03	1.59	34	13 S	47 S	0.72	0.60	+10	-61	-27	-30	-62
Upgraded Lighting at Urban Intersections	4	25	15	40	1.19	1.34	30	7 S	37	0.91	0.52	+20	-53	-8	-24	-61
Railroad Crossings with reduced alignment standards	6	12	37	49	7.80	17.62	12	15 S	27 S	3.75	6.25	0	-60	-45	-52	-65
Bridge Approaches with reduced alignment standards	2	7	11	18	5.63	10.00	8	5 S	13	4.06	4.55	+14	-55	-28	-28	-54
Underpasses	3	12	7	19	0.64	0.71	15	7	22	0.65	0.62	+25	0	+16	+2	-13
TOTAL	41	157	203	360	1.74	2.94	172	85 S	257 S	1.08	1.07	+10	-58	-29	-38	-64

a/ Assume 1/3 MV at night for rate calculations
"S" Indicates change is significant at the 0.10 level using the Chi-Square Test

reduction for upgraded illumination at urban intersections, 52 percent reduction at railroad crossings having reduced alignment standards, 28 percent reduction at bridge approaches with poor alignment, and a 2 percent night accident rate increase at underpasses. All categories but two, "underpasses" and "at bridge approaches," exhibited significant reductions in the night accident rate.

Eight possible accident warrants for safety lighting were compared with the present accident warrants which permit lighting if "... there are five or more accidents a year and 50 percent or more occurring under conditions other than daylight; or there are less than five accidents per year and three or more accidents per year occurring under conditions other than daylight." Five of the warrants were more effective than the present accident warrant in reducing the night accident experience.

It is recommended that safety lighting be considered at locations which experience 4 night accidents in one year or 6 or more night accidents in two years. It is also recommended that an average night accident rate reduction of 75 percent or an average after base rate of 0.8 accidents per million vehicles be used to estimate the number of future accidents at an intersection meeting the recommended accident warrants for a safety lighting installation. The preferred method is the use of the after base rate of 0.8.

Delineation

The following findings are based on a relatively small number of locations and, in some cases, a small accident experience; thus, the representativeness of the data is open to question. Additionally, data on items most relevant to the issues being investigated were sometimes unavailable. Consequently, the findings are of a provisional nature.

1. There was a reduction in total accident rates at the delineation projects reviewed.
2. Accident rates were reduced when double yellow stripes were placed next to cable barrier installations in freeway medians. No reductions were found when a single white stripe was used.

3. Accident rates remained the same when double yellow stripes were placed next to beam barrier installations in freeway medians.
4. Accident rates were reduced on conventional two-lane highways having curve radii of 500 ft or less when guide markers were placed on the outside of the curve.
5. Ran-off-the-road accident rates were reduced on two-lane conventional highways when a 2-in. edge stripe was used.

Guardrail

The study evaluated the current effectiveness of protective guardrail projects on conventional two-lane highways.

The delineation quality of protective guardrail has been quite effective in reducing accident rates with a 70 percent reduction at metal plate (white) guardrail installations and a 44 percent reduction at metal beam (gray) guardrail installations. Accidents were significantly reduced at nighttime. Total accident severity was also reduced; mainly the single vehicle accidents at metal plate guardrail installations.

Because of the greater effectiveness of the white metal plate guardrail, it is recommended that at locations where ran-off-the-road accidents predominate and guardrail warrants are satisfied, consideration be given to enhancing the delineation quality of the metal beam guardrail either by painting or some other means.

STUDY DESCRIPTION

In the past two decades, we have become accustomed to thinking of highway improvements in terms of six, eight and more lane freeways with their accompanying multimillion dollar price tags. This large allocation of monies for controlled-access highways may be directly equated to traffic safety, inasmuch as freeways are more than twice as safe as other roads (1). There are other safety improvements that can be made, however, whose price tags label them as bargains in the overall highway improvement program.

Although new or improved freeways will decrease the pressure on the presently overtaxed streets and roads, the fact remains that the motorist must drive on the conventional road system for at least part of his travel. Therefore, the California Division of Highways, for many years has channeled certain funds into a Minor Improvement Program to increase safety to the conventional road user. For Fiscal Year 1967-68 over $6,500,000 has been budgeted for "minor" safety projects. Projects undertaken in this program are minor in respect to funds expended (usually less than 50,000 dollars per project), but often are a major benefit in respect to increased safety for the road user. In addition, $7,200,000 will also be spent for safety-oriented projects of major size.

Study Objectives

The minor safety improvement projects have been accomplished with a minimum of guidelines for determining the best improvement type and with little knowledge as to expected safety benefits, especially for estimating the probable magnitude of the accident reduction expected.

The "Evaluation of Minor Improvements" study was designed to develop objective criteria for the evaluation of minor improvements, and thereby permit maximum safety benefits per dollar spent in the Minor Improvement Program. The objectives of the overall study, therefore, are the following:

1. To determine how effective the various types of minor improvements have been in reducing traffic accidents.
2. To determine what conditions are susceptible to improvement and how much improvement can be expected.
3. To determine methods and measures for predicting the magnitude of the accident reduction on proposed minor improvement projects.
4. To review present improvement warrants for validity and adequacy and to determine if new warrants are required.

Study Procedure

The method chosen for studying the effectiveness of minor improvements was the before-and-after study procedure. The procedure calls for the collection of data pertinent to the information sought, immediately before and immediately after the improvement is made. The advantage of the procedure is that variables which are not pertinent to the information sought are held fairly constant over both study periods. If all conditions other than the improvement made remain the same in both study periods, an associative relation can be assumed between the improvement and any changes in operational characteristics (e. g., between the improvement and accident reduction).

Data Collection

Instructions for the preparation of before-and-after study reports of project performance at minor improvement locations were issued to all California highway districts in 1958. The instructions included an outline of needed information and a suggested report form for submitting the information. The intent was to obtain similar, consistent and pertinent data. The length of the study periods were to be the same both before and after the installation and at least one year in length if possible. In addition, the before and after periods were to cover the same months to eliminate bias due to seasonal patterns.

There were approximately 500 minor improvement evaluation reports submitted. The types of improvements have been classified and separate reports will be published on each class of improvement. The classes of minor improvements are as follows:

1. Flashing beacons,
2. Safety lighting,
3. Guardrail,
4. Delineation,
5. Channelization,
6. Signs,
7. Reconstruction,
8. New traffic signals,
9. New traffic signals with channelization,
10. Modified traffic signals,
11. Modified traffic signals with channelization, and
12. Miscellaneous.

Methodology

Before and after periods of equal lengths were compared. To avoid bias due to seasonal fluctuations in accidents, the same number of each calendar month was used in each pair of before and after periods when fractional parts of a year were used (e. g., May 1961 to December 1962, before; May 1963 to December 1964, after).

The periods used were, insofar as was possible, immediately prior to an immediately after the improvement construction to reduce the influence of any general trend in accident rates. An investigation of a possible increasing or decreasing trend showed no such phenomenon. When the construction period was of short duration, it was placed in the before period, and in no cases were lengthy construction periods included in the analysis.

The possibility always exists that an improvement project may have been initiated because of an unusually high accident experience which was merely a reflection of a temporary condition in the before period. In such cases, even if nothing had been done, an accident reduction would probably have been observed in the after period (regression to the mean theory). The possibility of such an influence was investigated. Even though a few projects were initiated immediately after or during such high peaks, it was found that most projects were the result of sustained high levels of accident experience. Additionally, because of the time required to recognize the problem, investigate the causes, design a solution, prepare the necessary plans and specifications, obtain fundings, advertise for bids, and for the contractor to start construction, most before periods used in the individual studies do not coincide with the accident period that initiated the project. This, in effect, should have resulted in randomly selected before periods.

The before-and-after accident experience was generally compared in three ways, as follows:

1. Number of accidents and accident rate.
2. Number of equivalent property damage only (EPDO) accidents and EPDO accident rates.
3. The severity index (SI).

The accident rate is simply the number of accidents related to vehicle exposure (the total entering volume when considering intersections). For improvements involving substantial lengths of highway (over ½ mile), such as edgelining, exposure was measured in vehicle-miles of travel.

The EPDO was based on direct costs (2) of accidents by severity class. The direct costs and severity weight of accidents are as follows:

	PDO	Injury	Fatal
Direct cost	$400	$2000	$9000
Severity weight	1	5	23

Since a fatal accident is a relative rare event, the presence or absence of which may distort a small sample, a weighted average (based on the sum of all minor improvement projects) within each category was used where the individual categories were thought to have sufficient accident experience. Thus, the number of EPDO accidents equals the number of PDO's plus the weighted average (W) times the total number of injury (I) and fatal (F) accidents, e. g., EPDO equals PDO + W (I + F). When such a weighted average was not feasible, a weight of six was applied, since this represents the weight arrived at for the entire accident experience on the California State Highway System. The EPDO rate is simply the number of equivalent PDO accidents divided by some measure of exposure.

The severity index (SI) is the average severity of accidents for a given condition. It is computed by dividing the number of EPDO accidents by the total number of accidents before or after the improvement, $\left(\text{e.g., SI} = \frac{\text{No. of EPDO accidents}}{\text{Total accidents}}\right)$

Since SI is a measure of the average accident severity, if the number of EPDO accidents was not in the same proportion as the total number of accidents (before and after), the SI's and EPDO's could change in opposite directions. This is illustrated by the following example from the Railroad Flashing Beacon Summary.

(W=7)	PDO	Inj + Fat	Tot Accid	EPDO	SI
Before	6	6	12	48	4.0
After	0	2	2	14	7.0

The EPDO's reflect a substantial reduction after the improvement; whereas, the SI's indicate a substantial increase in average severity. It is readily apparent from the fact that accident frequency decreased in all severity classes that the after period shows an improvement. This improvement occurred in spite of the fact that the average severity per accident increased because the more severe accidents decreased proportionately less.

The EPDO should be considered as the criterion measure since it reflects both the number of accidents and the severity, and may be considered to measure the cumulative severity; whereas, the SI reflects only the mean severity.

General accident rates for the several years of the study period were reviewed to determine if a factor should be applied to adjust for a trend. No trend was determined. Therefore, no adjustments were made.

Statistical Significance Testing

The chi-square test was generally employed to establish whether the reductions in accidents were statistically significant (3). A confidence level of 0.10 was used. In other words, any significant difference could not have occurred by chance more than 10 times out of 100. The chi-square test involves the computation of the difference in observed and "expected" frequencies. The expected frequencies are computed according to the hypothesis that there are equal accident rates for both before and after periods. To determine the expected number of accidents for each period, the sum of accidents for both periods was distributed in proportion to the total vehicle exposure that occurred in each period.

In the sections on delineation and guardrail, the statistical reliability of observed changes is often indicated by a footnote, e. g., χ^2 at 1 df = 3.90, $P < 0.05$. This means that the computed chi-square value was 3.90. At one degree of freedom a value equal to or greater than 3.90 would be expected to occur by chance no more than 5 times out of 100. Thus, we can be 95 percent confident that the difference observed was a true difference and not one due to random sampling fluctuations (Yates' correction for continuity was utilized for all expected frequencies under 100).

Because of relatively few (generally less than 20) accidents occurring in the before period for any one location, a reduction even as high as 50 percent for the after period is rarely statistically significant; that is, for such a small sample, this amount of reduction could have occurred because of chance variation. Therefore, the hypothesis that the highway improvement caused the accident reduction cannot be accepted with confidence. However, a large sample which is the sum of several projects may show a significant reduction from before to after because of the added power due to the increased sample size. For instance, a 60 percent reduction from 10 to 4 is not statistically significant; whereas, a 22 percent reduction from 100 to 78 is significant (assuming equal volumes in both periods).

Because both EPDO accidents and rates and SI's are based on weighted values and do not represent frequencies, no statistical tests were conducted on them. Instead, rational inferences were made concerning their probable statistical strengths based on the reliability of the statistics of the original raw data (the original accident numbers).

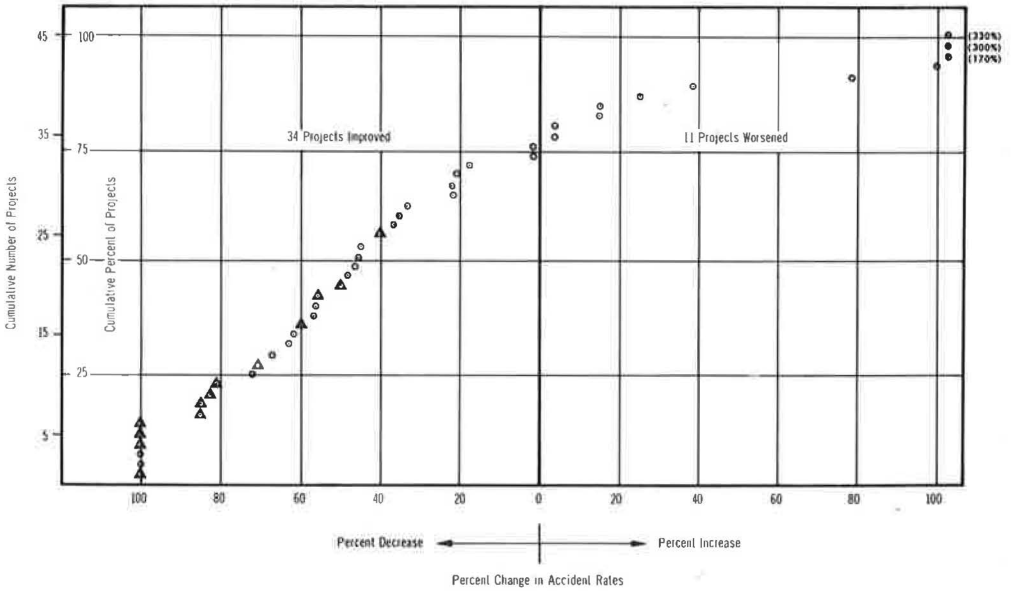
FLASHING BEACONS

Of the 52 flashing beacon projects evaluated, only 45 projects had sufficient information for a detailed comparative analysis. Approximately 75 percent of the projects had a reduction in accident rates although not all were statistically significant. Figure 1 illustrates that percent reduction of accident rates alone is an unreliable indicator of the success of the improvement.

Flashing beacons as a whole have been quite effective in reducing accidents (34 percent reduction), with an 83 percent reduction at railroad crossings, 40 percent reduction at intersections and 21 percent reduction at advance warning beacon intersections.

Presuming a 20-yr project life, the cost per accident reduced by flashing beacons averages \$38 and ranged from \$27 for beacons at intersections to \$56 for advance warning beacons to \$328 for flashing beacons at railroad crossings (Table 2).

Accidents were weighted by severity class, relative to their direct costs. This was done in an attempt to place a relative value on each severity class of accident. Based on the severity distribution experienced with the 45 flashing beacon projects, the



△ Indicates statistically significant at the 0.10 level using the Chi-Square Test

Figure 1. Minor improvement projects: flashing signals.

weighted number (W) for flashing beacons was determined to be seven. The equation for calculating the number of equivalent property damage only accidents is EPDO's = PDO's + 7 (injury + fatal accidents).

The flashing beacon projects were divided into two categories, those placed at an intersection as a control device, and those placed in conjunction with advance warning signs. The latter category is used to call attention to a location ahead, where a hazard exists to free traffic flow. For the sake of brevity, the two categories were called intersection beacons and advance warning beacons.

Flashing Beacons at Intersections

Table 3 summarizes the accident data for 29 minor improvement projects in which flashing beacons were installed at intersections. It is subdivided by red-yellow

TABLE 2

FLASHING BEACONS PROJECT AND COST SUMMARY

	PROJECTS 1/				ACCIDENTS				ACCIDENTS/YEAR						2/
							Net Change				Net Change				
	Number	Improved	Worsened	No Change	Before	After	Number	Percent	Before	After	Number	Percent	Total Cost	Average Cost Per Project	Cost/Accident Reduced
Intersection	29	11[S]	0	18	326	197	-129[S]	-40	204	124	-80	-39	$43,802	$1,510	$ 27
Advance Warning 3/	13	4[S]	0	9	155	122	-33[S]	-21	98	77	-21	-22	$23,641	$1,818	$ 56
Railroad Crossing	3	2[S]	0	1	12	2	-10[S]	-83	5	1	-4	-80	$26,216	$8,738	$328
TOTAL	45	17[S]	0	28	493	321	-172[S]	-34	307	202	-105	-34	$93,659	$2,081	$ 45

1/ Only those reports which contained sufficient data are listed

2/ Assuming a 20 year life of project

3/ Includes 2 Bridge Approach Projects

"S" Indicates change is significant at the 0.10 level using the Chi-Square Test.

TABLE 3

SUMMARY INTERSECTION FLASHING BEACONS

Category	Intersection	Period	Item	Projects: Total No	Projects: Improved	Projects: Worsened	Projects: No Change	Years of Experience	Single Vehicle: Ran off Road	Single Vehicle: Other	Single Vehicle: Sub-Total	Multiple Vehicle: Left Turn	Multiple Vehicle: Rear End	Multiple Vehicle: Crossing	Multiple Vehicle: Other	Multiple Vehicle: Sub-Total	Severity: PDO	Severity: Injury	Severity: Fatal	LT Cond: Day a/	LT Cond: Night a/	Total Accidents	Million Vehicles	Equivalent PDO (EPDO)	Severity Index (SI)
Red - Yellow Flashers	4 Leg Intersection	Before	No of Accidents	15				26 3/12	3	3	6	24	24	129	10	187	113	69	11	128	65	193	84.3	673	3.5
			Rate						0.04	0.04	0.07	0.28	0.28	1.53	0.12	2.22	1.34	0.82	0.13	2.28	2.34	2.29		7.98	
		After	No of Accidents	15	4 S	0	11	26 3/12	5	0	5	24	14 S	94 S	8	140 S	91 S	49 S	5	103 S	42 S	145 S	91.3	469	3.2
			Rate						0.05	0	0.05	0.26	0.15	1.03	0.09	1.53	1.00	0.54	0.05	1.69	1.38	1.59		5.14	
			% Rate Change						+25	-100	-29	-7	-46	-33	-25	-31	-25	-34	-62	-26	-40	-31		-36	
	3 Leg Intersection	Before	No of Accidents	6				6 11/12	19	2	21	5	5	9	5	24	18	22	5	26	19	45	31.3	207	4.6
			Rate						0.61	0.06	0.67	0.16	0.16	0.29	0.16	0.77	0.58	0.70	0.16	1.24	1.83	1.44		6.61	
		After	No of Accidents	6	2 S	0	4	6 11/12	4 S	0	4 S	3	5	8	2	18	12	10 S	0 S	13 S	9 S	22 S	32.2	82	3.7
			Rate						0.12	0	0.12	0.09	0.16	0.25	0.06	0.56	0.37	0.31	0	0.60	0.84	0.68		255	
			% Rate Change						-80	-100	-82	-44	0	-14	-63	-27	-36	-56	-100	-52	-54	-53		-61	
	Sub Total	Before	No of Accidents	21				33 2/12	22	5	27	29	29	138	15	211	131	91	16	154	84	238	115.6	880	3.7
			Rate						0.19	0.04	0.23	0.25	0.25	1.19	0.13	1.83	1.13	0.79	0.14	2.00	2.18	2.06		7.61	
		After	No of Accidents	21	6 S	0	15	33 2/12	9 S	0 S	9 S	27	19	102 S	10	158 S	103 S	59 S	5 S	116 S	51 S	167 S	123.5	551	3.3
			Rate						0.07	0	0.07	0.22	0.15	0.83	0.08	1.28	0.83	0.48	0.04	1.41	1.24	1.35		4.47	
			% Rate Change						-63	-100	-70	-12	-40	-30	-38	-30	-27	-39	-71	-30	-43	-34		-41	
4-Way Red	4 Leg Intersection	Before	No of Accidents	8				12	7	1	8	16	5	52	7	80	44	38	6	69	19	88	31.8	352	4.0
			Rate						0.22	0.03	0.25	0.50	0.16	1.63	0.22	2.52	1.38	1.19	0.19	3.25	1.78	2.77		11.07	
		After	No of Accidents	8	4 S	0	4	12	2	2	4	3 S	11	10 S	2	26 S	22 S	8 S	0 S	21 S	9 S	30 S	34.1	78	2.6
			Rate						0.06	0.06	0.12	0.09	0.32	0.29	0.06	0.76	0.65	0.23	0	0.92	0.79	0.88		2.29	
			% Rate Change						-73	+100	-52	-82	+100	-82	-73	-70	-53	-81	-100	-72	-56	-68		-79	
TOTAL		Before	No. of Accidents	29				45 2/12	29	6	35	45	34	190	22	291	175	129	22	223	103	326	147.4	1232	3.8
			Rate						0.20	0.04	0.24	0.31	0.23	1.29	0.15	1.97	1.19	0.87	0.15	2.27	2.10	2.21		8.35	
		After	No of Accidents	29	10 S	0	19	45 2/12	11 S	2	13 S	30	30	112 S	12 S	184 S	125 S	67 S	5 S	137 S	60 S	197 S	157.6	629	3.2
			Rate						0.06	0.01	0.08	0.19	0.19	0.71	0.08	1.17	0.79	0.43	0.03	1.30	1.14	1.25		3.99	
			% Rate Change						-70	-75	-67	-39	-17	-45	-47	-41	-34	-51	-80	-43	-46	-43		-52	

a/ Assume 2/3 MV for Day and 1/3 MV at night for rate calculations.
"S" Indicates change is significant at the 0.10 level using the Chi-Square Test

flashers at 4-leg intersections, red-yellow or single red flashers at 3-leg intersections, and 4-way red flashers at 4-leg intersections.

The 29 projects represent 45 years of experience before and 45 years after the installation of the beacons. Of the 29 projects, 11 showed an improvement based on the total number of accidents and on a statistical level of significance of 0.10 (χ^2). The number of equivalent property damage only accidents were also reduced to one-half.

On the basis of either total accidents or total equivalent PDO accidents, flashing signals at intersections caused a reduction in the accident rates. The rates were also reduced for each of the three subcategories.

In addition to the reduction in accident rates, flashing signals caused a reduction in accident severity in all categories. This is evident from an examination of the severity index (SI) or by noting that the percentage reduction in the accident rates increases as the severity increases. The reduction in severity and in the accident rates are greatest for the 4-way red flashers at 4-leg intersections and least for the red-yellow flashers at 4-leg intersections.

With the exception of the 3-leg intersections, the main problem in the before condition is the multiple-vehicle accident. In all cases, the multiple-vehicle accident rates were reduced markedly, especially the right-angle broadside collisions.

In the case of the 3-leg intersections, the accident problem was approximately evenly divided between the single-vehicle and multiple-vehicle categories. Most of the single-vehicle accidents were the result of vehicles on the minor leg of the intersection (stem) overrunning the intersection and running off the road. The flashers virtually eliminated this type of accident. At two locations, a single red flasher was placed facing the stem of the T. At the other four locations, a yellow flasher for each direction of the through traffic and a red flasher facing the stem was used. For this small sample, approximately the same results were obtained for both types of installation.

TABLE 4

RED-YELLOW FLASHERS (4-LEG)

			PROJECTS				ACCIDENT DESCRIPTION																	
								ACCIDENT TYPE								SEVERITY			LT COND.					
								SINGLE VEHICLE			MULTIPLE VEHICLE								a/	a/				
			Total No	Improved	Worsened	No Change	Years of Experience	Ran off Road	Other	Sub-Total	Left Turn	Rear End	Crossing	Other	Sub-Total	PDO	Injury	Fatal	Day	Night	Total Accidents	Million Vehicles	Equivalent PDO (EPDO)	Severity Index (SI)
CHANNELIZED	Before	No of Accidents	7				$13\frac{6}{12}$	2	3	5	15	17	74	6	112	80	33	4	82	35	117	61.8	339	2.9
		Rate						0.03	0.05	0.08	0.24	0.28	1.20	0.10	1.81	1.29	0.53	0.06	1.99	1.70	1.89		5.49	
	After	No of Accidents	7	1^S		6	$13\frac{6}{12}$	5	0	5	14	11	55^S	6	86^S	61^S	27	3	60^S	31	91^S	65.4	271	3.0
		Rate						0.08	0	0.08	0.21	0.17	0.84	0.09	1.31	0.93	0.41	0.05	1.38	1.42	1.39		4.14	
		% Rate Change						+167	-100	0	-12	-39	-30	-10	-28	-28	-23	-17	-31	-16	-26		-25	
NON CHANNELIZED	Before	No of Accidents	8				$12\frac{9}{12}$	1	0	1	9	7	55	4	75	33	36	7	46	30	76	22.5	334	4.4
		Rate						0.04	0	0.04	0.40	0.31	2.44	0.18	3.33	1.47	1.60	0.31	3.07	4.00	3.38		14.84	
	After	No of Accidents	8	3^S		5	$12\frac{9}{12}$	0	0	0	10	3	39^S	2	54^S	30	22^S	2	43	11^S	54^S	25.9	198	3.7
		Rate						0	0	0	0.39	0.12	1.51	0.08	2.08	1.16	0.85	0.08	2.49	1.28	2.08		7.65	
		% Rate Change						-100	0	-100	-2.5	-61	-38	-56	-38	-21	-47	-74	-19	-68	-38		-48	
TOTAL	Before	No of Accidents	15				$26\frac{3}{12}$	3	3	6	24	24	129	10	187	113	69	11	128	65	193	84.3	673	3.5
		Rate						0.04	0.04	0.07	0.28	0.28	1.53	0.12	2.22	1.34	0.82	0.13	2.28	2.31	2.29		7.98	
	After	No of Accidents	15	4^S		11	$26\frac{3}{12}$	5	0	5	24	14^S	94^S	8	140^S	91^S	49^S	5	103^S	42^S	145^S	91.3	469	3.2
		Rate						0.05	0	0.05	0.26	0.15	1.03	0.09	1.53	1.00	0.54	0.05	1.69	1.38	1.59		5.14	
		% Rate Change						+25	-100	-29	-7	-46	-33	-25	-31	-25	-34	-62	-26	-40	-31		-36	

a/ Assume 2.3 MV for Day and 1.3 MV at night for rate calculations

"S" Indicates change is significant at the 0.10 level using the Chi-Square Test

In general, flashing beacons at intersections were equally effective day or night, even though one might suspect flashers to be more effective at night when they should be more noticeable. Some drivers complained that the flashers were too bright at night. This condition has been alleviated in one district by reducing the voltage at night to give a similar level of brightness as in the daytime.

Red-Yellow Flashing Beacons—In California, red-yellow flashing beacons are usually provided where sight distance is extremely limited or where other conditions make it especially desirable to emphasize the need for stopping on one street and for proceeding with caution on the other. They are also used when there has been a preponderance of broadside or crossing type accidents. All red flashers are backed up by stop signs and stop bars.

Table 4 summarizes the flashing beacon category consisting of red and yellow flashers at 4-leg intersections. Eight of the 15 projects showed improvements based on equivalent PDO accidents. Of the total before accident problem, 95 percent were multiple-vehicle accidents, and the major portion of this problem was the right-angle crossing collisions. The installation of red-yellow flashers at 4-leg intersections reduced the accident rates approximately one-third.

The accident rate reduction was considerably greater at the nonchannelized intersections than at the channelized. Even so, the after accident rate at nonchannelized intersections was still higher than the before accident rate at channelized intersections. The nonchannelized intersections also showed a marked reduction in severity with no reduction in severity for the channelized intersections. Most of the reduction in severity for the nonchannelized intersections occurred at night when accidents are more severe.

Not only were the right-angle crossing collisions a major part of the before accident problem, it was also the only category in both the channelized and nonchannelized intersections that showed a statistically significant reduction in accidents.

Since only one project experienced an appreciable number of rear-end accidents, we do not know if flashing beacons at intersections have any effect on this type of accident. At this flasher installation, however, rear-end accidents were reduced 10 to 2. This was the first of a series of intersections encountered by motorists when entering a small town on the state highway.

Table 5 summarizes the accident data for red-yellow flashers at 3-leg intersections. Of the six projects, two showed a statistically significant improvement based on total accidents and four showed improvements based on equivalent PDO accidents. The projects and accidents were too few in number to make any further detailed analyses. As stated before, the problem in this category was mainly single vehicles running off

TABLE 5

RED-YELLOW FLASHERS (3-LEG)

			PROJECTS: Total No	PROJECTS: Improved	PROJECTS: Worsened	PROJECTS: No Change	Years of Experience	SINGLE VEHICLE: Ran off Road	SINGLE VEHICLE: Other	SINGLE VEHICLE: Sub-Total	MULTIPLE VEHICLE: Left Turn	MULTIPLE VEHICLE: Rear End	MULTIPLE VEHICLE: Crossing	MULTIPLE VEHICLE: Other	MULTIPLE VEHICLE: Sub-Total	SEVERITY: PDO	SEVERITY: Injury	SEVERITY: Fatal	LT. COND.: Day a/	LT. COND.: Night a/	Total Accidents	Million Vehicles	Equivalent PDO (EPDO)	Severity Index (SI)
CHANNELIZED	Before	No. of Accidents	3				3 $\frac{6}{12}$	12	0	12	5	2	5	2	14	8	13	5	15	11	26	22.2	134	5.2
		Rate						0.54	0	0.54	0.23	0.09	0.23	0.09	0.63	0.36	0.59	0.23	1.01	1.49	1.17		6.04	
	After	No. of Accidents	3	1^S		2	3 $\frac{6}{12}$	3^S	0	3^S	2	3	3	2	10	5	8	0^S	8	5	13^S	22.8	61	4.7
		Rate						0.13	0	0.13	0.09	0.13	0.13	0.09	0.44	0.22	0.35	0	0.53	0.66	0.57		2.68	
		% Rate Change						-76	0	-76	-61	+44	-43	0	-30	-39	-41	-100	-48	-56	-51		-56	
NON CHANNELIZED	Before	No. of Accidents	3				3 $\frac{5}{12}$	7	2	9	0	3	4	3	10	10	9	0	11	8	19	9.1	73	3.8
		Rate						0.77	0.22	0.99	0	0.33	0.44	0.33	1.10	1.10	0.99	0	1.80	2.67	2.09		8.02	
	After	No. of Accidents	3	1^S		2	3 $\frac{5}{12}$	1^S	0	1^S	1	2	5	0	8	7	2^S	0	5	4	9^S	0.4	21	2.3
		Rate						0.11	0	0.11	0.11	0.21	0.53	0	0.85	0.74	0.21	0	0.79	1.29	0.96		2.23	
		% Rate Change						-86	-100	-89	∞	-36	+20	-100	-23	-33	-79	0	-56	-52	-54		-72	
TCTAL	Before	No. of Accidents	6				6 $\frac{11}{12}$	19	2	21	5	5	9	5	24	18	22	5	26	19	45	31.3	207	4.6
		Rate						0.61	0.06	0.67	0.16	0.16	0.29	0.16	0.77	0.58	0.70	0.16	1.24	1.83	1.44		6.61	
	After	No. of Accidents	6	2^S		4	6 $\frac{11}{12}$	4^S	0	4^S	3	5	8	2	18	12	10^S	0^S	13^S	9^S	22^S	32.2	82	3.7
		Rate						0.12	0	0.12	0.09	0.16	0.25	0.06	0.56	0.37	0.31	0	0.60	0.84	0.68		2.55	
		% Rate Change						-80	-100	-82	-44	0	-14	-63	-27	-36	-56	-100	-52	-54	-53		-61	

a/ Assume 2/3 MV for Day and 1/3 MV at night for rate calculations.
"S" Indicates change is significant at the 0.10 level using the Chi-Square Test

the end of the minor road stem. Right-angle crossing collisions in this case were not reduced significantly. Again, severity was decreased most markedly at the nonchannelized intersections, but unlike the 4-leg intersections this reduction was not primarily due to a reduction in the nighttime accidents.

Table 6 compares the 21 red-yellow center suspended intersections of this study with a similar study of 25 beacon installations in Michigan (4). Both studies indicate greater rate reductions at low-volume intersections (below 8000 ADT) than at higher volume intersections (over 8000 ADT). Average rate reductions for this study (-34 percent) were greater than the Michigan study (-17 percent). Angle or crossing collisions predominate before and after the improvement, although there were overall rate reductions in both studies.

Four-Way Red Flashing Beacons—Four-way red flashing beacons have usually been installed in California where sight distance is extremely limited or where the minor leg entering volumes are high enough to require equal right-of-way status with the major legs. This appears to be in a minor to major leg entering volume ratio greater than 0.50. All red flashers in California are backed up by stop signs and stop bars. Four-way red flashers or 4-way stops at high-volume intersections are generally an interim condition until funds can be obtained for traffic signals.

Table 7 summarizes the accident data for the 4-way red flasher category.

There were eight projects in which 4-way red flashers were installed. At four locations, the total number of accidents was significantly reduced and at all eight locations the equivalent PDO accidents were reduced. The accident rate was reduced by two-thirds (68 percent) based on total accidents and by three-fourths (79 percent) based on equivalent PDO accidents. The before accident

TABLE 6

COMPARISON OF CENTER SUSPENDED RED-YELLOW FLASHING BEACON DATA: CALIFORNIA VS. MICHIGAN STUDY

Intersection Type	No. of Projects	Avg. ADT Before	After Period Accidents: Rate	After Period Accidents: % Change B to A
(a) Michigan Study				
T-intersection	5	6,700	1.78	- 6
4-leg undivided	15	8,200	1.50	-13
4-leg divided	2	7,800	2.90	-22
5 and 6 leg	3	9,000	1.01	-46
Total	25	8,000	1.60	-17
Projects with ADT < 8,000	17		2.23	-22
Projects with ADT > 8,000	8		1.12	-10
(b) California Study				
T-intersection	6	12,300	0.68	-53
4-leg undivided	9	6,100	1.85	-23
4-leg divided	6	11,700	1.35	-38
5 and 6 leg	0	—	—	—
Total	21	9,500	1.36	-34
Projects with ADT < 8,000	10		1.97	-53
Projects with ADT > 8,000	11		1.21	-23

TABLE 7

RED FLASHERS (4-LEG STOP)

Prior Condition			PROJECTS: Total No	Improved	Worsened	No. Change	Years of Experience	ACCIDENT DESCRIPTION — ACCIDENT TYPE — SINGLE VEHICLE: Ran off Road	Other	Sub-Total	MULTIPLE VEHICLE: Left Turn	Rear End	Crossing	Other	Sub-Total	SEVERITY: PDO	Injury	Fatal	LT. COND.: Day a/	Night a/	Total Accidents	Million Vehicles	Equivalent PDO (EPDO)	Severity Index (SI)
2 Way Stop Sign	Before	No. of Accidents	3				5	3	1	4	8	0	16	2	26	15	13	2	25	5	30	10.9	120	4.0
		Rate						0.28	0.09	0.37	0.73	0	1.47	0.18	2.38	1.38	1.19	0.18	3.43	1.39	2.75		11.01	
	After	No. of Accidents	3	2^{S}	0	1	5	1	2	3	3	3	2^{S}	0	8^{S}	7^{S}	4^{S}	0	5^{S}	6	11^{S}	11.7	35	3.2
		Rate						0.09	0.17	0.26	0.26	0.26	0.17	0	0.68	0.60	0.34	0	0.64	1.54	0.94		2.99	
		% Rate Change						-68	+89	-30	-64	∞	-88	-100	-71	-57	-71	-100	-81	+11	-66		-73	
4 Way Stop Sign	Before	No. of Accidents	2				4	3	0	3	4	4	4	0	12	5	8	2	8	7	15	10.8	75	5.0
		Rate						0.28	0	0.28	0.37	0.37	0.37	0	1.11	0.46	0.74	0.18	1.11	1.94	1.39		6.94	
	After	No. of Accidents	2	0^{S}	0	2	4	0	0	0	0	8	2	2	12	9	3	0	12	0^{S}	12	11.6	30	2.5
		Rate						0	0	0	0	0.69	0.17	0.17	1.03	0.78	0.26	0	1.56	0	1.03		2.59	
		% Rate Change						-100	0	-100	-100	+86	-54	∞	-7	+70	-65	-100	+40	-100	-26		-63	
Sub Total Stop Signs	Before	No. of Accidents	5				9	6	1	7	12	4	20	2	38	20	21	4	33	12	45	21.7	195	4.3
		Rate						0.28	0.05	0.32	0.55	0.18	0.92	0.09	1.75	0.92	0.97	0.18	2.28	1.66	2.07		8.99	
	After	No. of Accidents	5	2^{S}	0	3	9	1	2	3	3^{S}	11	4^{S}	2	20^{S}	16	7^{S}	0^{S}	17^{S}	6^{S}	23^{S}	23.3	65	2.8
		Rate						0.04	0.09	0.13	0.13	0.47	0.17	0.09	0.86	0.69	0.30	0	1.10	0.77	0.99		2.79	
		% Rate Change						-86	+80	-59	-76	+161	-82	0	-51	-25	-69	-100	-52	-54	-52		-69	
Red - Yellow Flashing Beacons	Before	No. of Accidents	3				3	1	0	1	4	1	32	5	42	24	17	2	36	7	43	10.1	157	3.7
		Rate						0.10	0	0.10	0.40	0.10	3.17	0.50	4.16	2.38	1.68	0.20	5.37	2.06	4.26		15.54	
	After	No. of Accidents	3	2^{S}	0	1	3	1	0	1	0^{S}	0	6^{S}	0^{S}	6^{S}	6^{S}	1^{S}	0	4^{S}	3	7^{S}	10.8	13	1.9
		Rate						0.09	0	0.09	0	0	0.56	0	0.56	0.56	0.09	0	0.56	0.83	0.65		1.20	
		% Rate Change						-10	0	-10	-100	-100	-82	-100	-87	-76	-95	-100	-90	-60	-85		-92	
TOTAL	Before	No. of Accidents	8				12	7	1	8	16	5	52	7	80	44	38	6	69	19	88	31.8	352	4.0
		Rate						0.22	0.03	0.25	0.50	0.16	1.64	0.22	2.52	1.38	1.19	0.19	3.25	1.79	2.77		11.07	
	After	No. of Accidents	8	4^{S}	0	4	12	2	2	4	3^{S}	11	10^{S}	2	26^{S}	22^{S}	8^{S}	0^{S}	21^{S}	9^{S}	30^{S}	34.1	78	2.6
		Rate						0.06	0.06	0.12	0.09	0.32	0.29	0.06	0.76	0.64	0.23	0	0.92	0.79	0.88		2.29	
		% Rate Change						-73	+100	-52	-82	+100	-82	-73	-70	-54	-81	-100	-72	-56	-68		-79	

a/ Assume 2/3 MV for Day and 1/3 MV at night for rate calculations.
"S" Indicates change is significant at the 0.10 level using the Chi-Square Test

problem was almost entirely a multiple vehicle one, and the major portion of this problem again was the right-angle crossing collision.

Regardless of the type of traffic control in operation before the installation of the 4-way red flashers, the severity as well as the accident rates were reduced. The severity was reduced to approximately one-half of that experienced in the before condition.

In the case of the two projects with the 4-way stop signs, there was no problem in the before period (15 accidents in four project years); and, as might be expected, the reduction in accidents was not statistically significant at the 0.10 level. However, the EPDO accidents were reduced and the severity index dropped from 5.0 to 2.5, thus reflecting a reduction of accident severity after the improvement.

Eight-Inch Versus Twelve-Inch Lens—Table 8 indicates that, with the exception of the 3-leg intersection projects, the 12-in. lens caused a greater percent reduction in accident rates than did the 8-in. lens. Greater accident rate reductions with the 12-in. lens were obtained in 4-way red flashing intersections than in the red-yellow intersections, and in nonchannelized intersection than in channelized intersection.

The 4-way red flasher intersections are further broken down in Table 9. Although sufficient projects were not available to make direct comparisons between 8-in. and 12-in. lenses, when prior red-yellow flashers were converted to 4-way red flashers, a greater reduction in accident rates was noted in projects that went from 8-in. lens to 12-in. lens than went from 12-in. red-yellow to 12-in. 4-way red lenses. Since the before rates were approximately the same in both cases, this suggests that lens size may be more important than mode of operation (color).

Fifteen red-yellow 4-leg intersections with prior condition of stop signs facing the minor legs were summarized in Table 10. Multiple vehicle accidents were significantly reduced in both the 8 and 12-in. lens groups but only the 12-in. lens projects

TABLE 8

SUMMARY OF 8-IN. VERSUS 12-IN. LENSES

		ACCIDENT RATES							
		8" LENS				12" LENS			
		No. of Projects	Before	After	Percent Change	No. of Projects	Before	After	Percent Change
RED-YELLOW INTERSECTIONS	Chann.	5	1.98	1.42 S	-28	5	1.29	0.81 S	-37
	Non-Chann.	5	2.47	1.65	-33	6	3.47	1.90 S	-45
	3-Leg	2	6.15	0.74 S	-88	4	1.01	0.68	-33
	4-Leg	8	1.92	1.50 S	-22	7	3.36	1.83 S	-46
Sub Total		10	2.09	1.47 S	-30	11	2.02	1.19 S	-41
4-WAY RED INTERSECTIONS	Chann.	1	1.82	1.38	-24	2	2.97	1.13 S	-62
	Non-Chann.	1	0.94	0.69	-27	4	3.70	0.65 S	-82
Sub Total		2	1.39	1.03	-26	6	3.48	0.80 S	-77
Total Intersections		12	1.99	1.41 S	-29	17	2.45	1.09 S	-56

"S" Indicates change is significant at the 0.10 level using the Chi-Square Test.

TABLE 9

REDUCTION IN RATES WHEN CONVERTING TO 4-WAY RED FLASHERS

Before Condition	Accident Rate	No. of Projects	After Condition	Accident Rate	% Change Rates, B to A
2-way stop signs	2.75	3	12-in. lens	0.94	-66
4-way stop signs	1.39	2	8-in. lens	1.03	-26
8-in. red-yellow center-suspended flashers	4.24	2	12-in. lens	0.33	-92
12-in. red-yellow center-suspended flashers	4.38	1	12-in. lens	2.22	-49
Subtotal red-yellow center-suspended	4.26	3	12-in. lens	0.65	-85
Total	2.77	8	4-way red flashers	0.88	-68

had significant reductions in crossing accidents. Small reductions were obtained in rear-end accidents in both groups. PDO and injury accidents were also significantly reduced in the 12-in. lens group, and although there were accident reductions in these severity groups for the 8-in. lens group, they were not statistically significant.

Night accidents were significantly reduced in both groups although there was a substantially greater reduction in accident rate for the 12-in. group. The 12-in. lens projects also had a significant reduction in daytime accidents with a three times greater percentage reduction in the accident rate over the 8-in. lens projects.

Investigation of Warrants for Intersection Flashing Beacons—An investigation was made to determine the best criteria for establishing warrants for 4-leg flashing beacons. Various arrays were made to determine dependent relationships with accidents, accident rates or EPDO accidents.

It was determined in the 4-way red intersections that the minor leg to major leg entering volume ratio varied from 0.61 to 1.00 with a mean of 0.70 for the eight projects studied. In the 15 red-yellow intersections, the ratio varied from 0.14 to 0.98 with a mean of 0.35.

Five red-yellow intersections had ratios of over 0.05. All of these projects had increases in EPDO accidents although only one was significantly worse. After two of these projects were converted to 4-way red operation, accidents were reduced.

Various trial warrants were applied to the red-yellow and 4-way red intersection data (Tables 11 and 12). These were compared to the results of all the projects as constructed. It is necessary to compare only accidents since the vehicular exposure is equal for all warrants. To compare the effects of different warrant criteria on all of the projects, it was necessary to estimate the accident experience in the after period for the "unwarranted" projects. Actual before-and-after data were available for the "warranted" projects. However, only before data were usable for unwarranted projects.

Therefore, the total accident experience in the after periods was composed of the actual warranted after accident experience plus the adjusted accident experience of the unwarranted projects. The after accident experience for unwarranted projects was

TABLE 10

RELATIVE EFFECT OF 8-IN. VS 12-IN. LENS (4-LEG RED-YELLOW)

AFTER LENS SIZE		PRIOR CONDITION 2 WAY STOP SIGNS	PROJECTS: Total No	PROJECTS: Improved	PROJECTS: Worsened	PROJECTS: No Change	Years of Experience	Single Vehicle: Ran off Road	Single Vehicle: Other	Single Vehicle: Sub-Total	Multiple Vehicle: Left Turn	Multiple Vehicle: Rear End	Multiple Vehicle: Crossing	Multiple Vehicle: Other	Multiple Vehicle: Sub-Total	Severity: PDO	Severity: Injury	Severity: Fatal	LT Cond.: Day a/	LT Cond.: Night a/	Total Accidents	Million Vehicles	Equivalent PDO (EPDO)	Severity Index (SI)
8"	Before	No of Accidents	8				18 6/12	2	3	5	15	18	77	5	115	82	29	9	78	42	120	62.6	348	2.9
		Rate						0.03	0.05	0.08	0.24	0.29	1.23	0.08	1.84	1.31	0.46	0.14	1.87	2.01	1.92		5.56	
	After	No of Accidents	8 S	2		6	18 6/12	5	0	5	15	13	62	6	96 S	72	25	4	72	29 S	101 S	67.2	275	2.7
		Rate						0.07	0	0.07	0.22	0.19	0.92	0.09	1.43	1.07	0.37	0.06	1.61	1.29	1.50		4.09	
		% Rate Change						+133	-100	-12	-8	-34	-25	+12	-22	-18	-20	-57	-14	-36	-22		-26	
12"	Before	No of Accidents	7				7 9/12	1	0	1	9	6	52	5	72	31	40	2	50	23	73	21.7	325	4.5
		Rate						0.05	0	0.05	0.41	0.28	2.40	0.23	3.32	1.43	1.84	0.09	3.45	3.20	3.36		14.98	
	After	No of Accidents	7	1 S		6	7 9/12	0	0	0	9	1	32 S	2	44 S	19 S	24 S	1	31 S	13 S	44 S	24.1	1.94	4.4
		Rate						0	0	0	0.37	0.04	1.33	0.08	1.83	0.79	1.00	0.04	1.93	1.63	1.83		8.05	
		% Rate Change						-100	0	-100	-10	-86	-45	-65	-45	-45	-46	-56	-44	-49	-46		-46	
TOTAL	Before	No of Accidents	15				26 3/12	3	3	6	24	24	129	10	187	113	69	11	128	65	193	84.3	673	3.5
		Rate						0.04	0.04	0.07	0.28	0.28	1.53	0.12	2.22	1.34	0.82	0.13	2.28	2.31	2.29		7.98	
	After	No of Accidents	15	3 S		12	26 3/12	5	0	5	24	14 S	94 S	8	140 S	91 S	49 S	5	103 S	42 S	145 S	91.3	469	3.2
		Rate						0.05	0	0.05	0.26	0.15	1.03	0.09	1.53	0.99	0.54	0.05	1.69	1.38	1.59		5.14	
		% Rate Change						+25	-100	-29	-7	-46	-33	-25	-31	-25	-34	-62	-26	-40	-31		-36	

a/ Assume 2.3 MV for Day and 1.3 MV at night for rate calculations.
"S" Indicates change is significant at the 0.10 level using the Chi-Square Test

obtained by adjusting the before experience in the ratio of after exposure to before exposure assuming that the accident rate did not change.

The accidents given in Tables 11 and 12 are total accidents. The projects declared unwarranted, however, are removed on the basis of specific number of accidents per year. The warranted projects meet the required number of accidents per year for that warrant. Since the before and after periods are equal for all projects it is not necessary to compare accidents on a per year basis, in fact by using the total periods, the changes are greater—thus more sensitive.

Possible warrants for red-yellow installations are given in Table 11. When an average of two-crossing, two-crossing-plus-left-turn, or three-crossing-plus-left-turn accidents is used in conjunction with the minor to major volume ratio of 0.50 or less in the before period, six projects not meeting these warrants were removed. Five of these are above the 0.50 ratio and the other one did not have enough crossing-plus-left-turn accidents in the before period to fall in this warranted group. These warrants

TABLE 11

EVALUATION OF VARIOUS WARRANTS 4-LEG RED-YELLOW

WARRANT DESCRIPTION	NUMBER OF PROJECTS WARRANTED	BEFORE: LEFT TURN	BEFORE: XING	BEFORE: PDO	BEFORE: INJ	BEFORE: FAT	BEFORE: TOTAL	BEFORE: EPDO	AFTER (Estimated): LEFT TURN	AFTER (Estimated): XING	AFTER (Estimated): PDO	AFTER (Estimated): INJ	AFTER (Estimated): FAT	AFTER (Estimated): TOTAL	AFTER (Estimated): EPDO	ESTIMATED REDUCTION: LEFT TURN	ESTIMATED REDUCTION: XING	ESTIMATED REDUCTION: TOTAL	ESTIMATED REDUCTION: EPDO
All projects	15	24	129	113	69	11	193	673	24	94	91	49	5	145	469	0	35	48	204
2 Xing Acc/year	13	24	129	113	69	11	193	673	24	89	86	50	6	142	478	0	40	51	195
3 Xing Acc/year	12	24	129	113	69	11	193	673	23	96	96	54	5	155	509	1	33	38	164
4 Xing Acc/year	10	24	129	113	69	11	193	673	23	106	96	61	9	166	586	1	23	27	87
2 Xing Acc/year Minor/Major Vol. ≤ 0.50	9	24	129	113	69	11	193	673	21	99	90	45	6	141	447	3	30	52	226
2 Xing + Lt. turn acc/yr. Minor/Major Vol. ≤ 0.50	9	24	129	113	69	11	193	673	21	99	90	45	6	141	447	3	30	52	226
3 Xing + Lt. turn acc/yr. Minor/Major Vol. ≤ 0.50	9	24	129	113	69	11	193	673	21	99	90	45	6	141	447	3	30	52	226
4 Xing + Lt. turn acc/yr. Minor Ent. Vol. is ≤ 0.50 of Major Ent. Vol.	7	24	129	113	69	11	193	673	21	109	99	53	8	160	526	3	20	33	147

NOTE: The "after" accident experience for "unwarranted" projects was estimated by adjusting the "before" experience in the ratio of "after" exposure (MV), to "before" exposure (MV). (Assumes no change in accident rate.)

TABLE 12

EVALUATION OF VARIOUS WARRANTS 4-LEG RED

WARRANT DESCRIPTION		BEFORE							AFTER (Estimated)							ESTIMATED REDUCTION			
		LEFT TURN	XING	PDO	INJ	FAT	TOTAL	EPDO	LEFT TURN	XING	PDO	INJ	FAT	TOTAL	EPDO	LEFT TURN	XING	TOTAL	EPDO
All Projects	8	16	52	44	38	6	88	352	3	10	22	8	0	30	78	13	42	58	274
2 Xing Acc/year $\frac{\text{Minor}}{\text{Major}}$ Vol.>0.50	5	16	52	44	38	6	88	352	9	13	21	17	2	40	154	7	39	48	198
2 Xing + Lt. turn accs Minor Ent. Vol.> .50 of Major Ent. Vol.	7	16	52	44	38	6	88	352	3	12	19	12	0	31	103	13	40	57	249
3 Xing + Lt. turn accs. Minor Ent. Vol>0.50 of Major Ent. Vol.	6	16	52	44	38	6	88	352	7	12	18	13	2	33	123	9	40	55	229
4 Xing + Lt. turn accs. & Minor Ent. Vol.>0.50 of Major Ent. Vol.	6	16	52	44	38	6	88	352	7	12	18	13	2	33	123	9	40	55	229

NOTE: The "after" accident experience for "Unwarranted" projects was estimated by adjusting the "before" experience in the ratio of "after" exposure (MV) to "before" exposure (MV). (Assumes no change in accident rate.)

appear to reduce approximately the same number of left-turn-plus-crossing accidents with a greater number of EPDO accidents reduced than those of the total number of projects studied. These reductions are accomplished with only nine of the 15 projects warranted. These same benefits would be accrued for only 60 percent of the expenditure of funds.

Possible warrants for a 4-way red intersection are given in Table 12. A warrant of two-crossing-plus-left-turn accidents per year has approximately the same number of accidents reduced and 25 less EPDO accidents reduced as all eight projects. With six projects meeting the warrant of three-crossing-plus-left-turn accidents per year, the total number of accidents reduced is still about the same with 45 EPDO less accidents reduced. The following warrants are indicated:

1. Flashing beacons shall be considered at 4-leg intersection locations which experience four or more left-turn-plus-crossing (broadside) accidents in one year; or
2. Six or more left-turn-plus-crossing accidents in two years (a small separate study was made of 100 intersections with three accidents in the first year; only 40 percent of these had a 2-year average of three accidents per year, whereas 66 percent of the 100 intersections examined having four accidents in the first year had a 2-yr average of three accidents per year).

If the above criteria have been met, the type of control at the intersection can be determined from the following:

1. If the minor to major entering volume ratio is 0.50 or less, red-yellow lens operation (2-way stop) should be considered.
2. If the minor to major entering volume ratio is greater than 0.50, 4-way red lens operation (4-way stop) should be considered.

It was felt that insufficient data were available for trial warrant analysis for 3-leg intersections and that additional projects are needed for further study.

Summary of Intersection Beacons—In summary, it can be said that flashing signals at intersections are very effective in reducing multiple-vehicle accidents, especially of the right-angle crossing collision type. In addition to causing large reductions in accident rates, these flashers are effective in reducing the severity of the accident.

Four-leg intersection data indicate that greater accident reductions can be expected by using a 12-in. lens with only a small increase in cost ($20 to $25 per flasher or approximately $100 per 4-leg intersection).

In the case of flashers installed at 3-leg intersections, the flashers are also effective in reducing the number of vehicles that run through the intersection from the minor road and off the highway where there is no continuation of the road. It appears that 8-in. lens size may be adequate for 3-leg intersections, although the sample available was too small to be conclusive. Since the additional cost is small, 12-in. lens installations are recommended for both 4-leg and 3-leg intersections.

Railroad Crossing Flashers—At-grade railroad crossings have been protected by a variety of devices including crossing signs, No. 8 automatic flashing red lights, and automatic gates in conjunction with the flashing lights.

Reports were available for only three projects of at-grade crossings with railroads (Table 13). The installations consisted of standard PUC No. 8 automatic flashing lights, X-bucks and bells. The total number of accidents was significantly reduced with no significant increase in nontrain accidents. EPDO accidents were also reduced thus indicating a reduction in severity. The major problem was vehicle-train accidents. These were reduced 100 percent. Daytime accidents as well as nighttime accidents were reduced.

TABLE 13

RAILROAD CROSSING FLASHING LIGHTS—STANDARD PUC NO. 8
(No. Projects 3, Improved 2*, Worsened 0, No Change 1)

Accidents	Before		After		
	No. of Accidents	Rate	No. of Accidents	Rate	% Change in Rate
PDO	6	0.57	0*	0	-100
Injury	5	0.48	2	0.15	- 69
Fatal	1	0.10	0	0	-100
Day[a]	6	0.86	1*	0.11	- 87
Night	6	1.72	1*	0.22	- 87
Involving train	11	1.05	0*	0	-100
Rear-end	0	0	2	0.15	∞
Total	12	1.14	2*	0.15	- 87
Experience (MV)		10.5		13.3	
Total No. years		7-6/12		7-6/12	

[a]Assume 2/3 MV Day and 1/3 MV at night.
*Change is significant at the 0.10 level.

The Public Utilities Commission published a before-and-after report (5) in 1965 involving 278 at-grade railroad crossings at which two standard No. 8 automatic flashing lights were installed. Three years' experience was obtained in the before and after periods in which total accidents dropped from 521, to 112, a 79 percent reduction.

In summary, railroad flashers were quite effective in reducing vehicle-train accidents with very small increases in rear-end accidents.

Advanced Warning Flashing Beacons

Table 14 summarizes the accident data for 12 minor improvement projects in which flashing beacons were placed in conjunction with an advance warning sign. Such signs are used to call attention to a location ahead where a hazard exists to free traffic flow.

TABLE 14

ADVANCE WARNING FLASHING BEACON SUMMARY

Flashers Prior To			Projects: Total No	Projects: Improved	Projects: Worsened	Projects: No. Change	Years of Experience	Single Vehicle: Ran off Road	Single Vehicle: Other	Single Vehicle: Sub-Total	Multiple Vehicle: Left Turn	Multiple Vehicle: Rear End	Multiple Vehicle: Crossing	Multiple Vehicle: Other	Multiple Vehicle: Sub-Total	Severity: PDO	Severity: Injury	Severity: Fatal	Lt. Cond.: Day a/	Lt. Cond.: Night a/	Total Accidents	Million Vehicles	Equivalent PDO (EPDO)	Severity Index (SI)
CURVE	Before	No. of Accidents					6	26	3	29	0	1	0	4	5	20	13	1	12	22	34	17.9	118	3.5
		Rate						1.45	0.17	1.62	0	0.06	0	0.22	0.28	1.12	0.72	0.06	1.01	3.69	1.90		6.59	
	After	No. of Accidents	4	2 S		2	6	12 S	0	12 S	0	3	0	2	5	10 S	7	0	8	9 S	17 S	19.4	59	3.5
		Rate						0.62	0	0.62	0	0.16	0	0.10	0.26	0.52	0.36	0	0.62	1.39	0.88		3.04	
		% Rate Change						-57	-100	-62	0	+167	0	-55	-7	-54	-50	-100	-39	-62	-54		-54	
INTERSECTION	Before	No. of Accidents					7	Not	Available							66	18	2	53	33	86	75.6	206	2.4
		Rate														0.87	0.24	0.03	1.05	1.31	1.14		2.72	
	After	No. of Accidents	5	1 S		4	7									39 S	28	0	45	22	67 S	77.1	235	3.5
		Rate														0.51	0.36	0	0.88	0.86	0.87		3.05	
		% Rate Change														-41	+50	-100	-16	-34	-24		+12	
SCHOOL	Before	No. of Accidents					6	1	3	4	0	2	3	1	6	8	2	0	9	1	10	26.0	22	2.2
		Rate						0.04	0.12	0.15	0	0.08	0.12	0.04	0.23	0.31	0.08	0	0.52	0.12	0.38		0.85	
	After	No. of Accidents	3			3	6	1	0	1	0	8	2	0	10	10	1	0	8	3	11	29.8	17	1.5
		Rate						0.03	0	0.03	0	0.27	0.07	0	0.34	0.34	0.03	0	0.40	0.30	0.37		0.57	
		% Rate Change						-25	-100	-80	0	+238	-42	-100	+48	+10	-63	0	-23	+150	-3		-33	
TOTAL	Before	No. of Accidents					19	27	6	33	0	3	3	5	11	94	33	3	74	56	130	119.5	346	2.7
		Rate						0.23	0.05	0.28	0	0.03	0.03	0.04	0.09	0.79	0.28	0.03	0.93	1.40	1.09		2.89	
	After	No. of Accidents	12	3 S		9	19	13 S	0 S	13 S	0	11 S	2	2	15	59 S	36	0	61 S	34 S	95 S	126.3	311	3.3
		Rate						0.10	0	0.10	0	0.09	0.02	0.02	0.12	0.47	0.29	0	0.72	0.81	0.75		2.46	
		% Rate Change						-56	-100	-64		+200	-33	-50	+33	-41	+4	-100	-23	-42	-31		-15	

a/ Assume 2/3 MV for Day and 1/3 MV at night for rate calculations.
"S" Indicates change is significant at the 0.10 level using the Chi-Square Test

The installation of a flashing yellow beacon may presently be warranted as an advance warning device for an intersection or other location under one or more of the following conditions:

1. Physical obstruction existing in the roadway or reduction in width.
2. Important intersection hidden by an obstruction or sharp curve in the highway.
3. Potentially hazardous horizontal or vertical alignment.
4. On the approach to a signalized intersection where the signal is unexpected.

The 12 projects represent approximately 19 years of experience in each of the before and after periods. Of the 12 projects, three showed a significant improvement based on the total number of accidents and four showed improvements based on the total number of EPDO's. Advance warning flashers as a class effected a significant reduction in accidents. Both the accident rate and EPDO rate were reduced after the improvement.

At curves, the problem was primarily a single-vehicle one—mainly ran-off-road accidents which were reduced significantly. Rear-end accidents increased significantly, with most of the accidents at schools. The accidents were primarily PDO's which showed a significant reduction. Both day and night accidents were significantly reduced with a greater accident rate reduction at night.

Intersection Advance Warning Flashers—Flashing beacons were placed on five projects to warn drivers of an intersection condition ahead. These have been placed where there is a hidden intersection at the end of a long tangent condition, or at the first signal into town to ease the transition from rural expressway to urban conditions. Total accidents were significantly reduced and EPDO accidents were increased. Single-vehicle and multiple-vehicle accident breakdowns were not made because this detail was not always available. The number of PDO accidents was significantly reduced. All other severity classes, and night and day accidents, showed no significant change.

Three of these projects had mast-arm mounted SIGNAL AHEAD signs, only one of which had a significant accident reduction. This project had a sight distance restriction. Accidents were reduced (not significant) on the second project and the EPDO's increased somewhat. The third project had an accident increase (not significant) and an EPDO accident increase after the improvement.

The fourth project was a T-intersection with a STOP AHEAD sign and flashers placed on the stem prior to the stop sign. Total accidents were not significantly reduced with a decrease of EPDO accidents, thus indicating a reduction in severity (SI was reduced from 7 to 1).

The fifth project was a mast-arm mounted CROSS TRAFFIC AHEAD sign with two flashers. There was no significant change in the total accidents. EPDO accidents remained about the same also. The severity was reduced somewhat (SI dropped from 4 to 3).

School Flashers—Flashers were in operation only during school hours on three projects to warn drivers of the school crossing. The beacons were placed 200 to 500 ft prior to the school crossing in each direction. The total number of accidents and EPDO accidents remained unchanged after the improvement. Increases were noted in multiple-vehicle accidents (rear-end, PDO accidents).

No school children were struck in either the before or after periods. Four rear-end accidents were initiated by children or pedestrians in the crosswalk in the after period. Three of these involved children and an adult crossing guard and the other involved two adult pedestrians (no crossing guard).

One of these projects showed an accident reduction from 8 to 4. This project has two flashers mast-arm mounted, in conjunction with a SCHOOL CROSSING sign placed in advance of the crossing in each direction. The other two projects had yellow flashers mounted on a 10-ft steel pole above a SCHOOL CROSSING sign prior to the crossings. Accidents increased from 1 to 4 and 1 to 3 after the installation of the flashers. The increases were not significant and could have occurred by mere chance.

These projects were requested by local school authorities and fulfilled very few or no engineering warrants for the improvement (there were only 10 accidents in 6 project years). The flashers were installed as required by law primarily in the interest of improved public relations when the school district was willing to pay half the cost.

Curve Warning Flashers—Flashers located prior to hazardous curves have been effective in significantly reducing accidents with a greater reduction in night accident rates than daytime (night accidents were significantly reduced). EPDO accidents were also reduced. The major before problem and the category showing the greatest improvement was the single-vehicle ran-off-road accident.

Bridge Approach Flares—Flashing beacons have been tried on sharp curves at the approach to narrow bridges at two locations. One of the projects experienced an increase in accidents from 22 to 27 with the accident rate going from 2.7 to 2.8 accidents per MV. The other project had a reduction in accidents from 3 to 0 with the accident rate reduced from 1.4 to 0. This project also had a reduction of equivalent PDO accidents which dropped from 9 to 0. The average accident rate for the two projects dropped slightly from 2.4 acc/MV to 2.3 acc/MV with 10.5 million vehicle exposure before and 11.7 MV after installing the flashers.

Summary of Advanced Warning Beacons—In conclusion, it can be said that advanced warning beacons are effective in reducing single-vehicle accidents of the ran-off-the-road variety. Greater accident reductions are realized at nighttime, although daytime accidents were also reduced. Rear-end accident increases were noted with the school flashers, which had otherwise little or no change in the accident experience after the improvement. The number of projects of each type were considered insufficient for any analyses for future warrants. The flashers in conjunction with SIGNAL AHEAD signs in particular should be further evaluated.

SAFETY LIGHTING IMPROVEMENTS

The purpose of highway lighting is to safeguard and facilitate both vehicular and pedestrian traffic at night by illuminating certain permanent features at locations which require additional care and alertness. It is expected that, with illumination, these features will be more readily comprehended and compensated for by the motorist.

This report concerns isolated lighting at spot or discontinuous locations. It does not evaluate the effect of continuous illumination. An evaluation of continuous illumination was previously reported (6).

Presently lighting is warranted in California at existing expressway and conventional highway intersections if one of the following conditions (7) is fulfilled:

1. A minimum vehicular volume, an interruption of continuous traffic or minimum pedestrian volume traffic signal warrant (see Appendix) is satisfied based on any single hour count which may be in darkness in winter months.
2. There are five or more accidents a year and 50 percent or more are occurring under conditions other than daylight.
3. Less than five accidents occur per year at any one location, with three or more accidents per year under conditions other than daylight.

The number of luminaires required for an intersection is dependent upon the area of the intersection. The California Planning Manual (7) requires a minimum of 0.2 horizontal foot-candles of illumination for the area bounded by the crosswalks, and a minimum of 0.8 horizontal foot-candles at the intersection of the centerlines of the entering streets. Figure 2 shows the minimum specification for a 20,000-lumen mercury vapor luminaire. Figures 3, 4 and 5 indicate typical installations of luminaires.

Lighting is installed either at State expense (or as a cooperative project with local agencies at existing intersections) or by a service agreement with a utility company whereby a monthly service charge is paid to cover installation and power costs. The cost of utility company installations varies from one location to another and ranges from \$5 to \$10 a month for a 20,000-lumen mercury vapor luminaire. The cost of a State-installed luminaire is approximately \$1,000 installation plus \$3 a month for power costs.

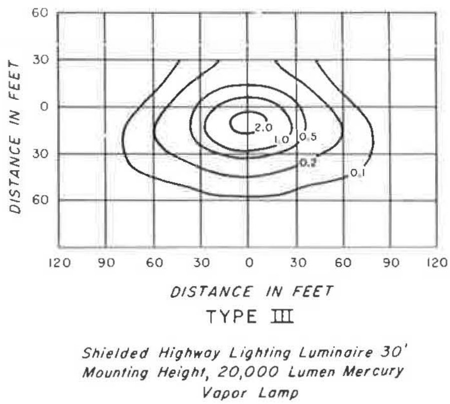


Figure 2. Isolux lines of minimum horizontal foot-candles.

Methodology

There were 47 safety lighting project evaluations (none of which were signalized intersections) available for analysis. However, six of these reports were not used because pertinent data were lacking. The remaining 41 projects consisted of 26 intersection lighting projects, four upgraded intersection lighting projects, six railroad crossing lighting projects, two bridge approach lighting projects, and three underpass lighting projects. The total before and after accident experience for the 41 projects was examined. However, the analysis was, of course, principally concerned with the evaluation of the reduction in the nighttime accident experience although changes in day and night accidents were also examined.

All Lighting Projects

Of the 41 projects used, 21 were warranted under the existing accident criteria (more than 5 accidents with more than 50 percent at night; or less than 5 accidents with 3 or more at night) for safety lighting installations. Even though almost half the projects were not warranted by accidents—some projects were warranted by other considerations (see Appendix)—the program of safety lighting installation has been effective in reducing accidents (Table 15). The 41 locations experienced a 36 percent reduction in the total accident rate and a 63 percent reduction in the night accident rate. However, the warranted projects exhibited over twice the nighttime percentage accident rate reduction as the unwarranted projects.

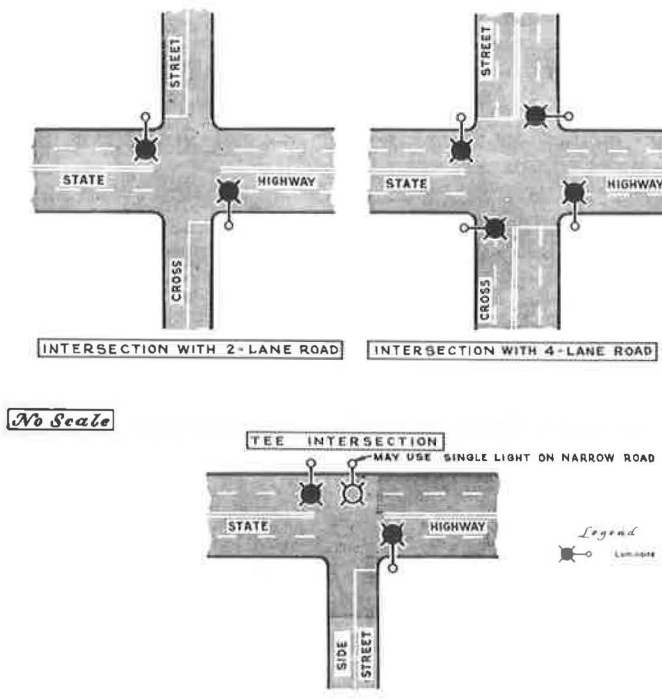


Figure 3. Intersection lighting—nonchannelized intersections.

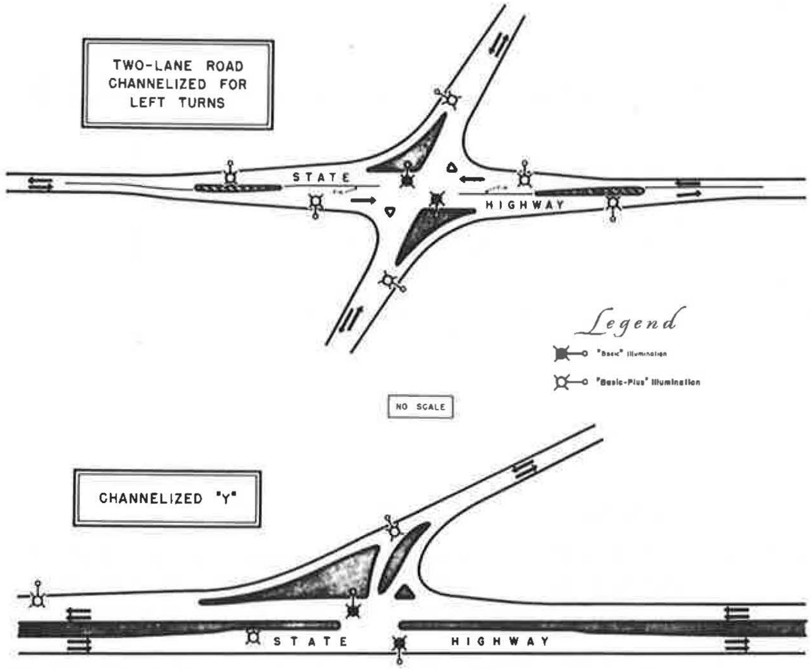


Figure 4. Intersection lighting—channelized intersections.

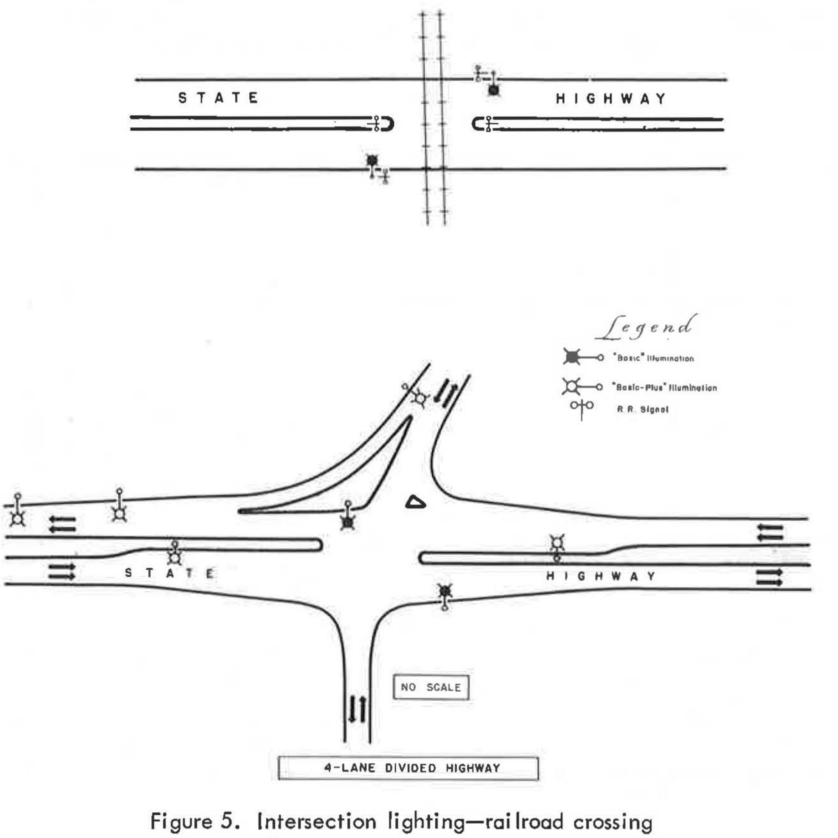


Figure 5. Intersection lighting—railroad crossing

TABLE 15

SUMMARY OF BEFORE AND AFTER ACCIDENT EXPERIENCE FOR ALL LIGHTING PROJECTS

	Number of Projects	Before Experience – Accidents – Day	Before – Accidents – Night	Before – Accidents – Total	Before – M.V.	Before – Accident Rate	Before – Night Acc. Rate b/	After Experience – Accidents – Day	After – Accidents – Night	After – Accidents – Total	After – M.V.	After – Accident Rate	After – Night Acc. Rate b/	Percent Change – Accidents – Day	Percent Change – Accidents – Night	Percent Change – Accidents – Total	Percent Change – M.V.	Percent Change – Accident Rate	Percent Change – Night Acc. Rate
Warranted Projects a/	21	59	145	204	94.8	2.15	4.59	57	43 S	100 S	100.8	0.99	1.28	-3	-70	-51	+6	-54	-72
Non-warranted Projects	20	98	58	156	116.5	1.34	1.49	115	42 S	157	136.4	1.15	0.92	+17	-28	+1	+17	-14	-38
TOTAL	41	157	203	360	211.3	1.70	2.88	172	85 S	257 S	237.2	1.08	1.08	+10	-58	-29	+12	-36	-63

a/ Warranted under present accident criteria of more than 5 accidents per year with more than 50% at night or less than 5 accidents with 3 or more at night.

b/ Assume 1/3 MV at night for rate calculations.

"S" Indicates change is significant at the 0.10 level using the Chi-Square Test

New Lighting at Intersections

Highway intersection lighting involved 26 of the projects analyzed. The intersection lighting was effective in reducing the total accident rate by 39 percent and the night accident rate by 64 percent. No significant improvement in the average severity at night was noted.

Table 16 indicates the before and after night accident experience for intersection lighting projects by type of location. At 3-leg intersections, 74 percent of the reduction in the night accident rate was attributed to the reduction in single-vehicle accidents (the primary type of single-vehicle accident involved proceeding straight ahead on the dead-ended leg). At 4-leg intersections, 70 percent of the reduction in the night accident rate was attributed to the reduction in crossing accidents.

TABLE 16

BEFORE AND AFTER NIGHT ACCIDENT EXPERIENCE FOR INTERSECTION LIGHTING PROJECTS

Location	Period		Projects – Total No	Projects – Improved	Projects – Worsened	Projects – No Change	Years of Experience	Single Vehicle – Ran off Road	Single Vehicle – Other	Single Vehicle – Sub-Total	Multiple Vehicle – Left Turn	Multiple Vehicle – Rear End	Multiple Vehicle – Crossing	Multiple Vehicle – Other	Multiple Vehicle – Sub-Total	Severity – PDO	Severity – Injury	Severity – Fatal	Total Accidents	Million Vehicles	Equivalent PDO (EPDO)	Severity Index (SI)
3 Leg	Before	No of Accidents					15	38	7	45	3	8	9	10	30	39	34	2	75	50.9	291	3.88
		Rate						0.75	0.13	0.88	0.05	0.16	0.18	0.20	0.59	0.76	0.67	0.04	1.47		5.63	
	After	No of Accidents	10	6 S	0	4	15	8 S	0 S	8 S	4	3	10	2 S	19 S	16 S	11 S	0	27 S	60.2	93 S	3.44
		Rate						0.13	0.00	0.13	0.07	0.05	0.17	0.03	0.32	0.26	0.19	0.00	0.45		1.54	
		% Rate Change						-83	-100	-85	+40	-69	-6	-85	-47	-66	-72	-100	-69		-73	
4 Leg 2 Lane	Before	No of Accidents					15	2	0	2	0	1	14	8	23	14	8	3	25	25.1	91	3.64
		Rate						0.08	0.00	0.08	0.00	0.04	0.56	0.32	0.92	0.56	0.32	0.12	1.00		3.63	
	After	No of Accidents	7	1 S	0	6	15	3	1	4	1	2	7 S	1 S	11 S	9	6	0	15 S	32.1	51 S	3.41
		Rate						0.09	0.03	0.13	0.03	0.06	0.22	0.03	0.34	0.28	0.19	0.00	0.47		1.59	
		% Rate Change						-12	∞	+62	∞	+50	-61	-91	-63	-50	-41	-100	-53		-56	
4 Leg 4 Lane	Before	No of Accidents					10	3	2	5	8	1	18	1	28	17	15	1	33	62.5	129	3.92
		Rate						0.05	0.03	0.08	0.73	0.02	0.29	0.02	0.46	0.27	0.24	0.02	0.53		2.07	
	After	No of Accidents	9	1 S	0	8	10	4	1	5	5	0	3 S	0	8 S	6 S	7	0	13 S	65.3	55	4.24
		Rate						0.06	0.02	0.08	0.08	0.00	0.05	0.02	0.12	0.09	0.11	0.00	0.20		0.84	
		% Rate Change						+20	-33	0	-38	-100	-83	0	-74		-54	-100	-62		-60	
TOTAL	Before	No of Accidents					40	43	9	52	11	10	41	19	81	70	57	6	133	138.5	511	3.85
		Rate						0.31	0.07	0.38	0.08	0.07	0.30	0.14	0.59	0.51	0.41	0.05	0.97		3.66	
	After	No. of Accidents	26	8 S	0	18	40	15 S	2 S	17 S	10	5	20 S	3 S	38 S	31 S	24 S	0 S	55 S	157.6	199 S	3.62
		Rate						0.10	0.01	0.11	0.06	0.03	0.13	0.02	0.24	0.20	0.15	0.00	0.35		1.26	
		% Rate Change						-68	-86	-71	-25	-57	-57	-86	-60	-61	-63	-100	-64		-66	

a/ Assume 2/3 MV for Day and 1/3 MV at night for rate calculations.

"S" Indicates change is significant at the 0.10 level using the Chi-Square Test

TABLE 17

UPGRADED LIGHTING AT URBAN INTERSECTION BEFORE AND AFTER ACCIDENT EXPERIENCE

	ACCIDENTS			M.V.	TOTAL ACCIDENT RATE	NIGHT ACCIDENT RATE a/
	DAY	NIGHT	TOTAL			
Before	25	15	40	33.6	1.19	1.34
After	30	7 S	37	40.5	0.91	0.52
% Change	+20	-53	-8	+20	-24	-61

a/ Assume 1/3 MV at night for rate calculations

"S" Indicates change is significant at the 0.10 level using the Chi-Square Test

Four-leg intersection projects were subdivided into two classes; one for those locations where the mainline was four lanes and the other for two-lane locations (minor road was two lanes in all cases). No significant differences were noted between the two subclasses.

Because of insufficient data, a detailed analysis of the effect of the number of luminaires installed was not possible. No discernible difference in the effectiveness of night accident reduction was found for using more than one luminaire. This, however, does not rule out the possibility of needing more than one luminaire for a given intersection geometry. In general, more luminaires should be considered as the area and complexity of the intersection increases (e. g., a 3-leg, 2-lane by 2-lane intersection probably needs only one luminaire, but a 4-leg, 4-lane by 4-lane channelized intersection would probably benefit more by the use of four luminaires).

Upgraded Urban Intersection Lighting

The analysis of improved lighting at four downtown high-volume (averaging 25,000 ADT) intersections from obsolete low-intensity lighting to 20,000-lumen mercury vapor luminaires indicates that equipment modernization can effectively reduce the night accident experience. The night accident rate was reduced by 62 percent (Table 17).

Lighting at Railroad Crossings

The analysis of six railroad crossing lighting projects shows the effectiveness of lighting in reducing high nighttime single-vehicle accident rates at locations which have unexpected reduced alignment standards (e. g., small radius reversing curves). At these six locations, the highway which was parallel to the railroad crossed from one side of the railroad to the other through sharp reversing curves. In each case, an extension of the road continued on as a secondary road forming Y-intersections at each side of the railroad crossing (Fig. 6). Most of the accidents were single-vehicle nighttime accidents in which the vehicles approaching the reversing curves were overrunning the first curve and running off the minor road.

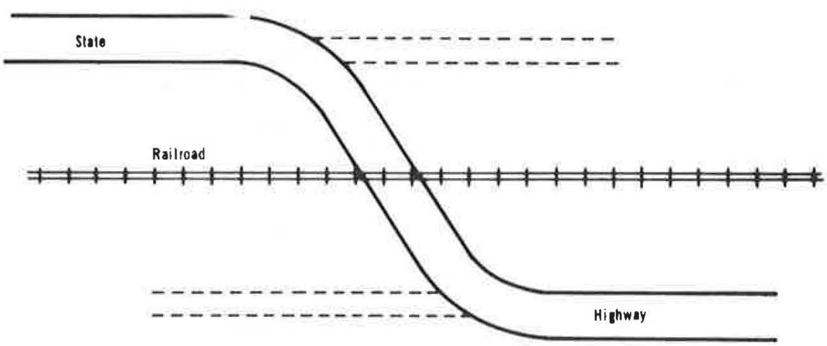


Figure 6. Typical reverse curve at railroad crossing.

Because of a predominance of nighttime single-car accidents, it was apparent that better delineation and/or illumination was needed to identify the curves. Previously, all of these crossings were protected by standard flashing railroad crossing lights and reverse curve signs. In addition, five of the Y-locations had continuous flashing yellow beacons. The illumination at these six railroad crossings was effective in reducing the night accident rate by 65 percent (Table 18).

Bridge Approach Lighting

Two projects involved safety lighting at bridge approaches. These locations are similar to the railroad crossing in that the roadway across the bridges has an unexpected reduced alignment standard. The safety lighting installed at these two locations was effective in reducing the night accident rate by 55 percent (Table 19).

TABLE 18

LIGHTING AT RAILROAD CROSSING BEFORE AND AFTER ACCIDENT EXPERIENCE

	ACCIDENTS					M.V.	TOTAL ACCIDENT RATE	NIGHT a/ ACCIDENT RATE
	SINGLE VEHICLE	MULTIPLE VEHICLE	DAY	NIGHT	TOTAL			
Before	47	2	12	37	49	6.3	7.80	17.62
After	27	0	12	15 S	27 S	7.2	3.75	6.25
% Change	-43	-100	0	-60	-45	+14	-52	-65

a/ Assume 1/3 MV at night for rate calculations
"S" Indicates change is significant at the 0.10 level using the Chi-Square Test

TABLE 19

BRIDGE APPROACH LIGHTING BEFORE AND AFTER ACCIDENT EXPERIENCE

	ACCIDENTS					M.V.	TOTAL ACCIDENT RATE	NIGHT a/ ACCIDENT RATE
	SINGLE VEHICLE	MULTIPLE VEHICLE	DAY	NIGHT	TOTAL			
Before	15	3	7	11	18	3.2	5.63	10.00
After	6 S	7	8	5	13	3.2	4.07	4.55
% Change	-73	+133	+14	-55	-28	-	-28	-55

a/ Assume 1/3 MV at night for rate calculations
"S" Indicates the change is significant at the 0.10 level using the Chi-Square Test.

TABLE 20

UNDERPASS LIGHTING BEFORE AND AFTER ACCIDENT EXPERIENCE

	ACCIDENTS			M.V.	TOTAL ACCIDENT RATE	NIGHT a/ ACCIDENT RATE
	DAY	NIGHT	TOTAL			
Before	12	7	19	29.7	0.64	0.71
After	15	7	22	33.9	0.65	0.62
% Change	+25	0	+16	+14	+0.02	-13

a/ Assume 1/3 MV at night for rate calculations
"S" Indicates the change is significant at the 0.10 level using the Chi-Square Test.

Underpass Lighting

There were three projects involving the lighting of underpasses. These locations did not indicate a bad night accident experience in the before period (Table 20). Consequently, no significant reduction was noted in the night accident rates.

Warrants for Safety Lighting

Because complete accident data were available only for intersection lighting projects, these projects were chosen as a base for investigating safety lighting warrants. Table 21 gives ten different possible criteria for installing safety lighting, indicating the number of projects which would qualify under the given criteria and the net benefit in night accident reduction.

The tabulations do not have a common basis for comparison. Therefore, the evaluation of the best criteria was accomplished by investigating the effect of the various criteria on all projects. In this method, the after period is evaluated by adding the accident experience for the after period of the projects warranted to the estimated after accident experience for the projects not warranted. The accident experience for the after period of the projects not warranted was estimated by assuming no change in the before accident rate and no change in the before percentages of accidents by severity class. Table 22 gives the total improvements for the ten different warrants. Because the vehicular exposure is equal for all warrants, it is necessary to compare only the accidents per se.

In Table 22, only 15 of the 26 projects met the present accident warrants in the before period. As explained previously, the before period used in this analysis did not necessarily correspond to the before period which initiated the improvement. In the initial before periods of the 11 unwarranted projects, 4 met the accident or accident and volume warrants, 3 met volume warrants, 2 had less than a year's experience but had already

TABLE 21

NET REDUCTION IN NIGHT ACCIDENTS FOR VARIOUS WARRANTS AT INTERSECTIONS

WARRANT DESCRIPTION	NO. OF PROJECTS WARRANTED	BEFORE								AFTER								REDUCTION						
		PDO	INJ	FAT	TOT	MV	RATE	EPDO	EPDO RATE	PDO	INJ	FAT	TOT	MV	RATE	EPDO	EPDO RATE	PDO	INJ	FAT	TOT	RATE	EPDO	EPDO RATE
All Projects	26	70	57	6	133	138.5	0.96	511	3.70	31	24	0	55	157.6	0.35	199	1.26	39	33	6	78	0.61	312	2.44
5 acc/year with 50% at night min., or less than 5 acc/year with 3 acc. at night min. 1/	15	53	47	4	104	87.9	1.19	410	4.68	15	13	0	28	93.0	0.30	106	1.14	38	34	4	76	0.89	304	3.54
2 night accidents per year min.	21	64	56	6	126	105.9	1.19	498	4.71	28	17	0	45	112.0	0.40	147	1.31	36	39	6	81	0.79	351	3.40
3 night accidents per year min.	18	57	52	6	115	98.2	1.18	463	4.72	16	13	0	29	103.4	0.28	107	1.04	41	39	6	86	0.90	356	3.68
4 night accidents per year min.	11	41	35	2	78	45.8	1.70	300	6.55	9	6	0	15	49.1	0.30	51	1.04	33	29	2	63	1.40	249	5.51
3 night accidents per year with 50% injury & fatal accidents minimum	14	46	48	5	99	75.8	1.31	417	5.50	14	11	0	25	80.0	0.31	91	1.14	32	37	5	74	1.00	326	4.36
0.50 night accident rate min.	20	60	52	5	117	76.4	1.54	459	6.00	24	13	0	37	81.6	0.45	115	1.42	36	39	5	80	1.09	344	4.58
1.00 night accident rate min.	14	49	40	5	94	43.0	2.18	364	8.48	20	10	0	30	45.1	0.67	90	1.99	29	30	5	64	1.51	274	6.49
2.00 equivalent PDO rate minimum	18	56	51	5	112	69.5	1.61	448	6.45	23	12	0	35	73.9	0.47	107	1.45	33	39	5	77	1.14	341	5.00
8 nighttime EPDO's per year minimum	20	60	55	6	121	103.3	1.17	487	4.72	22	14	0	36	109.5	0.33	120	1.15	38	41	6	85	0.84	367	3.57

1/ Present Accident Warrant

TABLE 22

TOTAL ESTIMATED REDUCTION IN NIGHT ACCIDENTS FOR VARIOUS WARRANTS AT INTERSECTIONS

WARRANT DESCRIPTION	NO. OF PROJECTS WARRANTED	BEFORE					AFTER (estimated) 1/					ESTIMATED REDUCTION				
		PDO	INJ	FAT	TOTAL	EQUIV PDO'S	PDO	INJ	FAT	TOTAL	EQUIV PDO'S	PDO	INJ	FAT	TOTAL	EQUIV PDO'S
All Projects	26	70	57	6	133	511	31	24	0	55	199	39	33	6	78	312
5 acc/year with 50% at night min. or less than 5 acc/year with 3 acc. at night min.	15	70	57	6	133	511	37	26	2	65	233	33	31	4	68	278
2 night accidents per year minimum	21	70	57	6	133	511	36	18	0	54	162	34	39	6	79	349
3 night accidents per year minimum	18	70	57	6	133	511	33	19	0	52	166	37	38	6	81	345
4 night accidents per year minimum	11	70	57	6	133	511	43	31	4	78	288	27	26	2	55	223
3 night accidents per year with 50% injury + fatal accidents min.	14	70	57	6	133	511	44	22	1	67	205	26	35	5	66	306
0.50 night accident rate minimum	20	70	57	6	133	511	36	19	1	56	176	34	38	5	77	335
1.00 night accident rate minimum	14	70	57	6	133	511	45	30	1	76	262	25	27	5	57	249
2.00 equivalent PDO rate minimum	18	70	57	6	133	511	40	19	1	60	180	30	38	5	73	331
8 nighttime EPDO'S per year minimum	20	70	57	6	133	511	35	16	0	51	147	35	41	6	82	364

1/ The after periods were estimated by adding the after period for the projects "warranted" to the "estimated after period" (assuming an accident rate and percent severity breakdown equal to the before period) for the projects "not warranted".

experienced three night accidents, 1 project did not quite meet the 50 percent night accident criterion (9 out of 21), and the last project involved the only unlit intersection in a series of lighted intersections.

Five of the nine warrants investigated showed a greater night accident reduction than the present warrant (5 accidents per year with 50 percent occurring at night or less than 5 accidents per year with 3 accidents at night minimum). The following three warrants gave the greatest reduction and were approximately equally effective:

1. Two night accidents per year minimum (79 night accidents reduced with 21 projects).
2. Three night accidents per year minimum (81 night accidents reduced with 18 projects).
3. Eight night EPDO's per year minimum (82 night accidents reduced with 20 projects).

Since the estimated reduction in night accidents is almost identical with all three warrants, it appears that the best warrant is the second (three night accidents per year minimum) since it involves the least expenditure in funds—only 18 projects. Therefore, it is recommended that safety lighting be considered at locations which experience 4 or more night accidents in one year or 6 or more night accidents in two years.

ESTIMATING FUTURE ACCIDENTS

When considering what type of remedial measure is needed and whether a specific measure is warranted, it is necessary to examine specific types of "susceptible" accidents, because, in general, each type of possible remedial measures affects only certain types of accidents under specific conditions of geometry and traffic. For instance, it would be fruitless to install lighting if there were no nighttime accidents.

Therefore, warrants previously recommended are based on specific classes and numbers of accidents under specific conditions, e. g. , a minimum of three crossing plus left-turn accidents per year with a ratio $\leqq 0.50$ for the minor to major leg traffic volumes is needed to warrant red-yellow flashers at 4-leg intersections.

TABLE 23

Improvement Type	No. of Projects	Percent Rate Reduction	After Accident Rate	
Red-yellow flashers	9	51 (50)	1.13	(1.1)
4-way red flashers	6	77 (75)	0.80	(0.8)

To estimate the number of accidents that will occur after an improvement is made, the after accident rate of all types of accidents should be used (not the after rate of susceptible accidents or the percent reduction in susceptible accidents). The reasons for this are (a) all susceptible accidents are not generally reduced; (b) there may be a trade-off between types of accidents (e.g., rear-end accidents may increase at signals, etc.); and (c) "nonsusceptible" accidents may also decrease, although generally, at a reduced rate (e.g. , flashers may call attention to the intersection and reduce rear-end accidents). In the case of lighting, however, an exception was made. Only nighttime accidents were considered, the reason being that the only possible daytime effect of lighting is an occasional fixed object (pole) involvement. These daytime fixed object accidents should be extremely rare. An examination of the study projects confirmed this assumption.

Two prediction methods (average percent reduction in accident rates and an average base after rate) to estimate the number of future (expected) accidents were examined. Both methods require an estimate of the after period exposure in terms of million vehicles entering the intersection during a future period of time, the expected project life.

Flashing Beacons at 4-Leg Intersections

Table 23 gives the observed percent reduction in total accident rates and the total accident after rate for warranted (meeting previously recommended warrants) 4-leg intersection flashing beacons projects. The recommended "rounded" values are shown in parentheses. Rounded values are sufficiently accurate, especially when considering the relatively few projects on which these values are based.

Only the fifteen 4-leg warranted flashing beacon intersections consisting of a red-yellow and 6 all-red flashers were used for the estimates since other flasher type installations were too few for analysis.

The results of an analysis of both estimating methods is given in Table 24. Both methods give about the same absolute total difference and algebraic difference from the observed number of accidents. Both methods also result in similar sum of squared differences. Thus, it appears that approximately the same results are obtained by using either method, although the after base rate method seems somewhat better.

An examination on an individual project basis revealed that neither method was superior on the basis of which method provided the greatest number of expected predictions closest to the observed after accident experience. It is felt, however, that if a larger sample of intersections were reviewed, the base rate method of estimating accidents would prove to be superior. Intuitively, one would expect that if the accident problem had existed in the before period because of a lack of the remedial treatment, then one

TABLE 24

ESTIMATING ACCIDENTS AT 4-LEG FLASHING BEACON INTERSECTIONS

Flasher	Percent Rate Reduction	No. of Accidents			Algebraic Diff.[a] From Observed No. of Accidents		Difference Squared	
		Observed	Estimated					
			% Rate Change Method	Base Rate Method	% Rate Change Method	Base Rate Method	% Rate Change Method	Base Rate Method
Red-Yellow	50	66	68.8	64.2	+2.8 (18.0)	-1.8 (20.0)	67.8	62.6
All Red	75	18	18.9	18.0	+0.9 (12.1)	0.0 (11.0)	39.4	26.2
Total		84	87.7	82.2	+3.6 (30.1)	-1.8 (31.0)	107.2	88.8

[a]Total absolute difference from observed in parentheses (total deviation regardless of sign for all projects).

TABLE 25

CALCULATIONS

Observed Data: Before: Accidents	Observed Data: Before: Rate (a)	Observed Data: After Accidents	Calculated After Data: MV (b)	Calculated After Data: Estimated Rate (c)	Predicted After Number of Accidents: % Reduced Method (d)	Predicted After Number of Accidents: After Base Rate Method
11	3.56	2	3.2	1.78 50% of (a)	5.7 (b) x (c)	3.5 1.1 x (b)

would expect to reduce the accidents to a similar level at all locations. Using an average percentage reduction in rate would result in still high after rates at very high before rate locations, and in an unreasonably low after accident rate at relatively low before accident rate locations. The after base rate method is, therefore, recommended.

An example of calculations to estimate the future expected number of accidents for a single project using both methods is given in Table 25.

For this project, there were actually two accidents in the after period compared to a calculated number of 5.7 and 3.5 accidents, respectively, for the two methods. Both methods can be expected to give better accident estimates for a group of projects than for an individual installation, because of the normal chance fluctuations of accidents at individual intersections which are compensatory in a large group.

Safety Lighting at Intersections

After accident estimates of 18 safety-lighted warranted (based on previous warrant recommendations) intersection projects were made. Both total accident data and nighttime accident data were reviewed. Night rates were reduced 76 percent to an average rate of 0.84 and total accident rates were reduced 53 percent to an average rate of 0.89 for the warranted projects. For reasons discussed earlier, only nighttime accidents were considered.

Table 26 gives the observed percent reduction in night accident rate and the night accident after base rate for the 18 warranted intersection safety lighting projects. The other locations (railroad crossing, underpasses, bridges, etc.) are basically different situations than intersection and were too few in number for detailed warrant analysis. The recommended rounded values are shown in parentheses.

The result of an analysis of both estimating methods is given in Table 27. Both methods result in approximately the same total expected number of accidents, total algebraic difference, and total absolute difference. The sum of the squared differences is somewhat less for the after base rate method. On an individual basis, neither method results in more predictions closer to the observed value.

When estimating expected nighttime accidents by either the percent rate reduction or the after base rate method, it is necessary to use the estimated nighttime exposure. This is usually in the order of one-third of the total exposure.

Non-Four-Leg Intersection Flashers and Nonintersection Lighting

There were too few projects or accidents in miscellaneous categories (3-leg red-yellow flashers at intersections; railroad flashers; advance warning flashers prior to curves, intersections and schools; and the nonintersection lighting at railroad crossings, bridge approaches and underpasses) to establish new warrants and, therefore predictive accident parameters.

TABLE 26

INTERSECTION LIGHTING

No. of Projs.	Percent Rate Reduction	After Accident Rate (Night)
18	77 (75)	0.84 (0.80)

TABLE 27

ANALYSIS OF BOTH ESTIMATING METHODS

Percent Rate Reduction	No. of Night Accidents: Observed	No. of Night Accidents: Predicted: % Rate Change Method	No. of Night Accidents: Predicted: Base Rate Method	Algebraic[a] Diff. From Observed Accidents: % Rate Change Method	Algebraic[a] Diff. From Observed Accidents: Base Rate Method	Difference Squared: % Rate Change Method	Difference Squared: Base Rate Method
75	29	29.3	27.3	+0.3 (19.3)	-1.7 (18.5)	44.9	25.6

[a]Total absolute difference from observed in parentheses (total deviation regardless of sign for all projects).

TABLE 28

COMPARISON OF OBSERVED AND EXPECTED ACCIDENTS

Improvement Type	Parameters Used		Number of Accidents		
				Predicted	
	% Rate Reduced	Base Rate	Observed	% Rate Reduced Method	Base Rate Method
Flashing beacons (total accidents, all types)					
Intersection:					
3-leg red yellow (6)*	50	0.7	22	23.2	22.6
RR crossing (3)	80	0.2	2	3.2	2.6
Advance warning prior to:					
Curve (4)	50	1.0	17	18.6	19.4
Intersection (5)	20	1.0	67	70.0	77.1
Curve and intersection (9)	30	1.0	84	87.2	96.5
School crossing (3)	0	0.4	11	13.3	11.9
Bridge approaches (2)	0	2.4	27	28.2	28.1
Nonintersection safety lighting (night accidents only):					
RR crossing (6)	60	6.3	15	17.5	15.2
Bridge Approach (2)	50	4.6	5	5.6	5.1
Underpass (3)	10	0.7	7	7.1	7.7

*Number of projects in parentheses.

Execpt for advance warning flashers prior to curves and intersections, there was a wide range of values in after rates. The percent rate reductions generally varied to a much lesser extent; therefore, percent rate reductions are recommended for these interim guides.

Table 28 gives the observed and predicted after accident numbers for these project types. In spite of the small number of projects, the estimated accident totals are reasonably close to the observed totals.

Recommended Predictive Parameters

The parameters recommended for predicting after accident experience are given in Table 29.

DELINEATION

Delineation is only one of the many factors that a traffic engineer must consider in his attempts to have traffic flow smoothly and relatively free of accidents having highway characteristics as predominant causal factors. But it is a very important factor. Through delineation techniques, the engineer attempts to provide the driver with as much information concerning the width and alignment of the roadway as possible. There are many delineation techniques available and a few of them will be evaluated in this report.

TABLE 29

RECOMMENDED PARAMETERS

Improvement Type	% Reduction in Rate	After Base Rate
Intersection flashers[a]:		
4-leg red yellow	50	1.1[b]
3-leg red yellow	50[b]	0.7
4-way red	75	0.8[b]
Railroad crossing	80[b]	0.2
Advance warning flashers[a]:		
Curves and intersections	30	1.0[b]
School zones	0[b]	Not applicable
Bridge approaches	0[b]	Not applicable
Safety lighting[c]:		
Intersection	75	0.8[b]
Railroad crossing	60[b]	Not applicable
Bridge approach	50[b]	Not applicable
Underpasses	10[b]	0.7

[a]Based on all types of accidents.
[b]Preferred method.
[c]Based on night accidents.

The California Division of Highways has spent over $26,000 on the projects discussed in this report. These projects are only a sampling and represent a modest proportion, at best, of the monies California is spending to make driving as safe as is technologically possible. Because of rapid progress in the development of new delineation techniques, certain relatively new devices are not discussed in this report because of a lack of data.

Results and Findings

Delineation Projects in General—The projects can be divided into two categories: (a) steel or

TABLE 30

ACCIDENT RATES BY SEVERITY[a]

Period	Fatal	Injury	PDO	Total	MVM
Before	0.02 (54)	0.68 (1561)	0.87 (2007)	1.58 (3622)	2298.2
After	0.02 (48)	0.60 (1573)	0.83 (2187)	1.44 (3808)	2636.3

[a]Based only on projects reporting severity—24 projects.

TABLE 31

TOTAL ACCIDENT RATES BY DAY AND NIGHT[a]

Period	Day	Night	Total	MVM
Before	1.62 (244)	2.34 (174)	1.86 (418)	225.3
After	1.47 (222)	2.02 (150)	1.65 (372)	225.0

[a]8 projects; day volumes are assumed to constitute two-thirds of total.

timber guidemarkers and (b) painted lines (median striping, edge lines, no passing stripe).

The data are based on five guidemarker projects and 27 projects where painted lines were used as a means of delineating the roadway. Some projects contained data from more than one location so that much more than 33 locations are represented.

Accident Rates in General—The accident rates for before and after periods by severity are given in Table 30. All rates given here and in succeeding tables are in terms of accidents per million vehicle-miles (MVM) unless otherwise stated. Figures in parentheses are number of accidents.

The rate for fatal accidents remained the same for both periods. However, injury accident rates fell[1] as did PDO rates,[2] and the total accident rate dropped from 1.58 to 1.44[3] The drop appears to be attributable primarily to a reduction in injury and PDO rates.

Where such information was available, the day-night dichotomy of total accidents was made—resulting in the rates given in Table 31.

While a cursory look at Table 31 might lead one to believe that a reduction had been accomplished, statistical analysis showed that no significant change had been made; that is to say, such reductions may very well be attributable to sampling fluctuations.

Effect of Median Striping

Median striping consists of painted single or double stripes placed on the inside (left) shoulder immediately adjacent to the inside traffic lane. Its purpose is to delineate the median. Because projects falling into this category are of various types, each subdivision will be discussed separately. No summary analysis of the effectiveness of painted lines as a delineation measure in reducing accidents is available because of complexities created by the various reporting methods employed.

Median Striping and Driving Behavior—The placement of median striping is done with the hope that delineating the median area will encourage the drivers in the median lane to drive closer to the left edge of their lane than they would otherwise. Such action would also enable drivers in lanes to the right of the median lane to shift slightly to their left thereby alleviating a crowding situation.

A before-and-after type study was conducted to ascertain if such effects do occur. The location of the study was on an urban freeway (where the benefits of striping would be expected to be greatest) with a 1966 two way AADT of 69,000. The three eastbound lanes were marked with reflectorized traffic tape "ticks" as a field of reference for observers stationed above the freeway on an overcrossing structure. The observers reported their observations into a tape recorder. One observer was posted for each lane and one other individual kept track of lane volumes. Three separate observation periods were used to record data indicative of the behavior of (a) low-volume daylight traffic (±1300 vph eastbound); (b) high-volume daylight traffic (±2500 vph eastbound); and (c) low-volume night traffic (±1300 vph eastbound).

[1] χ^2 at 1 df = 13.08, $P < 0.001$.
[2] χ^2 at 1 df = 2.79, $P < 0.10$.
[3] χ^2 at 1 df = 14.19, $P < 0.001$.

TABLE 32

MEAN DISTANCES (IN FEET) OF LEFT SIDE OF VEHICLE FROM LEFT EDGE OF LANE

Traffic Condition	Median Lane		Center Lane		Right Lane	
	Before	After	Before	After	Before	After
Daytime, low-volume	2.47	2.58	2.61	2.28*	2.57	2.64
Daytime, high-volume	2.44	1.98*	2.24	2.39	2.58	2.95*
Night, low-volume	2.70	2.15*	2.57	2.43	2.50	2.28*

*Significant at less than 0.05 level.

TABLE 33

ACCIDENT RATES BY MEDIAN ASSOCIATION[a]

Period	Median	Non-Median	MVM
Before	0.50 (1930)	1.25 (4773)	3830.7
After	0.52 (2196)	1.13 (4804)	4242.3

[a]22 projects.

Following the data collection for the before period, a standard double yellow median stripe was painted on the median shoulder. Then observations for the after period data were made. The results are summarized in Table 32. It appears that in slightly over 50 percent of the observations a change in mean distance from the left edge of lane occurred. In four cases, the change was in the hypothesized direction (to the left as indicated by a decrease in the mean) and in one case (the lane most distant from the stripe) the change was to the right. Thus, it may be argued that the median stripe appears to affect driving behavior somewhat in the hypothesized manner.

The preceding results are somewhat conservative in that the before period situation did not represent a total absence of delineation. Some artificial lighting was present near the test area and the median shoulder was asphalt, whereas the traveled way was concrete. Possibly, more positive results would have been obtained had the test been conducted on a segment of freeway where less delineation was available in the before period. Additionally, since the measurements are actually ratings by human observers, consistent rating errors made on the before and after observations could reduce the probability of illustrating an effect. Finally, it may be that any effects of the median stripe would be heightened on a freeway with higher lane volumes than used in the study.

Median Striping and Total Accidents—Projects (N=22) where total accidents were reported showed a significant reduction from 1.75 to 1.65 accidents per MVM.[4]

To determine what types of accidents were being reduced, the accidents were dichotomized into those that were median associated and those that were not. It should also be pointed out that these are freeway locations where high volumes and speeds are present.

As reflected in Table 33, the median associated accident rate did not change significantly[5] while the non-median associated accident rate decreased.[6] Two factors may account for these findings. Drivers in the median lane can drive closer to the median and feel safer in doing so when the striping is present. In turn, drivers in non-median lanes can drive closer to their left lane lines creating more clearance between traffic streams. Again, the effects of median striping would depend somewhat on the contrast already existing between median shoulder and traveled way as well as the amount of artificial lighting available at night.

The analysis included median stripe projects without regard to the presence or absence of a median barrier or the type of median barrier, if present. On the assumption that these factors may, as has been shown in the past, influence accident and severity rates, analyses were conducted on the various combinations of median barriers and stripes reported.

Painted Double Stripe, No Median Barrier—Table 34 gives a summary of six median stripe projects where no barrier was present in the median. Five projects utilized double yellow median stripes and one employed double white median stripes. The fatal and injury median associated accident rates did not change significantly. However, the PDO accident rate increase proved significant[7] as did the total median accident

[4]χ^2 at 1 df = 11.82, $P < 0.001$.
[5]χ^2 at 1 df = 1.26, $P > 0.10$.
[6]χ^2 at 1 df = 21.96, $P < 0.001$.
[7]χ^2 at 1 df = 14.88, $P < 0.001$.

TABLE 34

MEDIAN ASSOCIATED ACCIDENT RATES BY SEVERITY[a]
(No Barrier—Painted Median Stripe)

Period	Fatal	Injury	PDO	Total	MVM
Before	0.02 (12)	0.25 (200)	0.24 (194)	0.51 (406)	792.7
After	0.01 (10)	0.27 (233)	0.35 (303)	0.63 (546)	870.0

[a]6 projects.

TABLE 35

NON-MEDIAN ASSOCIATED ACCIDENT RATES BY SEVERITY[a]
(No Barrier—Painted Median Stripe)

Period	Fatal	Injury	PDO	Total	MVM
Before	0.01 (9)	0.44 (345)	0.61 (485)	1.06 (839)	792.7
After	0.01 (8)	0.39 (343)	0.63 (547)	1.03 (898)	870.0

[a]6 projects.

TABLE 36

TOTAL ACCIDENT RATES BY SEVERITY[a]
(No Barrier—Painted Median Stripe)

Period	Fatal	Injury	PDO	Total	MVM
Before	0.03 (21)	0.69 (545)	0.86 (679)	1.57 (1245)	792.7
After	0.02 (18)	0.66 (576)	0.98 (850)	1.66 (1444)	870.0

[a]6 projects

TABLE 37

SELECTED DAY-NIGHT ACCIDENT RATES[a]
(No Barrier—Painted Median Stripe)

Period	Day Median Rate	Night Non-Median Rate	Total Day Rate	Total Night Rate
Before	0.40 (210)	1.41 (369)	1.28 (680)	2.16 (565)
After	0.59 (342)	1.21 (347)	1.53 (893)	1.92 (551)

[a]6 projects

rate.[8] Thus, the increase in the total median accident rate appears to be predominantly attributable to the increase in PDO rate.

The picture for non-median associated accidents is somewhat different as reflected in Table 35. No significant changes appeared in any of the categories. Finally, an analysis of total accident rate was made utilizing all the accident experience these six projects yielded (Table 36). Chi-square tests showed that only the PDO accident rate changed significantly.[9] However, the increase was not enough to influence the total rate.

In summary, with no median barrier present, increases in the PDO and total categories of median accident rates were observed. However, all severity categories of non-median accidents remained constant. Finally, total (median plus non-median) PDO rates increased. Thus, it appears that at least in those situations where the median contained no barrier, no reduction in accident rates of any severity type can be attributed to median striping.

Day-Night Effectiveness—With regard to the day-night factor, chi-square analysis showed that where no barrier existed, median associated day accident rates increased.[10] Night non-median associated accidents meanwhile decreased.[11] The net effect on total accidents was that day accidents increased[12] and night accidents decreased.[13] Other comparisons were not significant. Results are summarized in Table 37.

Median Striping with Median Barriers—Painted Double Yellow Stripes, Beam Barrier: Table 38 gives the accident rates of three projects where beam median barriers were present. There were no significant changes in any of the categories listed. A day-night dichotomy of the accident experience was not made because of a lack of such information. No changes on non-median associated accident rates or total accident rate were observed. Thus, striping did not appear to alter accident rates.

Painted Double Yellow Stripes, Cable Barrier: Table 39 summarized the accident rates experienced in three projects at cable barrier locations and where painted double yellow median striping was used. Of the categories, there was a small reduction in the injury median accident rates[14] and a corresponding decrease in the total median accident rates.[15] Meanwhile, fatal median accidents increased from 0 to 5. However,

[8]χ^2 at 1 df = 9.65, $P < 0.01$.
[9]χ^2 at 1 df = 6.54, $P < 0.02$.
[10]χ^2 at 1 df = 20.53, $P < 0.001$.
[11]χ^2 at 1 df = 4.28, $P < 0.05$.
[12]χ^2 at 1 df = 12.46, $P < 0.001$.
[13]χ^2 at 1 df = 3.90, $P < 0.05$.
[14]χ^2 at 1 df = 2.89, $P < 0.10$.
[15]χ^2 at 1 df = 3.55, $P < 0.10$.

TABLE 38

MEDIAN ASSOCIATED ACCIDENT RATES BY SEVERITY[a]
(Beam Barrier—Painted Double Yellow Stripe)

Period	Fatal	Injury	PDO	Total	MVM
Before	— (3)	0.04 (28)	0.25 (156)	0.29 (187)	634.8
After	— (2)	0.03 (22)	0.26 (164)	0.29 (188)	639.8

[a]3 projects.

TABLE 39

MEDIAN ASSOCIATED ACCIDENT RATES BY SEVERITY[a]
(Cable Barrier—Painted Double Yellow Stripe)

Period	Fatal	Injury	PDO	Total	MVM
Before	0.00 (0)	0.11 (69)	0.51 (316)	0.62 (385)	624.5
After	0.01 (5)	0.08 (51)	0.45 (288)	0.54 (344)	641.6

[a]3 projects (2 other projects did not provide severity data).

this is not very meaningful because of the small numbers involved; thus, a slight decrease in median associated accident rates was observed which appeared to be predominantly due to a decrease in injury accident rates. Total accident rates decreased from 2.37 to 2.09 accidents per MVM.[16] Non-median accident rates declined also from 1.76 to 1.55 accidents per MVM.[17] No day-night dichotomy of accident experience was available. No further analysis of non-median or total accidents was possible.

Painted Single White Stripe, Cable Barrier: A former technique which is no longer used was to paint a single white stripe along the median. Six projects fell into this category (one project did not report accident severities); results are given in Table 40. (Median associated accident rates are more appropriate here but were unavailable by severity.) The total accident rate increased[18] primarily due to an increase in PDO rates.[19] A day-night dichotomy was impossible to obtain. Median accident rates increased from 0.52 to 0.69 accidents per MVM while non-median accident rates decreased from 0.76 to 0.69 accidents per MVM.[20] The finding that total accident rates increased lends support to the Division of Highways' decision to discontinue this method of delineating the median.

Painted Double Yellow Stripe vs Single White Stripe, Cable Barrier: A question might be raised as to the relative effectiveness of double yellow striping versus single white striping on accident rate reduction. Table 41 gives a summary of the total accident rate experience of 12 projects which could be analyzed to provide relevant information. The double yellow striping projects started out with a higher median associated accident rate[21] than the single white projects but finished with a lower rate.[22] The total accident rate for white stripe projects increased[23] from 1.37 to 1.56 accidents per MVM while that for yellow stripe projects dropped from 2.47 to 2.05 accidents per MVM.[24] Total non-median accident rates on white stripe projects did not change significantly, while those for yellow stripe projects decreased from 1.82 to 1.53 accidents per MVM. Thus, yellow median striping seems to be a far better technique of reducing accidents.

By way of summary, in those situations where the median contained no barrier, no reduction in accident rates whatsoever could be found. With beam barriers present, the same was true. However, when cable barriers were present, median striping appeared to reduce median injury and total median accident rates.

[16] χ^2 at 1 df = 11.74, $P < 0.001$.
[17] χ^2 at 1 df = 8.56, $P < 0.01$.
[18] χ^2 at 1 df = 2.88, $P < 0.10$.
[19] χ^2 at 1 df = 3.38, $P < 0.10$.
[20] χ^2 at 1 df = 19.23, $P < 0.001$ (median accidents); χ^2 at 1 df = 2.73, $P < 0.10$ (non-median accidents).
[21] χ^2 at 1 df = 5.73, $P < 0.05$.
[22] χ^2 at 1 df = 24.62, $P < 0.001$.
[23] χ^2 at 1 df = 15.11, $P < 0.001$.
[24] χ^2 at 1 df = 42.67, $P < 0.001$.

TABLE 40

TOTAL ACCIDENT RATES BY SEVERITY[a]
(Cable Barrier—Single White Stripe)

Period	Fatal	Injury	PDO	Total	MVM
Before	0.01 (7)	0.50 (383)	0.77 (590)	1.23 (980)	766.8
After	0.01 (6)	0.52 (425)	0.85 (701)	1.38 (1132)	821.9

[a]6 projects (1 other project did not provide severity data).

TABLE 41

MEDIAN ASSOCIATED ACCIDENT RATES BY TYPE OF STRIPING[a]
(Cable Barrier Present)

Period	Single	White	MVM	Double	Yellow	MVM
Before	0.53	(614)	1161.3	0.65	(676)	1032.3
After	0.68	(859)	1265.0	0.52	(556)	1072.6

[a]7 white stripe projects; 5 yellow stripe projects (2 yellow stripe projects which did not provide total median associated accident data by severity were added to those in Table 38. One white stripe project which did not provide total accident data by severity was added to those in Table 40).

With cable barriers, non-median accident rates also declined as well as overall total accident rates (total median plus total non-median accidents). Finally, when a cable barrier was present in the median, the double yellow stripe proved far superior to the single white stripe in reducing accident rates.

Reflectorized Guidemarkers

At Curves—Guidemarkers are used to delineate the road for the motorist. They are simple white paddles mounted on timber or steel posts placed adjacent to the roadway in full view of the oncoming driver. Reflectors are mounted on the paddles for night delineation. The guidemarker installations discussed here are all situated at points where horizontal curves are present. Table 42 summarizes the findings

TABLE 42

ACCIDENT RATES BY SEVERITY ON GUIDEMARKER INSTALLATIONS AT CURVES[a]

Period	Fatal	Injury	PDO	Total	MVM
Before	0.11 (5)	0.66 (29)	0.82 (36)	1.59 (70)	44.0
After	0.07 (3)	0.36 (16)	0.61 (27)	1.04 (46)	44.1

[a]4 projects.

TABLE 43

ACCIDENT RATES BY DAY-NIGHT FACTOR ON GUIDEMARKER INSTALLATIONS AT CURVES[a]

Period	Day	MVM	Night	MVM
Before	0.98 (29)	29.5	2.28 (33)	14.5
After	0.58 (17)	29.5	1.64 (24)	14.6

[a]4 projects.

by severity of four guidemarker projects. Injury[25] and total rates[26] showed a statistically significant decrease.

With regard to the day-night factor, the distribution of accident rates where known is given in Table 43. Day or night accident rates did not drop significantly because of the small numbers of accidents involved.[27] One report showed no improvement in any severity or day-night category. It was not included in the data of Table 43 because MVM figures could not be determined from it.

Another analysis of the effect of guidemarker installations in preventing accidents consists of determining their effectiveness within curves of various radii. The data in Table 44 summarize the experience of 221 locations. Five radii classifications plus a total category were created. Only the category "500 or less" showed a significant change.[28] All other differences can be assumed to be the result of random sampling fluctuations. It should be pointed out, however, that the "500 or less" category had over twice the accident experience of any other category. Therefore, it is possible that effects may also have occurred in the other categories but a larger accident experience is needed to detect them.

No day-night comparisons showed any significant changes. It was impossible to determine if ran-off-road accident rates were reduced because of the nature in which the data were reported. It would be desirable, however, to attempt such an analysis in the future since guidemarkers should have the greatest effect on these accidents.

[25] χ^2 at 1 df = 3.23, P < 0.10.
[26] χ^2 at 1 df = 4.61, P < 0.05.
[27] χ^2 at 1 df = 2.64, P > 0.10 (day); χ^2 at 1 df = 1.12, P > 0.30 (night).
[28] χ^2 at 1 df = 3.17, P < 0.10.

TABLE 44

TOTAL ACCIDENT RATES BY SHARPNESS OF CURVE[a]

Radius (ft)	Before	MV	After	MV
500 or less	1.89 (64)	33.8	1.33 (48)	36.2
501-1000	0.97 (21)	21.6	1.04 (24)	23.1
1001-2000	0.28 (4)	14.3	0.59 (9)	15.3
2001-5000	0.37 (12)	32.6	0.57 (20)	34.9
More than 5000	0.29 (5)	17.5	0.37 (7)	18.7
Total	0.88 (106)	119.8	0.84 (108)	128.2

[a]1 project.

TABLE 45

ACCIDENT RATES OF GUIDEMARKER INSTALLATIONS AT BRIDGE APPROACHES

Period	Fatal	Injury	PDO	Total	MV
Before	0.00 (0)	0.03 (7)	0.07 (18)	0.10 (25)	252.9
After	0.00 (0)	0.02 (6)	0.04 (9)	0.06 (15)	252.9

TABLE 46

TOTAL ACCIDENT RATES BY SEVERITY[a]
(Right Edge Stripe)

Period	Fatal	Injury	PDO	Total	MVM
Before	0.09 (15)	0.78 (137)	1.00 (176)	1.86 (328)	176.1
After	0.08 (14)	0.75 (131)	1.00 (174)	1.83 (319)	174.4

[a]2 projects.

TABLE 47

ACCIDENT RATES BY SEVERITY[a]
(No Passing Stripe)

Period	Fatal	Injury	PDO	Total	MVM
Before	0.00 (0)	2.00 (5)	5.60 (14)	7.60 (19)	2.5
After	0.00 (0)	1.61 (5)	0.97 (3)	2.58 (8)	3.1

[a]3 projects.

At Bridge Approaches—Reflectorized guidemarkers were placed on the left and right sides of approaches to 99 bridges located on a two-lane desert highway. Table 45 gives the accident rates reported. Only PDO rates decreased significantly.[29] Although night accidents decreased from 20 to 10, this did not prove significant.[30] Day accident rates remained unchanged.

In summary, the reflectorized guidemarker installations have experienced a decrease in the total accident rate in some cases. The effects have been observed on curves with radii less than 501 ft. The ability of guidemarkers to reduce accident rates at curves of larger radii has not been observed. Bridge approaches with reflectorized guidemarkers have shown a reduction in PDO accidents.

Right Edgeline

A technique for delineating the right edge of roadway and the shoulder is to place a solid painted stripe on the shoulder, 1 ft from the edge of the traffic lane. Additionally, when the shoulder width exceeds 8 ft, diagonal markings, 12 in. wide, may also be included at 100-ft intervals. Only projects consisting of 2-in. wide solid stripes without diagonal markings will be discussed here. Table 46 depicts the accident rates observed on about 72 miles of two-lane highways. There is a remarkable stability present which indicates that in no category did the striping influence accident rates. This is somewhat in conflict with other studies (8, 9, 10 and 11). However, it might be argued that the purpose of edge striping is to reduce one type of accident rate—the ran-off-road type. Consequently, an analysis of such accidents was made separating them from the remainder of the total. It was found that ran-off-road accident rates decreased from 0.63 to 0.48.[31] Non-ran-off-road accident rates did not change significantly, going from 1.23 to 1.35. Therefore, it seems tenable that the edge striping did reduce ran-off-road accident rates. With regard to day-night accident rates no significant changes were observed.

No-Passing Stripe

No-passing striping delineates only the centerline of the roadway and is used to prevent passing activity on curves (vertical or horizontal) where sight distance is too short for such maneuvers. It does not fall into the delineation (guidance) category but rather is a regulatory device. A no-passing stripe is a solid double yellow stripe placed along the centerline of the roadway.

[29] χ^2 at 1 df = 3.38, $P < 0.10$.
[30] χ^2 at 1 df = 2.70, $P > 0.10$ (χ^2 needed is 2.71 for $P = 0.10$ at 1 df).
[31] χ^2 at 1 df = 3.50, $P < 0.10$.

An analysis of the accident experience of three projects over a 9-yr period was made on the data in Table 47. No fatal accidents occurred during the entire survey period. The injury accident rate decline was not significant because of a small number of accidents (5 in each period). However, the PDO accident rate decreased.[32] The total accident rate decreased also.[33] Thus, the decrease in the total accident rate seems to be primarily the result of a decrease in PDO accidents. Additionally, day accident rates declined from 6.47 to 1.43 accidents per MVM.[34] However, night accidents showed no significant change.

Since the purpose of the stripe is to eliminate passing accident rates, it is interesting to note if this goal was accomplished. An examination of the data showed that the number of passing accidents dropped from 6 to 1 while non-passing accidents fell from 13 to 7. The number of passing accidents was too small to test although the drop appears to be quite real. Non-passing accidents were tested and the change was found to be not significant.[35] Thus, any effects the striping may have had on accident reduction still remain a matter of conjecture.

Reducing Accident Severity

The task of reducing accident severity is very complex. Part of the complexity lies in the fact that many times one does not know if an "improvement" is really effected. For instance, if a reduction in fatal and/or injury categories is observed without an increase in the PDO category, it can still be assumed or accepted that accident severity has been reduced. However, what is to be concluded when one severity category, PDO for instance, experiences a rate increase while the fatal or injury accident rate decreases? Then how much was gained (or lost) becomes a matter of subjective judgment rather than an unbiased conclusion clearly demonstrable by empirical methods and measurement.

An attempt to provide a measure of severity that reflects both the accident rate and the severity of the accident making up that rate is embodied in the EPDO concept. EPDO's reflect the cumulative severity of accidents and are computed here by adding the number of PDO's to the product of 6 times the number of fatal plus injury accidents. The Severity Index (SI) measures the average (mean) severity of all accidents in a given period. Additionally, an EPDO rate obtained by dividing total EPDO accidents by million vehicle-miles of exposure (MVM) provides a common basis for comparing all projects against each other.

Table 48 was prepared to show the three severity measures of the project types discussed in this report. It should be emphasized that Table 48 is a summary only of those projects where severity was reported. Nothing is known of the accident severities of projects not included. Because of this, one should not assume that subcategories within a delineation type will necessarily add up to the total values given for that type. For instance, the EPDO accident rate figures for total accidents on right edgeline stripe projects are based on data supplied by two reports. However, the data on ran-off-road accidents under this same delineation category are based on one report because the other report did not classify ran-off-road accidents by severity. Consequently, one cannot subtract the ran-off-road accident rates from the total rate to obtain non-ran-off-road rates. This more complicated procedure was employed so the data reported here could be based on as large an accident experience as was available for each situation.

As a rule, those categories with a higher total EPDO count can be assumed to also have larger accident frequencies. However, this does not necessarily mean that they represent more dangerous situations. Other factors such as the number of vehicles passing through and the length of the highway segment being observed varied from

[32] χ^2 at 1 df = 8.31, $P < 0.01$.
[33] χ^2 at 1 df = 6.23, $P < 0.02$.
[34] χ^2 at 1 df = 5.19, $P < 0.05$.
[35] χ^2 at 1 df = 2.58, $P > 0.10$.

TABLE 48

DELINEATION AND ACCIDENT TYPES BY SEVERITY

Delineation and Accident Types	EPDO		SI		EPDO RATE	
	Before	After	Before	After	Before	After
Median striping with no barrier:						
Median accidents	1466	1761	3.61	3.23	1.85	2.02
Non-median accidents	2609	2653	3.11	2.95	3.29	3.05
Total accidents	4075	4414	3.27	3.06	5.14	5.07
Double yellow median striping with cable barrier, median accidents	730	624	1.90	1.81	1.17	0.97
Double yellow median striping with beam barrier, median accidents	342	308	1.83	1.64	0.54	0.48
Single white median striping with cable barrier, total accidents	2930	3287	2.99	2.90	3.82	4.00
Reflectorized guidemarkers at curves, total accidents	240	141	3.43	3.07	5.45	2.24
Reflectorized guidemarkers at bridge approaches, total accidents	60	45	2.40	3.00	0.24[a]	0.18[a]
No passing stripe, total accidents	44	33	2.32	4.13	17.60	10.65
Right edgeline stripe:						
Ran-off-road accidents	1083	1044	3.32	3.27	6.18	5.99
Total accidents	183	193	3.81	3.64	3.76	4.11

[a]Based on million vehicles (MV) instead of million vehicle-miles (MVM).

TABLE 49

PROJECT TYPE BY EFFECTIVENESS AND CRITERION MEASURE

Description	Improved[a]	No Change	Worsened	Total
Total	5	23	4	32
Median striping, no barrier	0	5	1	6
Double yellow median stripe, cable barrier	3	1	1	5
Double yellow median stripe, beam barrier	0	3	0	3
Single white median stripe, cable barrier	1	4	2	7
Reflectorized guide-markers at curves	0	5	0	5
Reflectorized guide-markers at bridge approaches	0	1	0	1
Right edge stripe	0	2	0	2
No-passing stripe	1	2	0	3

[a]$\chi^2 = 2.71$ at 1 df; $P < 0.10$.

report to report. Consequently, only EPDO rates computed on the common basis of MVM provide a gage of relative accident liability coupled with severity.

Table 48 indicates that EPDO rates and SI's appear to be decreasing, in general. Just which reductions can be considered statistically significant and attributable to the delineation improvement is unknown. Total EPDO accidents, generally, have increased but so has traffic volume.

Evaluation of Individual Projects

A final analysis was made to determine the success or failure of each project on an individual basis. It was felt that a project considered a success should reduce more accidents than it would cause and this would be reflected in total accident rates. All analyses were made with this in mind. Table 49 summarizes the results. In short, only five projects could show improvements while 27 could not.

Some of the "no change" findings could be attributable to statistically small accident samples. The four "worsened" projects are definitely a cause for concern.

CONCLUSIONS

The findings are somewhat discouraging. Naturally, the hope was to demonstrate much more improvement than appears to have been accomplished. It may be that the worth of the delineation measures lie not so much in terms of accident reduction but in terms of the "near misses" which might have been averted and the psychological comfort they may provide the driver (12). These benefits are much harder to assess and require different approaches than those employed in this report. Possibly, other factors such as the amount of contrast between shoulder and traveled way, width of shoulder, and terrain features, should be considered in attempts to evaluate when and where the various delineation measures are effective.

Presently, new methods of delineation are being developed and evaluated. For instance, retroreflective raised pavement markers of various colors are now being placed on California highways. Perhaps future studies will show these to be more effective devices.

Warrants and Predictive Variables

Except for the installation of delineation on curves of 500-ft radius or less, no attempt could be made to establish warrants or predictive variables because of the dearth of accident data found in the delineation categories studied. The small number of projects available for study was usually accompanied by small accident experience which impeded analysis.

A 30 percent total accident rate reduction can be expected for installations of delineators on curves where the radius does not exceed 500 ft.

GUARDRAIL

The main purpose of guardrail is to reduce the severity of accidents of vehicles leaving the traveled way, generally going over an embankment or striking fixed objects. This is accomplished by absorbing energy (reducing deceleration rate) by deflection of the guardrail and by redirecting the vehicle into a safer path. A secondary purpose of guardrail is to provide increased delineation of the edge of the highway, and to reduce the frequency of accidents caused by reduced visibility (fog, night, rain, etc.) or poor or hidden edge of highway demarcation.

The need for guardrail on the roadway is generally determined by considering the following factors (13, 14): height of embankment, steepness of embankment, alignment, roadbed width, accident history, speed and volume of traffic, visibility and climatic conditions.

Analysis by Location

Fourteen guardrail projects were examined and the percent change in accident rates before to after was plotted (Fig. 7). Twelve of these projects were improved although only three were significantly so.[36]

Eleven of the 14 projects are summarized in Table 1. The other three projects involved improvements in addition to guardrail or did not fit into the summarized categories and are discussed separately.

Protective guardrail has been placed on the outside of curves, the inside of curves, the combination of the two, and at bridge ends. The projects in these categories are summarized in Table 50. Of the 11 projects representing identical 14½-yr before and after periods, only two projects had significant accident reductions, the other 9 indicated no significant change. As a group, however, accident reduction was statistically significant.

Total Projects—Total accidents were reduced[37] and the equivalent EPDO's were halved from 149 to 78. (Note: The statewide average of severity breakdowns is used to determine W.) Therefore, W = 6 for this category and EPDO = PDO + 6 (injury + fatal). Total accident rates were reduced 60 percent and the EPDO rate 66 percent.

In addition to the reduction in accident rates, severity was reduced at protective guardrail installations. This is reflected by the decrease in the SI or by noting that the percentage reduction in accident rates increased as the severity increased. Night accidents[38] were reduced 27 percentage points greater than day accidents.

The majority of the before problem is single vehicle, namely ran-off-road accidents which were significantly reduced.[39] Any reductions of accidents must be attributed to

[36] χ^2 at 1 df = 2.71, $P < 0.10$.
[37] χ^2 at 1 df = 13.63, $P < 0.001$.
[38] χ^2 at 1 df = 12.46, $P < 0.001$.
[39] χ^2 at 1 df = 7.53, $P < 0.01$.

TABLE 50

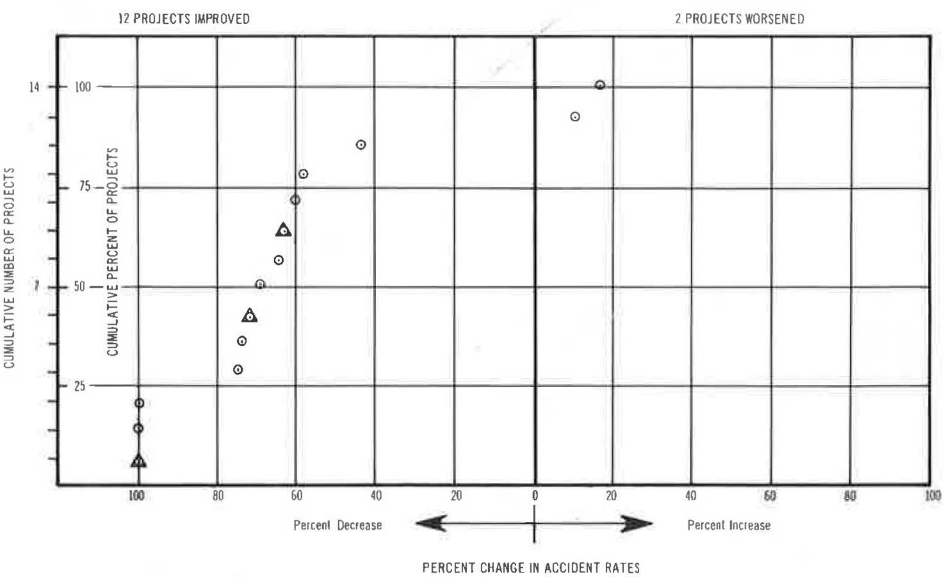
PROTECTIVE GUARDRAIL SUMMARY

			PROJECTS				ACCIDENT DESCRIPTION														
								ACCIDENT TYPE				SEVERITY			LIGHT CON.						
								SINGLE VEHICLE													
			TOTAL NO.	IMPROVED	WORSENED	NO CHANGE	YEARS OF EXPERIENCE	HIT FIXED OBJECT	RAN OFF ROAD	OTHER	SUB TOTAL	MULTIPLE VEHICLE	PDO	INJURY	FATAL	DAY a/	NIGHT a/	TOTAL ACCIDENTS	MILLION VEHICLES	EQUIVALENT PDO (EPDO)	SEVERITY INDEX (SI)
OUTSIDE OF CURVE	Before	No. of Accidents					$8\frac{10}{12}$	3	16	5	24	4	14	13	1	9	19	28	9.3	98	3.5
		Rate						0.32	1.72	0.53	2.58	0.43	1.51	1.40	0.11	1.45	6.13	3.01		10.53	
	After	No. of Accidents	6	1 S	0	5	$8\frac{10}{12}$	1	5 S	0	6 S	6	7	4 S	1	6	6 S	12 S	11.1	37	3.1
		Rate						0.09	0.45	0	0.54	0.54	0.63	0.36	0.09	0.81	1.62	1.08		3.34	
		% Rate Change					3	-72	-74	-100	-79	+26	-58	-74	-18	-44	-74	-64		-68	
INSIDE OF CURVE	Before	No. of Accidents											4	6	0	4	6	10	5.0	40	4.0
		Rate							NOT				0.80	1.20	0	1.20	3.59	2.00		8.00	
	After	No. of Accidents	2	0	0	2	3		AVAILABLE				4	5	0	4	5	9	6.3	34	3.8
		Rate											0.64	0.79	0	0.95	2.38	1.43		5.40	
		% Rate Change											-20	-34	0	-21	-34	-29		-32	
OUTSIDE AND INSIDE OF CURVE	Before	No. of Accidents					1	0	3	0	3	1	0	4	0	1	3	4	1.3	24	6.0
		Rate						0	2.31	0	2.31	0.77	0	3.08	0	1.16	6.73	3.08		18.45	
	After	No. of Accidents	1	0	0	1	1	0	2	0	2	0	1	1	0	1	1	2	1.6	7	3.5
		Rate						0	1.25	0	1.25	0	0.63	0.63	0	0.94	1.88	1.25		4.38	
		% Rate Change							-45	0	-45	-100	∞	-80	0	-19	-73	-59		-76	
BRIDGE ENDS	Before	No. of Accidents					$1\frac{7}{12}$	3	1	0	4	3	1	3	3	2	5	7	9.0	37	5.3
		Rate						0.33	0.11	0	0.44	0.33	0.11	0.33	0.33	0.33	1.67	2.33		4.11	
	After	No. of Accidents	2	1 S	0	1	$1\frac{7}{12}$	0	0	0	0	0	0	0	0	0	0 S	0 S	10.2	0	0
		Rate						0	0	0	0	0	0	0	0	0	0	0		0	
		% Rate Change						-100	-100	0	-100	-100	-100	-100	-100	-100	-100	-100		-100	
TOTAL b/	Before	No. of Accidents					$14\frac{5}{12}$	6	20	5	31	8	19	26	4	16	33	49	19.6	149	4.1
		Rate						0.31	1.02	0.25	1.58	0.41	0.97	1.33	0.20	1.23	5.05	2.50	(24.6)	10.15	
	After	No. of Accidents	11	2 S	0	9	$14\frac{8}{12}$	1 S	7 S	0	8 S	6	12	10 S	1	11	12 S	23 S	22.9	78	3.4
		Rate						0.04	0.31	0	0.35	0.26	0.52	0.44	0.04	0.72	1.57	1.01	(29.2)	3.41	
		% Rate Change						-87	-70	-100	-78	-37	-46	-67	-80	-42	-69	-60		-66	

"S" Indicates change is significant at the 0.10 level using the Chi-Square Test.

a/Assumed 2/3 MV Day and 1/3 MV Night for Rate Calculations

b/Totals of Available Data Only for the Respective Columns



△ Indicates change is significant at the 0.10 level using the Chi-Square test.

Figure 7. Protective guardrail.

TABLE 51

ACCIDENTS INVOLVING 28 BRIDGES

Study	PDO	Injury	Fatal	Total Accidents	MV	EPDO	SI
Before:							
No. of accidents	20	31	7	58	101.1	248	4.3
rate (acc/MV)	0.20	0.31	0.07	0.57		2.45	
After:							
No. of accidents	7	5*	1	13*	50.0	43	3.3
rate (acc/MV)	0.14	0.10	0.02	0.26		0.86	
% rate change	-30	-68	-71	-54		-65	

*Accident change is significant at 0.10 level of chi square test.

the delineation quality of the guardrail. This is analyzed later in the report. The small number of multiple-vehicle accidents remained approximately the same.

Guardrail on Outside of Curves—Six guardrail installations on the outside of curves on rural two-lane highways were reviewed. Only one was significantly improved; the other five showed no change.

Two of these projects involved closing a gap in the existing guardrail on the curve. It was felt at the time of the improvement that the opening of the guardrail created a break in the delineation around the curve, and led some drivers to believe the highway proceeded through the guardrail opening. Ran-off-road accidents for these two projects were reduced from 8 to 0.[40]

The six projects represented approximately nine project years in each period. As in the general case of all 14 projects, the total number of accidents were[41] reduced and the SI dropped as did the EPDO accidents. This reduction occurred despite a 16 percent increase in exposure (MV) in the after period. Also the ran-off-road[42] and night[43] accidents were reduced.

Guardrail on Inside of Curves—Guardrail was placed for these projects on the inside of two curves where a side hill condition existed and the outside of the curve was in cut. No changes of accident frequency were noted even though the MV increased 26 percent. Sufficient data were not available for further analysis in this category.

Guardrail on Inside and Outside of Curve—Guardrail was placed on both sides of a 650-ft radius curve on a mountainous two-lane highway. Only small insignificant reductions of accidents were noted (4 to 2) in the 1 yr periods.

Guardrail Flares at Bridge Ends—Two guardrail installations at bridge ends were reviewed, one of which showed a significant[44] reduction in accidents. This bridge is a two-lane, 2-way bridge 26 ft wide which is 8 ft narrower than the approaching roadway. Two of the three multiple-vehicle accidents were head-on collisions in which one vehicle struck the bridge rail and bounced into an opposing vehicle. Apparently the delineation quality of the bridge flares on the right of traffic was responsible for eliminating the accident problem at this location. In the other project, there were two fatal single-car accidents in which the vehicles struck the bridge end. After the improvement, no accidents were reported.

The results of a previous study (15) of 28 bridges in California in which a guardrail flare was also used on the right of approaching traffic is summarized in Table 51. The accidents involve the bridge or bridge rails. This experience is on a rural two-lane highway with 10- to 12-ft lanes and 2- to 8-ft shoulders. The effective roadway width of the bridges is 24.5 ft. The injury accidents[45] and the total number of accidents[46] were reduced. The severity of the accidents was also reduced as reflected by the SI drop and the increasing percentage reductions of accident rates as severity increases.

Miscellaneous Projects—Three miscellaneous guardrail projects were submitted and are discussed separately. One project involved guardrail placed on the inside and outside of reversing curves. Also some frequently struck trees were removed and advisory speed (W46R) signs with oversize curve (W3R, W4R) signs were installed. Total

[40] χ^2 at 1 df = 6.50; $P < 0.02$.
[41] χ^2 at 1 df = 8.72; $P < 0.01$.
[42] χ^2 at 1 df = 6.68; $P < 0.01$.
[43] χ^2 at 1 df = 8.13; $P < 0.01$.
[44] χ^2 at 1 df = 3.20; $P < 0.10$.
[45] χ^2 = 5.15 at 1 df; $P < 0.05$.
[46] χ^2 = 6.36 at 1 df; $P < 0.02$.

Figure 8. Metal plate (left) and metal beam (right) guardrail.

accidents were reduced[47] from 33 to 12 with equal exposure in both periods. The problem was mainly single-vehicle ran-off-road accidents which were reduced.[48]

The second project also had multiple improvements consisting of increasing the superelevation and placing guardrail on the outside of the curve on a narrow two-lane rural highway bridge. The guardrail was flared into a bridge structure to transition out the shoulder. Ran-off-road accidents were reduced[49] from 6 to 0 with approximately equal exposure in the before and after periods.

The third project was at a T-intersection in which vehicles from the stem of the T (State highway) were exceeding the safe speed for the left turn of the continuation of the highway and over-ran the intersection into existing buildings. Guardrail was placed in a headon position in front of the buildings at the edge of the road. Total accidents remained about the same (from 3 to 2) despite an 18 percent increase in exposure.

Metal Plate vs Metal Beam Guardrail

Three types of guardrail have been installed on California State highways. Originally, timber rails were used. Later the curved metal plate rail was installed; and, starting in 1960, the metal beam (W section) rail became the standard guardrail design.

As mentioned before, any accident reductions associated with guardrail installations can only be attributed to the delineation quality of the rail. This then is an effort to compare the delineation qualities of the two types of guardrail. The metal plate guardrail is painted white as opposed to the dull coloring of the protective zinc coating on the metal beam rail.

Metal beam guardrail posts have reflector assemblies placed facing oncoming traffic at 25-ft intervals on a radius of curvature of 1500 ft or less or where the length of guardrail is 100 ft or less. At all other locations the reflector assemblies are placed at 50-ft intervals. Figures 8 and 9 show the two types of guardrail and the beam guardrail reflectors.

Eleven projects are summarized in Table 52, with six metal plate rail installations and five metal beam guardrail improvements. The metal beam guardrail at the T-intersection is also included.

Although both types of rails caused accident reductions, all classes of accident rates or severity rates (except multiple vehicle) are reduced a greater percentage by the

[47] $\chi^2 = 8.56$ at 1 df; $P < 0.01$.
[48] $\chi^2 = 7.67$ at 1 df; $P < 0.001$.
[49] $\chi^2 = 4.89$ at 1 df; $P < 0.05$.

Figure 9. Metal beam guardrail reflector assemblies.

TABLE 52

METAL PLATE VS METAL BEAM GUARDRAIL

			PROJECTS: Total No.	PROJECTS: Improved Significantly	PROJECTS: Worsened Significantly	PROJECTS: No Change	Years of Experience	Accident Type, Single Vehicle: Hit Fixed Object	Accident Type, Single Vehicle: Ran Off Road	Accident Type, Single Vehicle: Other	Accident Type, Single Vehicle: Sub Total	Accident Type: Multiple Vehicle	Severity: PDO	Severity: Injury	Severity: Fatal	Light Con.: Day a/	Light Con.: Night a/	Total Accidents	Million Vehicles	Equivalent PDO (EPDO)	Severity Index (S.I.)
METAL PLATE	Before	No. of Accidents					7 8/12	3	13	5	21	5	11	11	4	10	17	26	17.2	101	3.9
		Rate						0.17	0.76	0.29	1.22	0.29	0.64	0.64	0.23	0.87	2.97	1.51		5.87	
	After	No. of Accidents	6	2	0	4	7 5/12	0	3 S	0	3 S	6	5	3 S	1	5	4 S	9 S	20.0	29	3.2
		Rate						0	0.15	0	0.15	0.30	0.25	0.30	0.05	0.79	0.60	0.45		1.45	
		% Rate Change						-100	-80	-100	-88	+3	-61	-53	-78	-9	-80	-70		-75	
METAL BEAM	Before	No. of Accidents					6	5	12	1	18	4	9	13	0	5	17	22	5.2	87	4.0
		Rate						0.96	2.31	0.19	3.46	0.77	1.73	2.50	0	1.44	9.83	4.23		16.74	
	After	No. of Accidents	5	0	0	5	6	2	6	5	13	1	5	9	0	6	8 S	14	5.9	59	4.2
		Rate						0.34	1.02	0.85	2.20	0.17	0.85	1.53	0	1.53	4.07	2.38		10.00	
		% Rate Change						-65	-56	+348	-36	-78	-51	-39	0	+6	-58	-44		-40	

"S" Indicates change is significant at the 0.10 level using the Chi-Square Test.
a/ Assumed 2/3 MV Day and 1/3 MV Night for rate calculations

metal plate guardrail. This is despite the fact that the various before rates of the metal beam locations are much higher than the metal plate locations. The beam rail rates had the potential for greater reductions; yet decreased a lesser amount than the plate rail rates. Therefore, either the delineation quality of the metal beam rail is less, or other improvements may be needed at these locations.

Ran-off-road,[50] injury,[51] and total accidents[52] are significantly reduced in the metal plate guardrail category. Night accident reductions occurred at both metal plate[53] and metal beam installations,[54] although this is the only classification of accidents that is significantly reduced at the metal beam rail locations. When single-vehicle ran-off-road and hit fixed object accidents (a type of ran-off-road accident) are added together

[50] χ^2 at 1 df = 6.55; $P < 0.02$.
[51] χ^2 at 1 df = 4.49; $P < 0.05$.
[52] χ^2 at 1 df = 9.96; $P < 0.01$.
[53] χ^2 at 1 df = 8.86; $P < 0.01$.
[54] χ^2 at 1 df = 3.70; $P < 0.10$.

in the metal beam category, there is a significant reduction in these accidents[55] with a rate reduction of 58 percent. The total ran-off-road accident reduction for beam guardrail is still not as great as the metal plate guardrail total ran-off-road accidents.[56] Metal plate guardrail total ran-off-road rates are reduced 84 percent.

Table 53 is a dichotomy of ran-off-road accidents comparing the severity of accidents at metal plate rail locations with that of metal beam rail locations.

Table 53 data indicates significant reductions in PDO[57] and total accidents[58] at six metal plate guardrail installations. EPDO accidents have almost been eliminated with a corresponding reduction in the severity index. Total ran-off-road accidents at five metal beam locations were significantly reduced[59] with EPDO accidents approximately halved and the SI slightly reduced.

TABLE 53

SEVERITY OF RAN-OFF-ROAD ACCIDENTS

Guardrail	MV	PDO	Injury	Fatal	Total	EPDO	SI
Metal Plate:							
Before	17.2	8	5	3	16	56	3.5
After	20.0	2	1	0	3	8	2.7
Metal Beam:							
Before	5.2	7	10	0	17	67	3.9
After	5.9	4	4	0	8	28	3.5
Total:							
Before	22.4	15	15	3	33	123	3.7
After	25.9	6	5	0	11	36	3.3

TABLE 54

GUARDRAIL VS NON-GUARDRAIL ACCIDENTS
(After guardrail installation)

Accidents	PDO	Injury	Fatal	Total	EPDO	SI
Guardrail	3	7	0	10	45	4.5
Non-guardrail	9	5	1	15	45	3.0
Total	12	12	1	25	90	3.6

TABLE 55

SINGLE-VEHICLE AND MULTIPLE-VEHICLE ACCIDENTS[a]

Accidents	PDO	Injury	Fatal	Total	EPDO	SI
Single Vehicle:						
Before	18	18	3	39	144	3.7
After	7	9	0	16	61	3.8
Multiple Vehicle:						
Before	3	4	2	9	39	4.3
After	3	3	1	7	27	3.9

[a]Before MV = 22.4, After MV = 25.9.

Guardrail vs Non-Guardrail Accidents

Table 54 dichotomizes all of the after period accidents into those involving or not involving guardrail by severity. Ten accidents involving guardrail were reported, seven of which were injury accidents. More than likely additional drive away PDO's involving guardrail occurred which were not reported.

Although the number of equivalent property damage only accidents (EPDO) is the same for both classes, the severity index (SI) or average severity of the guardrail involved accidents is higher. Therefore, if the purpose of the rail is to delineate the highway, the consequences of running off the highway and hitting the rail should be carefully weighed. For locations where the vehicle can safely travel off the highway in its projected course for a reasonable distance, a "softer" delineating material should be used. Since guardrail is an expensive delineator ($4.50 to $5.00 per lin ft), a need exists to develop an effective continuous delineation device which is softer and will permit a vehicle to pass through it without serious injury.

Care should also be taken when considering the placement of the new experimental weathered guardrail. This type of guardrail because of its aesthetically pleasing quality of blending with the landscape, is a very poor delineator. Therefore, in areas requiring delineation of the highway, especially at night, this type of guardrail would do a very poor job and accidents may ensue. A delineating material needs to be developed which, when placed on the face of the rail, would be unobtrusive in the daytime, but would provide delineation at night.

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[55] χ^2 at 1 df = 3.70; $P < 0.10$.
[56] χ^2 at 1 df = 9.52; $P < 0.01$.
[57] χ^2 at 1 df = 3.24; $P < 0.10$.
[58] χ^2 at 1 df = 9.58; $P < 0.01$.
[59] χ^2 at 1 df = 3.81; $P < 0.10$.

TABLE 56

ANALYSIS OF FIXED OBJECT COLLISION INDEX

FIXED OBJECT TYPE		NUMBER OF ACCIDENTS				EXPOSURE VOLUME (Billion Vehicles)	SEVERITY INDEX a/ (SI)	PROBABILITY INDEX b/ (PI)	COLLISION INDEX c/ (CI)
		Fatal	Injury	PDO	Total				
Bridge-rail Ends		19	79	25	123	14.35	7.9	8.6	67.9
Guardrail @ Bridge-rail Ends d/	W	16	191	199	406	40.76	4.3	10.0	43.0
Abutments & Piers		51	183	59	293	34.17	8.3	8.6	71.4
Guardrail @ Abutments & Piers	W	8	36	28	72	13.20	6.2	5.5	34.1
Light Poles		26	401	305	732	179.84	4.6	4.1	18.9
Guardrail @ Light Poles		1	23	13	37	2.20	4.8	16.8	80.6
Steel Signposts Adjacent to Shoulder		11	112	146	269	24.53	4.1	11.0	45.1
Guardrail @ Steel Posts Adjacent to Shoulder	W	1	36	31	68	15.65	4.0	4.3	17.2
Steel Sign Posts In Gore Area		7	27	17	51	1.01	7.0	50.5	353.5
Guardrail @ Steel Sign Posts In Gore Area	W	15	220	116	351	20.25	5.2	17.4	90.5
TOTAL		155	1308	939	2402	345.96	5.3	7.0	37.1
Timber Sign Posts		3	165	624	792	NA	2.1	NA	NA

a/ Based on severity weights of 25, 6 and 1
b/ PI expressed as accidents per billion vehicles
c/ CI = SI x PI
d/ "W" Warranted - on the basis of the Collision Index

Single-Vehicle and Multiple-Vehicle Severity

Eleven projects with sufficient data were dichotomized into single vehicle and multiple vehicle accidents by severity in Table 55.

Although some reductions were noted in both single and multiple vehicle accidents, only the single vehicle accidents were significantly reduced.[60] Although the average SI of single-vehicle accidents was not changed, the EPDO was reduced to less than one-half. Referring to Table 53, it is apparent that the severity of single-vehicle ran-off-road accidents at metal plate guardrail installations was reduced. However, when metal plate and metal beam guardrail projects are combined, no severity change is indicated. Small differences in total single vehicle accidents and ran-off-road accidents are caused by a small number of miscellaneous single-vehicle accidents which did not leave the road. The numbers of multiple-vehicle accidents were too small for meaningful analysis.

It appears, then, that metal plate guardrail reduces both the frequency and severity of single vehicle accidents. The sample size, however, is probably too small to generalize.

Warrants and Predictive Parameters

The numbers of individual types of installations were too small for analysis of possible warrants or for developing accident predictive parameters. However, a previous study (14) was able to determine for guardrail installations adjacent to fixed objects, the effect on accident frequency, accident severity, and/or the combined effect of frequency and severity. Because of limited research resources, the effect of guardrail on severity only was determined in the case of embankment guardrail protection.

Table 56 (14, Table 9) indicates which types of fixed objects warrant guardrail protection. The collision index is the best parameter on which to judge the overall effects of a specific fixed object type when protected with guardrail. It is simply the product of the severity index and probability index. It is on this basis that the W for warranted was placed in the "fixed object type" column.

[60] PDO χ^2 at 1 df = 5.61; $P < 0.02$; Inj χ^2 at 1 df = 3.69; $P < 0.10$. Total χ^2 at 1 df = 12.33; $P < 0.001$.

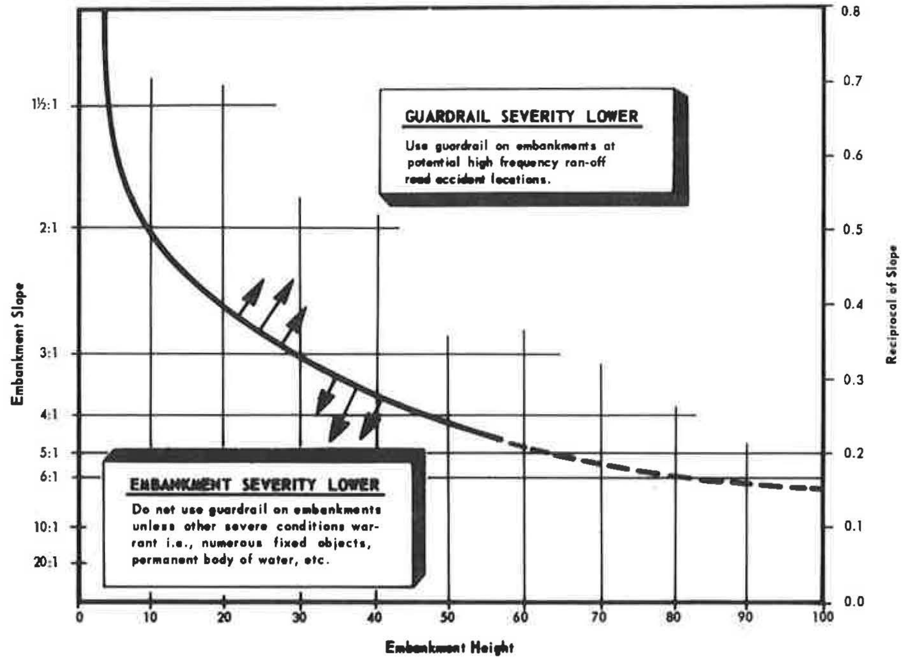

Figure 10. Severity comparison of embankment vs guardrail.

The number of accidents can be predicted using the probability index. For instance, if guardrail is installed at bridge rail ends, one could expect 10.0 accidents per billion vehicles driving by the bridge end, an increase of 1.4 accidents per billion vehicles of exposure, although the respective SI's show a reduction in severity of almost one-half. In the case of guardrail placed adjacent to piers, 5.5 accidents per billion vehicles can be predicted, a decrease of 3.1 accidents per billion vehicles.

Figure 10 (14, Fig. 5) can be used to assess on a severity basis only, whether guardrail might be warranted on embankments. Accident frequency cannot be predicted from Figure 10.

Conclusions and Recommendations

The numbers of projects and accidents were considered too few to make generalizations and more data are needed. Consideration should be given to additional research at protective guardrail installations. However, the following conclusions and recommendations were determined from the available data.

1. Protective guardrail has been quite effective in reducing accident rates at locations of poor alignment or at two-lane bridges whose widths are less than the approach width of the highway. Specifically, night accidents and single vehicle ran-off-road accidents were reduced.
2. In the after period, accidents involving guardrail have a higher average severity than non-guardrail accidents.
3. The delineation quality of the metal plate guardrail appears to be greater than that of metal beam guardrail as evidenced by greater accident reductions. Single-vehicle accident severity was reduced only at metal plate rail installations. It is recommended that at locations where ran-off-road accidents predominate and guardrail warrants are satisfied, consideration be given to enhancing the delineation qualities of the metal beam guardrail either by painting or by some other means. There is a need to develop a new material which, when placed on the guardrail face, will be unobstrusive in the daytime, yet act as a delineator at night.

4. It is recommended that a continuous delineation device be developed which would allow out-of-control vehicles to pass through it without serious injury to the vehicle. This device would be placed at locations where the accident severity of running off the road would be less than hitting a standard guardrail. It should be less expensive than metal beam guardrail.

ACKNOWLEDGMENT

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Appendix

Warrants for Traffic Control Signals

WARRANTS	Urban Conditions	Rural Conditions
(1) *Minimum Vehicular Volume**		
(a) The total vehicular volume per hour entering the intersection from all approaches for any 8 hours of an average day must average	750 veh.	500 veh.
(b) In addition, the total vehicular volume per hour entering the intersection from the minor street or streets for the same 8 hours must average	175 veh.	125 veh.
(2) *Interruption of Continuous Traffic* *		
(a) The vehicular volume per hour entering the intersection on the major street for any 8 hours of an average day must average	750 veh.	500 veh.
(b) In addition, the combined vehicular and pedestrian volume per hour entering the intersection from the minor street or streets for the same 8 hours must average	75	50
(c) And, the average vehicular speed on the major street must exceed	20 mph.	35 mph.
(3) *Minimum Pedestrian Volume* *		
(a) The pedestrian volume per hour crossing the major street for any 8 hours of an average day must average	250 peds.	125 peds.
(b) In addition, the vehicular traffic per hour entering the intersection from the major street for the same 8 hours must average	600 veh.	300 veh.
(c) And the average vehicular speed on the major street must exceed	15 mph.	30 mph.
(4) *Coordinated Movement* A coordinated signal system may be warranted if a majority of the signalized intersections composing the system comply with one or more of the established warrants, and if the system fits an over-all time-space diagram. Signals at an intersection may be warranted as part of a coordinated system if they fit into an existing time-space diagram.		
(5) *Accident Hazard* Five or more reported accidents of types susceptible of correction by a traffic control signal have occurred within a recent 12-month period.		
(6) *Combination* Where no one warrant is satisfied but two or more are satisfied to the extent of 80 percent or more of each of the stated values.		

NOTES:

1. Left turn movements from the major street may be included with minor street volumes if a separate signal phase is to be provided for the left-turn movement.
2. Accidents susceptible of correction by a separate left-turn signal phase may be included in the Accident Hazard Warrant.

* Also presently used for safety lighting

Development of Techniques for Analysis of Operation of Major Interchanges

JOSEPH A. WATTLEWORTH, CLAUDE ARCHAMBAULT, and
CHARLES E. WALLACE, Texas Transportation Institute

ABRIDGMENT

•A great deal of work has been done on the analysis of freeway sections and freeway systems. This work has led to the development of freeway control systems which can result in substantially higher levels of service on freeways. This work has concentrated primarily on sections containing only the normal interchanges in which the operation was unaffected by major external features.

In large urban freeway networks the operation at major interchanges (freeway-freeway interchanges) is becoming an overriding problem. Because of the volumes involved in the movements at major interchanges, the problems associated with many of these interchanges are quite severe. The purpose of the paper is to present some preliminary attempts at developing an analysis technique which can be used to evaluate the operation of major interchanges. The paper identifies the factors which most frequently lead to operational problems at major interchanges. The measures of effectiveness of major interchange performance are also presented.

The paper presents an investigation of two analysis techniques which were tested on the Lodge-Ford and Lodge-Davison freeway-freeway interchanges in Detroit. These were (a) an input-output technique and (b) an aerial photographic technique. Some further thoughts on combining these into a single procedure for the analysis of major interchanges are given.

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