THE ADAPTABILITY OF THE TRAFFIC CONFLICTS TECHNIQUE TO THE URBAN OFF - RAMP SITUATION

Thesis for the Degree of M. S.

MICHIGAN STATE UNIVERSITY

DONALD JAMES MERCER, P. E.

1974

A B S T R A C T

THE ADAPTABILITY OF THE
TRAFFIC CONFLICTS TECHNIQUE
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by

Donald James Mercer, P.E.

The Traffic Conflict Technique, developed to study the operation of intersections, is expanded and tested at six off ramps, three of parallel design and three of taper design.

Nine types of conflicts are tabulated over a total of ninety minutes at each ramp, using video tape. The mean numbers of total conflicts per minute is compared to the accident rates at those ramps.

The Traffic Conflict Technique is found to be 94 percent reliable in measuring the quality of operation of urban off ramps. The conflict rate is found to increase as the flow increases. No differences in operation are found between parallel and taper ramps. A procedure for conducting future conflict measurements at urban off ramps is presented.

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Donald James Mercer, P.E.

A THESIS

Submitted to
Michigan State University
in partial fulfillment of the requirements
for the degree of

MASTER OF SCIENCE

Department of Civil Engineering

1974

A C K N O W L E D G E M E N T

Thanks are due to the Michigan Department of State Highways and Transportation for providing the data used in this project and for the use of its data processing equipment to help reduce the data.

Many employees of the Department were consulted in the progression of this work. Each willingly contributed all that was asked of him and more, the net result being a far better paper than otherwise would have been written; if one could have been written at all.

Mr. Gail Blomquist has been most helpful to me on this project as well as all my work at Michigan State University.

Special thanks go to the one who has been my prime encourager, my editor, my typist, my proofreader, and, temporarily, my widow on this project: my beloved wife Mary JoAnn.

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CHAPTER 1

INTRODUCTION

THESIS

The following thesis is proposed:

> The Traffic Conflicts Technique developed by Perkins and Harris to measure accident potential at intersections can be adapted to measure the quality of operation of urban off ramps. If so, the technique can be used to detect any significant differences between the operations of off ramps of the parallel design and those of taper design.

This thesis is tested in this work by tabulating the conflicts by number and by type, that occurred at six different urban off ramps. Three of these ramps were of parallel design and three were of taper design. The two-year accident experiences of the ramps are calculated, in Accidents per 100-Million Vehicles, on the premise that the accident rate is symptomatic of the quality of operation. The goal of this work is to find a meaningful relationship, if there is any, between the conflict rates and the accident rates at these six ramps. In addition, the data are analyzed to determine any relationships that exist between the various traffic flow rates and the rate of conflicts.

BACKGROUND

The question of which type of off-ramp design best serves the driver has been debated among highway engineers for a number of years; it is currently being posed by the Michigan Department of State Highways and Transportation.

The parallel design, which has a short added lane before the gore, provides abundant deceleration length off the thru lanes, but the added pavement area can induce erratic movements. The taper design, which leaves the thru lanes directly, forces the driver into a stereotype path; but it also provides him with a small target and may cause him to slow excessively on the thru lanes.

Numerous studies of the two types of ramps have generally favored the taper design, based on the path that is followed by the driver. Those studies (see Chapter 2) found that at parallel ramps the driver tended to follow a long flat taper rather than drive the path presented by the pavement. So, it is usually concluded, it is better to provide the driver with the path he wishes to drive. Accident data has not shown a significant difference between the two types. But accident data may be too insensitive to detect subtle changes in driver performance that may result from the difference in the ramps.

The path-driven argument is discounted by those who favor the parallel design. They argue that the greater target presented to the driver by the parallel ramp is of more importance. They feel that the small target and the short distance to maneuver provided by a taper ramp leads to erratic movements in the thru lane by the exiting driver.

To help resolve these differences, the Michigan Department of State Highways and Transportation is conducting a study of the operations of the two types of urban off ramps. The thesis presented in this work was developed during the conduct of the Department's study. It was determined that the thesis would be tested by this work; the results of this work and the data used will be used by the Department as one aspect of its study.

CHAPTER 2

LITERATURE REVIEW

Being that off ramps are a major feature of a limited-access highway and are apt to create friction in the traffic stream, numerous studies have been conducted to find the optimum design of some aspect of off ramps. Seven have attacked the taper versus parallel design question.

The first of these was Conklin's study of two rural off ramps in Oregon (1). One of these was a taper ramp with a deflection of 4°10', providing a 530 foot opening, reduced to 280 feet by paint lines. The other was a 470 foot parallel ramp that was followed by a 138 foot radius curve. Conklin measured speed and placement of exiting vehicles. He found a 22 mph reduction (45.5 to 23.5 mph) at the parallel ramp and a 3 mph reduction (49 to 46 mph) at the taper ramp. On lateral placement, he found that nearly all exit vehicles were off the thru lane at the midpoint of the taper ramp, compared to less than half for the parallel ramp. Only 20 percent of the exiting vehicles at the parallel ramp left the thru lanes in the first 200 feet. From this, Conklin concluded that the taper ramp was "definitely superior" (1, p. 16) to the parallel ramp, both in speed of operation and placement of vehicles. It should be noted that he actually compared a normal taper ramp to a substandard parallel ramp, and his work compared not only the ramps up to the gore but also the curvature beyond the gore.

Pinnell and Keese (13) studied ten ramps, both on and off ramps, in Texas. Their work concentrated on the on ramps, but they pointed out that at a parallel ramp 5 percent of the vehicles used the ramp as designed, 35 percent followed a direct taper path, and the remaining 60 percent made a delayed move onto the ramp. They concluded that "This lack of usage [of the added lane] is related to the exit ramp driver's desire to follow a natural and easy path." (13, p. 57)

Fisher (3) evaluated the accident experience and operation of New Jersey freeways. He found that "nearly all drivers will use a deceleration lane of the parallel type if it is 1200 feet long. When the length is decreased to less than 800 feet, some drivers will not use them and the accident rate is increased" (3, p. 130). The three taper ramps he studied were extremely short, not typical of current design practices.

Jouzy and Michael (5) studied speed and placement of vehicles on several designs of on and off ramps in Indiana. All off ramps were tapers. They found that drivers began to decelerate on the thru lanes more than 1000 feet in advance of the ramp. They theorize that drivers "desired to follow a natural straight path of exiting with a minimum of maneuvering" (5, p. 51). They also observed that ramps with almost identical geometrics had different patterns of vehicle behavior. They favored a 1200-foot taper.

Lind and Hong (6) analyzed the accident experience on Milwaukee's expressways. Concerning off ramps, all of which were tapers, they found no correlation between the design and any type of accident (6, p. 44). They also noted "Drivers appeared to decelerate slightly on thru lanes and will not move over to the deceleration lane until they have a good view of the ramp nose or exit ramp, or both"(6, p. 44).

Davis and Williams measured headways, speeds, lateral placement, and deceleration rates for six parallel ramps in Toronto (2). They found that the drivers entered the ramp early and followed a long taper path and that drivers were not clearing the thru lane before decelerating. Therefore, "A direct-taper type of exit would seem to be indicated since it would appear to fit the vehicle paths better than the taper plus added parallel deceleration lane" (2, p. 73).

Mercer completed a study of rural off ramps in Michigan (8), comparing driver behavior at parallel and taper ramps on the basis of speed reduction, path driven, and accident rates. He found speed reductions of about 7 mph at four parallel ramps and between 9 and 13 mph at three taper ramps; the difference at the taper ramps was significantly greater. That study produced more evidence that the parallel ramps are driven with a taper path. There was no difference found between the accident rates at the two types of ramps.

Other studies on ramp design included Fukutome and Moskewitz (4) work in California, intended to determine the optimum length of tangent. They found that exiting vehicles began to decelerate 135 to 220 feet ahead of the beginning of the ramp. If there is a surplus deceleration distance, drivers maintained their speed for the first part of the ramp, then decelerated.

Taylor concluded that "The direct-taper deceleration lane is operationally superior to the parallel-lane type" (14, p. 22); based on a review of the same literature as discussed here, principally Conklin's and Jouzy and Michael's works. Taylor, in another aspect of his work, defined eight "erratic movements" at ramp gores (14, p. 3). In general, these were not as sensitive as the conflicts developed for the Traffic Conflicts Technique.

Pahl (9) found that thru vehicles approaching an off ramp tend to move to the left, then return to the right at the gore.

Tipton, Carrell, and Pinnell (15) argue that "The fact that parallel deceleration lanes are not driven as constructed may not necessarily be a bad feature" (15, p. 12). The parallel lane, they feel, has an "advantage under high density conditions . . . ." because it can "offset undesirable geometric features . . . ." (15, p. 12).

Martin, Newman, and Johnson (7) add the comment that "Congestion, if present, will usually occur upstream of the off ramp due to lane changing and overloading of Lane 1 by vehicles desiring to use the off ramp." (7, p. 29)

It is arguments such as presented in the last two references that are the basis of the feeling of some engineers that parallel ramps will provide better performance on urban freeways.

The criteria generally used to compare the operations of the two types of ramps have been vehicle placement, speed, and accident data. But a satisfactory answer to the parallel versus taper question is yet to be found. The Traffic Conflicts Technique has been tested and proven worthy for several applications (18) at intersections, thus it has been proposed to use that method to again study the urban off-ramp question. That criterion may be more sensitive to differences in operation than are the others and can be easily measured with the equipment on hand at most highway agencies.

Before the technique can be used, it is necessary to first determine if it will produce meaningful results. It is for that purpose that this work has been undertaken.

CHAPTER 3

PROCEDURE USED

TRAFFIC CONFLICTS TECHNIQUE

The procedure used to obtain data for this work is an expansion of the Perkins-Harris technique for detecting conflicts at intersections. In the abstract of the Procedures Manual, Perkins wrote:

> The Traffic Conflicts Technique was developed . . . to be a measure of traffic accident potential. A Traffic Conflict occurs when a driver takes evasive action, brakes or weaves, to avoid a collision. The evasive action is evidenced by a brakelight indication or a lane change by the offended driver. (12 p ii)

Perkins and Harris found that ". . . a high level of association exists between the traffic conflict and reported accident frequencies. In particular, high accident frequencies are always associated with high conflict frequencies." (11 p 22)

Most of their work was done on intersections. One study was conducted at a freeway curve and exit area, for which they defined different conflicts than were used at intersections. (10) These conflicts, with others added, were used in this work. (Figure 1)

Slow Vehicle. A slow vehicle is one that appreciably slows by braking for no apparent reason as it approaches the off ramp. Such a braking indicates that the driver has lost confidence in his ability

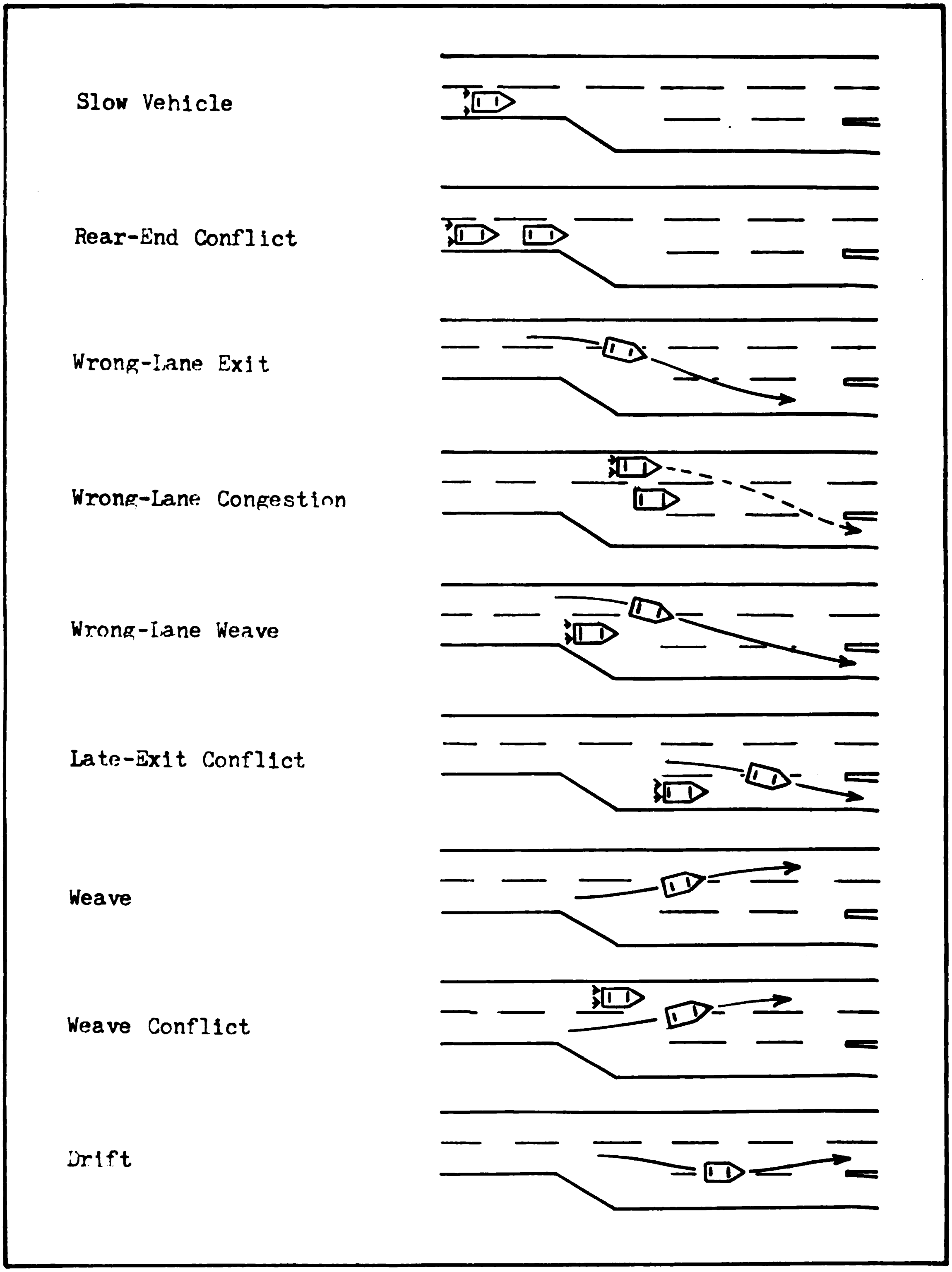


Figure 1.
Diagrams of Conflict Types

to negotiate the roadway immediately ahead at his current speed. Since all ramps studied were designed to accommodate an in-gear deceleration rate, this braking action is an erratic move.

A Slow-Vehicle Conflict occurs when there is a brakelight indication and there is no external stimulus to warrant the braking. Whether the vehicle exited is also noted.

Rear-End Conflict. A rear-end conflict is the situation of one vehicle appreciably slowing, resulting in a following vehicle braking to avoid collision.

A Rear-End Conflict occurs when there is a brakelight indication on a following vehicle. There is only one Rear-End Conflict per incident, even if more than one following vehicle brakes. The lead vehicle is also recorded as a Slow Vehicle if the criteria listed above apply.

Wrong-Lane Exit. A wrong-lane exit is an exit movement by a vehicle that begins in some lane other than the outside lane. The driver may have gotten into an inside lane for several reasons: he may have been confused, not realizing that his exit was so near; he may have gotten trapped, unable to make a safe weave into the outside lane; or he may have been attempting to pass the slower moving outside lane and exited from the inside lane deliberately.

A Wrong-Lane Exit occurs when a vehicle makes a direct move from an inside lane to the ramp. Such a move may precipitate two other types of conflicts:

Wrong-Lane Congestion. A Wrong-Lane Congestion conflict is the situation of a wrong-lane exit vehicle being unable to make his move smoothly due to a vehicle in the outside lane and being forced to slow to allow the other vehicle to cross his intended path.

A Wrong-Lane Congestion Conflict occurs when a Wrong-Lane Exit vehicle applies its brakes and allows another vehicle in the lane to its right to go ahead of it.

Wrong-Lane Weave. A wrong-lane weave conflict is the situation of a wrong-lane exit vehicle crossing directly in front of another vehicle, causing that vehicle to apply its brakes to avoid collision.

A Wrong-Lane Weave Conflict occurs when a Wrong-Lane Exit vehicle crosses a lane to his right and the first vehicle directly behind him applies its brakes.

Late-Exit Conflict. A late-exit conflict is the situation of one vehicle entering the off-ramp upstream from another vehicle that is on the ramp, resulting in the second vehicle applying its brakes.

A Late Exit Conflict occurs whenever one vehicle passes and then enters the exit ramp ahead of a second vehicle, resulting in a brake-light indication from the second vehicle.

Weave. A Weave is a complete lane change to the left, either from the outside lane to an inside lane or from the ramp to the outside lane. Such moves are considered to be erratic moves, the result of a rear-end or slow vehicle conflict or confusion by the drivers. Lane changes to the right were not considered weaves. Such moves are commonplace near and upstream from the gore as the traffic redistributes itself as a result of the vehicles lost at the ramp.

A Weave occurs when a vehicle makes a complete lane change to the left, either from the outside lane to the inside lane or from the ramp to the outside lane. Such a move can precipitate another conflict:

Weave Conflict. A weave conflict is the situation of a Weave vehicle making its move so close in front of another vehicle that the second vehicle must brake to avoid a collision.

A Weave Conflict occurs when a Weave, as defined above, occurs and the first following vehicle in the lane entered has a brakelight indication.

Drift. A Drift is a partial lane change, in which the vehicle crosses partly into the adjacent lane or the ramp and then returns to its original lane. This is an erratic movement, that is expected primarily at those ramps at which the thru lanes are on a curve.

A Drift occurs when a vehicle encroaches onto an adjacent lane or the ramp and then returns to its original lane.

DATA COLLECTION TECHNIQUE

Data for this work were obtained by use of video tape. A camera was set up on a structure, if possible, or on the slope and set to view the gore and at least 500 ft upstream from it. Due to equipment limitations, only 30 minutes could be taped at one time. At least three such tapes were taken at each of the six study sites; two during near peak periods and one during an off-peak period.

The data were taken from the tapes in the office. Conflicts were recorded by type and by time of occurrence to the nearest 0.1 minute as measured by the meter on the tape deck. Volumes were also recorded for each minute in three different listings: exit volume, outside lane volume, which included the vehicles that exited, and the sum of all remaining lanes. In taking data from the first tapes at most sites, volume data were taken for only a "typical" 10-minute period. Thus, for most sites there were 90 minutes of conflict data and 70 minutes of volume data available for analysis.

Accident data were obtained from the Department of State Highways and Transportation files for 1971 and 1972 for each ramp. It is difficult to ascertain from an accident report exactly what factors triggered the incident and where the incident actually began. So only general limits were used: all accidents that occurred along the ramp-thru lane interface or about 200 ft on either side of the interface were used, with no attempt to determine whether an exit maneuver was involved.

Volume data were obtained from the Department of State Highways and Transportation records for 1971; one-half of the two-way average daily traffic was used in calculating accident rates.

STATISTICAL ANALYSIS

The data from all three 30-minute observations were combined to form a data file for each study site. These files contained the following information for each individual minute of observation:

1. Traffic Flow
   - a) for the inside lane(s)
   - b) for the outside lane, including exiting vehicles
   - c) for exiting vehicles
   - d) total flow
2. Number of Occurrences of each type of conflict
   - a) Slow Vehicle
   - b) Rear-End Conflict
   - c) Wrong-Lane Exit
   - d) Late-Exit Conflict
   - e) Weave
   - f) Total Conflicts

Four types of conflicts occurred a total of no more than twice in all observations at all six sites, and were not included in the statistical analysis: Wrong-Lane Congestion, Wrong-Lane Weave, Weave Conflict, and Drift.

The basic statistics (mean value per minute and standard deviation) were found for each of the ten items listed above.

To test the first point of the thesis, various conflict rates and flow rates were calculated and compared to the accident rates at the six ramps (in Accidents per 100-Million Vehicles):

1. Mean number of Conflicts per minute
2. Number of Conflicts per exiting vehicle
3. Number of Conflicts per total flow
4. Number of Conflicts per average flow per thru lane
5. Mean number of conflicts per minute per proportion exiting
6. Mean number of conflicts per minute times the proportion exiting
7. Total flow in vehicles per minute
8. Exit flow in vehicles per minute
9. Outside Lane flow in vehicles per minute
10. Inside Lane flow in vehicles per minute

The reliability of the first point of the thesis was determined by the significance level of the hypothesis: The slope of any of the above relationships is not zero. The slope of the relationship having the greatest potential was so tested, using the <u>t</u>-statistic.

The slope of the regression equation for the taper ramps was similarly tested against the slope of the equation for the parallel ramps.

The frequencies of occurrence of the different types of conflicts were compared to the frequencies of occurrence of the different types of accidents.

To determine if the rate of conflict per minute is dependent on the various flow rates and to determine if the number of exiting vehicles per minute could be predicted by counting the thru flow, six potential linear relationships were investigated:

Total Conflict = f(thru flow, exit flow)

Total Conflict = f(outside lane thru flow, exit flow)

Total Conflict = f(exit flow)

Total Conflict = f(total flow)

Exit Flow = f(total flow)

Exit Flow = f(outside lane flow)

To compensate for the high frequency of minutes with 0 conflicts, the regression equations were calculated three times for each ramp at one-minute, two-minute, and three-minute increments.

The multiple linear equations were tested against the null hypothesis: either independent variable is not significant. The linear equations were tested against the null hypothesis: R is not different than zero. The minimum values for R to cause rejection of that null hypothesis are a function of the sample size; for the sample sizes used in this work, they are:

Sample Size (min.)	R	R^2
90	0.203	0.041
70	0.235	0.055
68	0.238	0.057
45	0.294	0.086
35	0.333	0.111
34	0.338	0.114
30	0.360	0.130
23	0.412	0.170
22	0.422	0.178

Because the number of occurrences of each type of conflict was small, (most commonly 0, to a maximum of 6) there would be no meaningful correlation between individual types of conflicts and traffic flows. Because the total conflicts per minute was also low (no more than 12) and the ratio of conflicts to flow was also small (generally less than 1:10), it was expected that there would be low correlation between conflicts and the flow.

Because the increase in the flow rate may, through increased congestion, breed additional conflicts, the relationship between conflicts and flow may be curvilinear. It cannot be exponential (Conflicts = B x $(\text{Flow})^A$) due to the high frequency of 0 conflicts per minute. So it was determined to compute a second-degree polynomial equation:

$$\text{Conflicts} = B_0 + B_1 \times (\text{Flow}) + B_2 \times (\text{Flow})^2$$

This equation was viewed subjectively to determine if it appeared to be a better predictive equation than the linear equation.

The computations for these analyses were performed by the Michigan Department of State Highways and Transportation's Burroughs B5700 computer. The basic statistics and linear regression were computed using the BASIS (Burroughs Advanced Statistical Inquiry System) package. A separate program was written by the author (in FORTRAN IV) to solve for the second-degree polynomial equations.

CHAPTER 4

S T U D Y S I T E S

BASIS FOR SELECTION

Six different urban off ramps, three of parallel design and three of taper design, were studied in this work. The criteria for selection were:

1. There should be no unusual alignment on either the ramp or the thru lanes that could induce erratic behavior by the driver.
2. There should be sufficient deceleration distance on the ramp to allow drivers to decelerate in gear to the ramp speed after they completely clear the thru lanes.
3. There should be a point about 1000 ft upstream from the gore at which the video camera could be placed to give adequate coverage of the ramp-thru lane interface.
4. During the observation period, there should be no congestion on either the ramp or the thru lanes that would result in conflicts other than those to be analyzed.

As the design practices of the Department have evolved, all the ramps of the Detroit area freeway system are of the taper design and all suitable ramps in other Michigan metropolitan areas are of the parallel design. This introduces a factor into the data: the sites used to represent the different types of ramps also represent different driving populations.

PARALLEL RAMPS (Figure 2)

P.1 I 496 EB to US 127 NB, Lansing

P.2 I 496 EB to Walnut Street, Lansing

P.3 I 496 WB to US 27 SB, Lansing

TAPER RAMPS (Figure 3)

T.1 I 75 NB to 7 Mile Road, Detroit

T.2 US 10 NB to Meyers Street, Detroit

T.3 I 75 SB to M 39, Lincoln Park

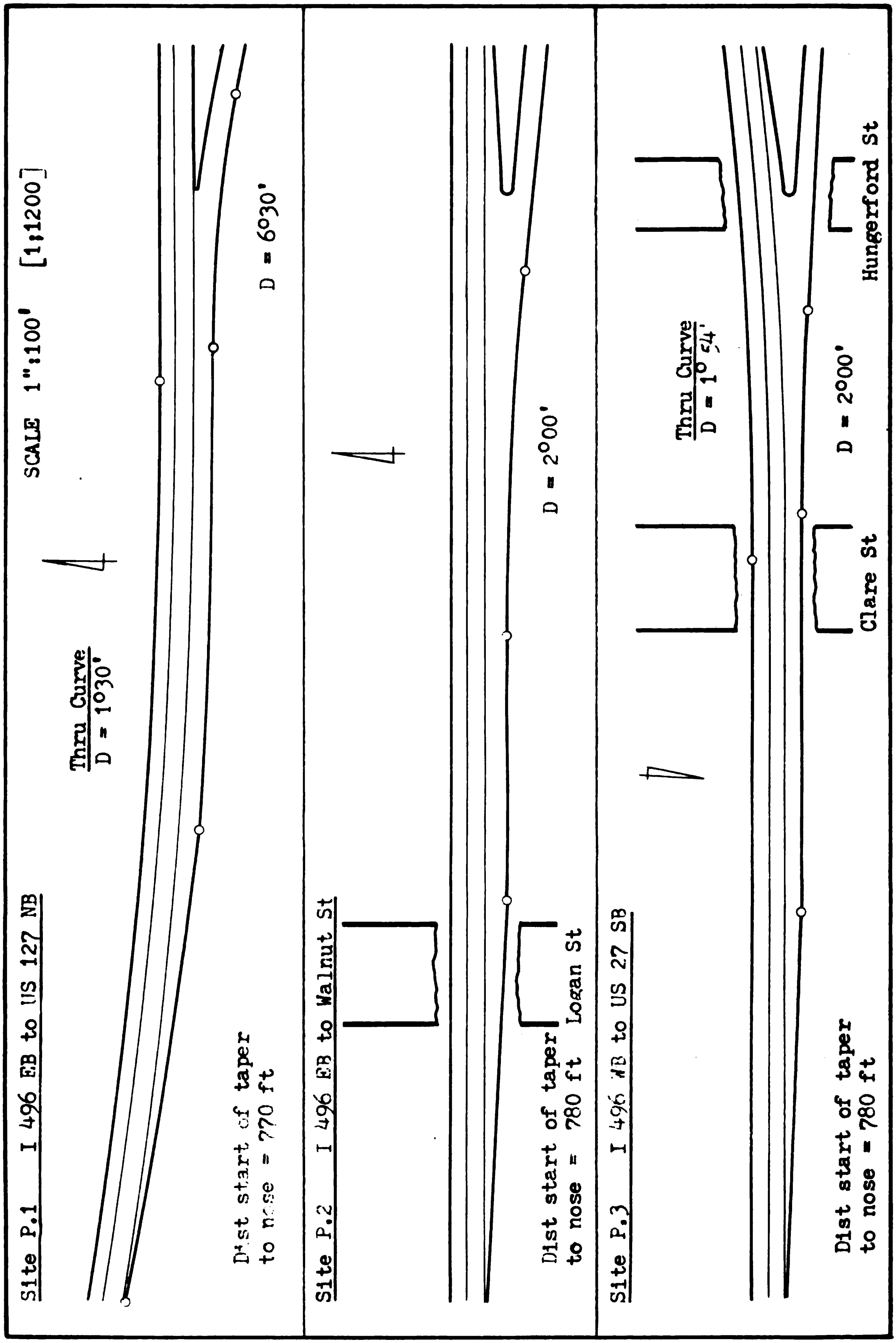


Figure 2
Alignments of Parallel Ramps

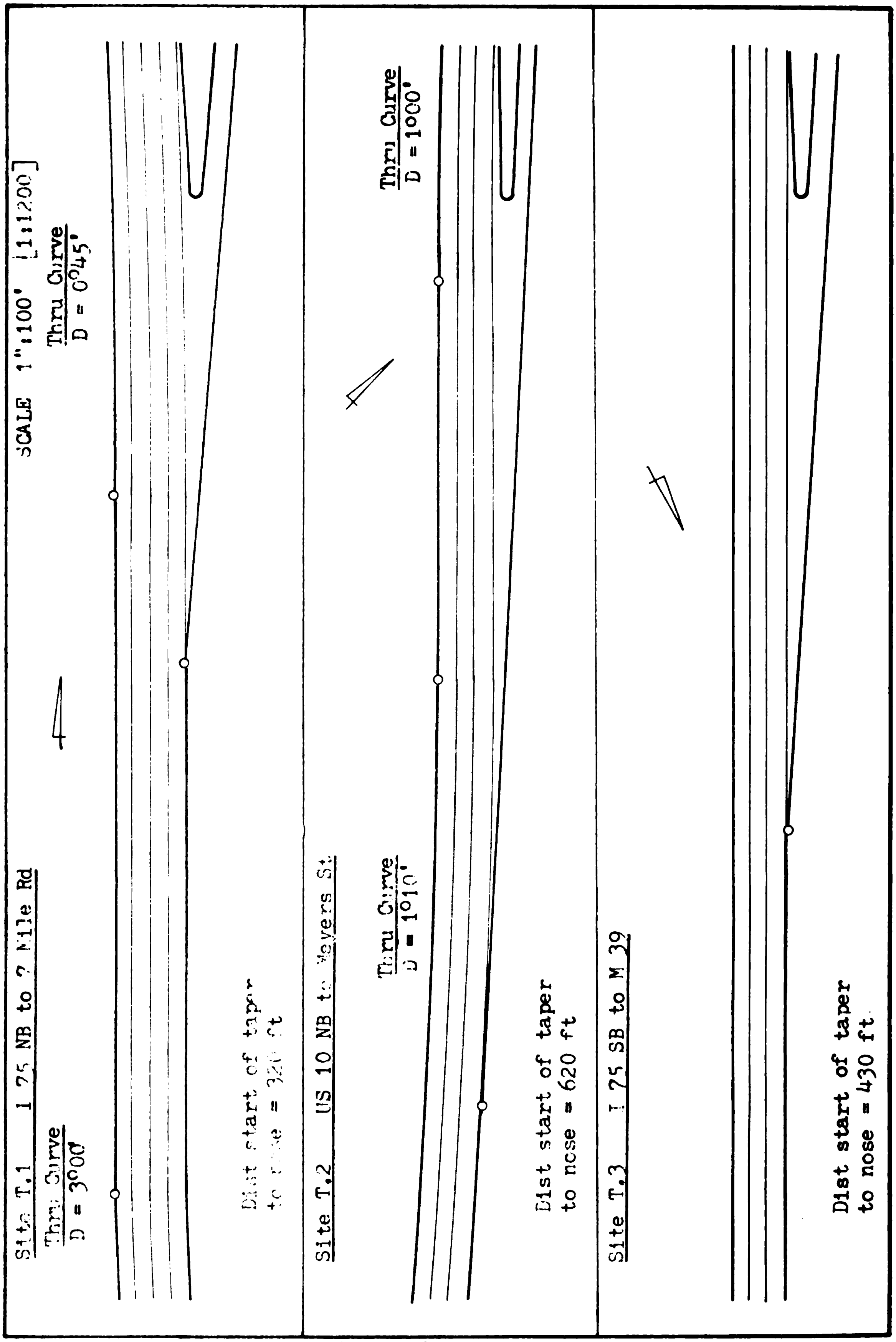


Figure 3
Alignments of Taper Ramps

CHAPTER 5

RESULTS

The distribution of each type of conflict by frequency per minute, the various flow rates, and the predictive equations for conflicts and exits as a function of flow are given in Appendix 1.

The two-year accident experience and collision diagram for each ramp is given in Appendix 2. The accident rates are tabulated on page 50.

CONFLICT/ACCIDENT CORRELATION

For the ten methods of measuring the conflict rate, the following linear relationships were found:

$$\text{Acc}/100\ \text{MV} = AX + B$$

where X = Conflict rate

Conflict Rate	A (Slope)	B (Constant)	Stand. Error	R	R^2
Mean Conflicts per min.	36.0435	- 5.5680	19.875	0.803	0.65
Conflicts per exit	303.8470	10.3413	29.937	0.442	0.20
Conflicts per Total Flow	656.9650	19.4583	27.838	0.552	0.30
Confl/min/mean flow	367.3205	12.2242	21.746	0.603	0.36
Confl/min/prop exit	0.7919	40.5401	33.335	0.050	0.002
Confl/min x prop exit	51.1202	19.9824	22.082	0.750	0.56
Total Flow per min.	- 0.2899	57.7708	26.326	0.258	0.07
Exit Flow per min.	3.1947	6.4614	23.911	0.480	0.23
Outside Lane Flow per min.	1.0476	23.9197	26.756	0.190	0.04
Inside Lane Flow per min.	- 0.3945	55.0074	25.793	0.323	0.10

The level of significance for the slope of the first equation was calculated using the <u>t</u>-test; that test assumes that the sample comes from a normal population. The calculated value was t = 2.70, which corresponds to a level of significance of more than 94 percent (Appendix 2, p. 51).

Because of the high correlation found for the sixth equation, the possibility that the proportion exiting is a significant factor was investigated by calculating a multiple linear regression line for the equation:

Accident rate =f(Conflicts per minute, Proportion Exiting)

From that calculation, it was found that the proportion exiting is not a significant factor.

The easiest variable to measure are the various flow rates. For that reason, the regression lines for accident rates as a function of the per-minute flow rates were calculated; but those relationships were not significant.

The linear regression found above is based on the assumption that the independent variable (Conflicts per minute) is an absolute value and the dependent variable (Accidents per 100 MV) is an estimate, with a mean and variance and was calculated by minimizing the vertical distance between the data points and the regression line. In truth both sets of data are estimates; to account for this an orthogonal regression line was calculated. This method finds the line that minimizes the perpendicular distance from the line to each data point. The orthogonal linear regression line found was:

Acc/100 MV = 55.8388 x (Confl per min) - 32.9073

Both regression lines are estimates of the true regression line between all six data points.

The data points, both regression lines, and confidence interval for the simple linear regression are plotted in Figure 4.

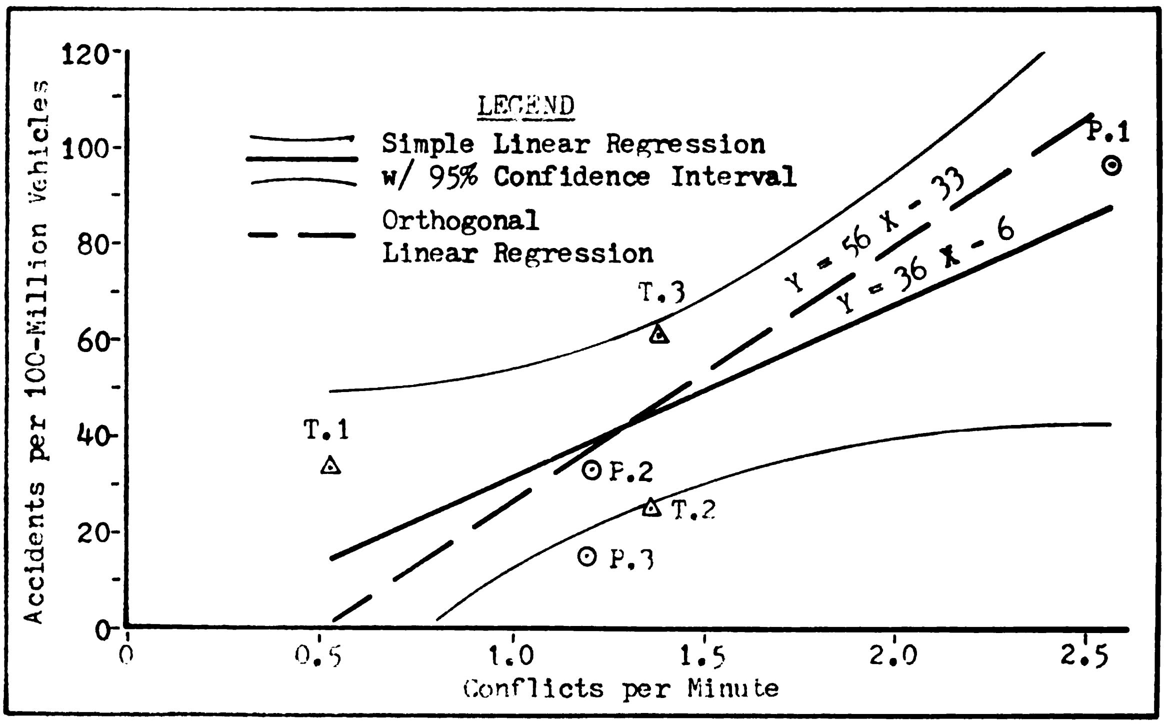


FIGURE 4.
Linear Regression Plots for Acc Rate = f(Conflict per minute)

CONFLICT TYPE/ACCIDENT TYPE COMPARISON

The data were further analyzed to determine if the types of conflicts that were observed were indicative of the types of accidents that occurred. For this portion of the study, the sum of the rates of Slow-Vehicle Conflicts, Rear-End Conflicts, and Late Exits were compared to the proportion of Rear-End accidents; and the sum of the rates of Wrong-Lane and Weaves were compared to the proportion of Angle Accidents. None of the conflict types were considered to be indicative of run-off-roadway accidents. The data are tabulated below:

Ramp	Prop. of Conflicts SV+RE+LE	WL + W	Proportion of Accidents Rear-End	Angle	Off-Road
P.1	0.69	0.31	0.42	0.08	0.50
P.2	0.79	0.21	0.50	0.25	0.25
P.3	0.49	0.51	0.50	0.00	0.50
T.1	0.69	0.31	0.42	0.33	0.25
T.2	0.54	0.46	0.40	0.30	0.30
T.3	0.50	0.50	0.57	0.29	0.14

There are no correlations evident from that data.

TAPER RAMPS/PARALLEL RAMPS COMPARISON

The linear regression lines for the three parallel ramp data points and for the three taper ramp data points are:

	Standard Error	R^2
Parallel:		
Acc/100 MV = 52.73 x Conf/min. - 39.16	6.91	0.96
Taper:		
Acc/100 MV = 12.48 x Conf/min - 26.15	14.35	0.11

Because of the small number of data points, the difference in those two slopes cannot be shown to be significantly different ($t = 1.13$, while $t_{.95}$ w/1 DF = 12.7).

One goal of this work is to determine if there is any difference between the quality of operation of Taper and Parallel design of urban off ramps. Since the basis for measuring that quality of operation is the accident rate at each ramp, it would be incorrect to attempt to use that same relationship to test those same six ramps. The operational difference sought-after would be evident only if one type of ramp had significantly higher accident rates than did the other. That phenomenon did not occur at the six test ramps.

The taper design ramps had a higher proportion of angle accidents; this may be due to the higher volumes at the taper ramps and so it is not at this time considered significant.

CONFLICTS AS A FUNCTION OF FLOW

At four of the six study ramps, there was a significant relationship between the rate of conflicts per minute and the flow rates. Using two- and three-minute increments generally produced slightly higher correlation coefficients. For each set of ramp data, the various slopes tended to remain constant in the three calculations.

Although there are significant linear relationships between conflicts and flows, the correlation coefficients are low, meaning that the linear equation should not be used to predict the number of conflicts.

The second-degree polynomial equations calculated generally produced a U-shape curve. This is the result of attempting curve-fitting on poorly related data; it does not indicate that there is an optimum flow rate to achieve a minimum number of conflicts.

The data points recorded and the linear and second-degree regression equations calculated for one-minute increments are shown in Figures 5 through 10.

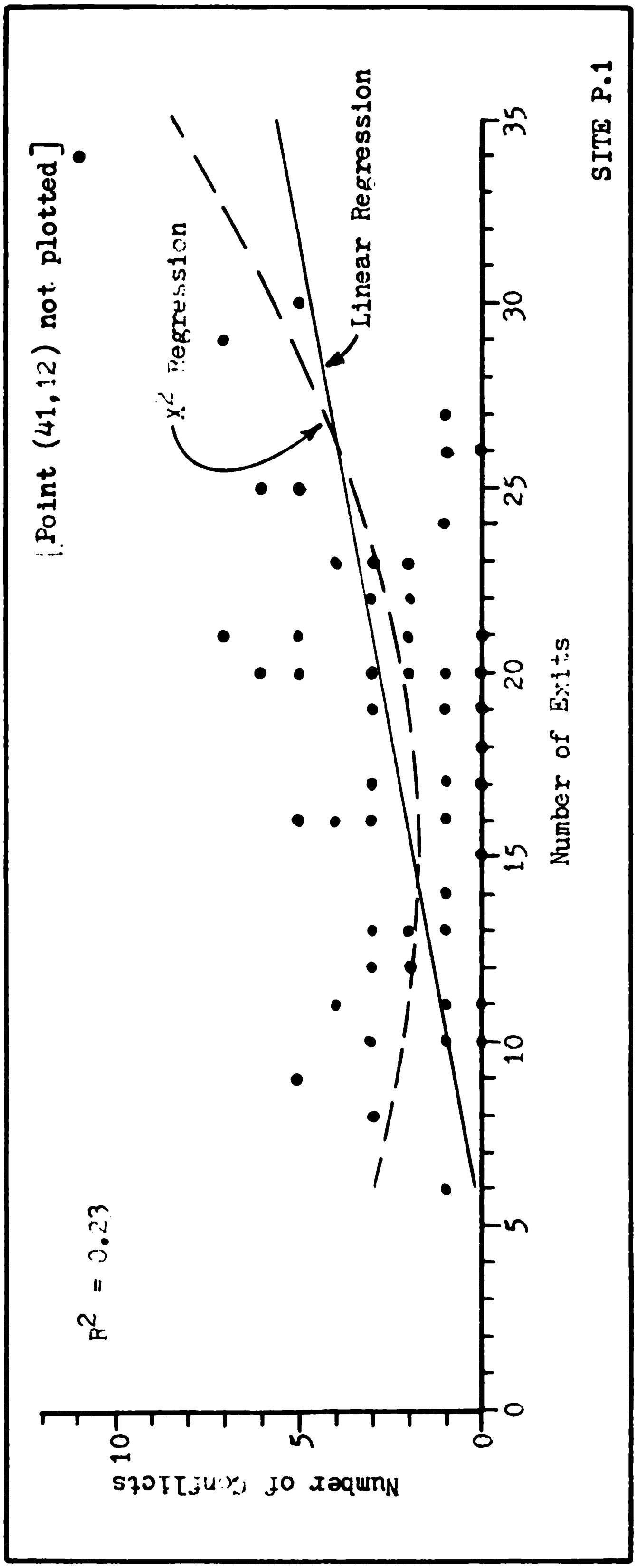


Figure 5
Data Points for Site P.1

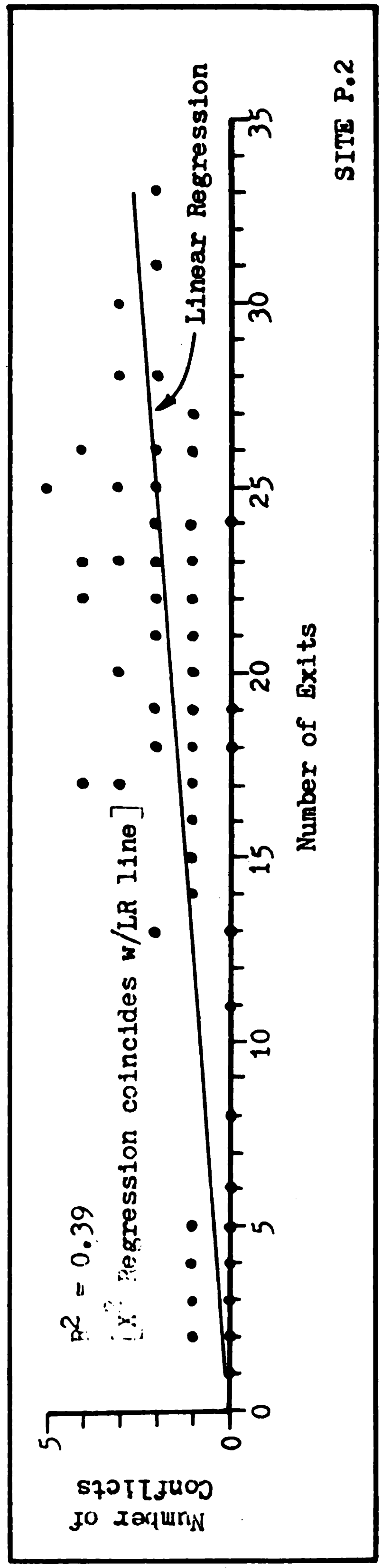


Figure 6
Data Points for Site P.2

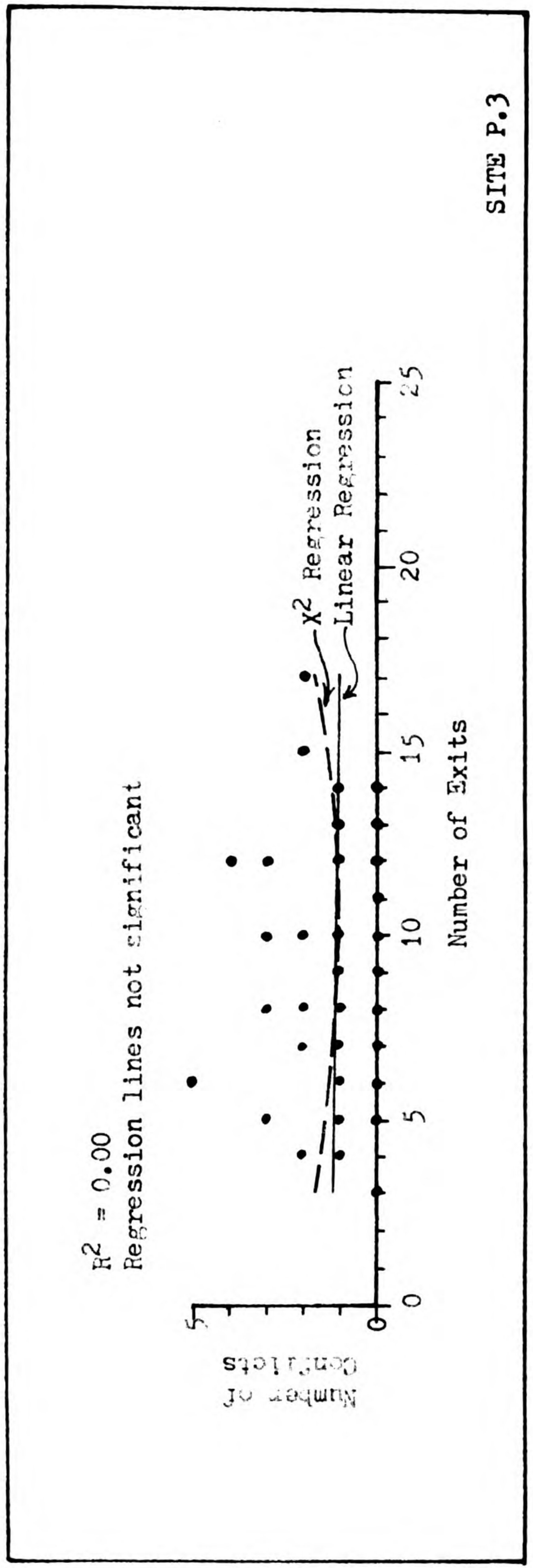


Figure 7
Data Points for Site P.3

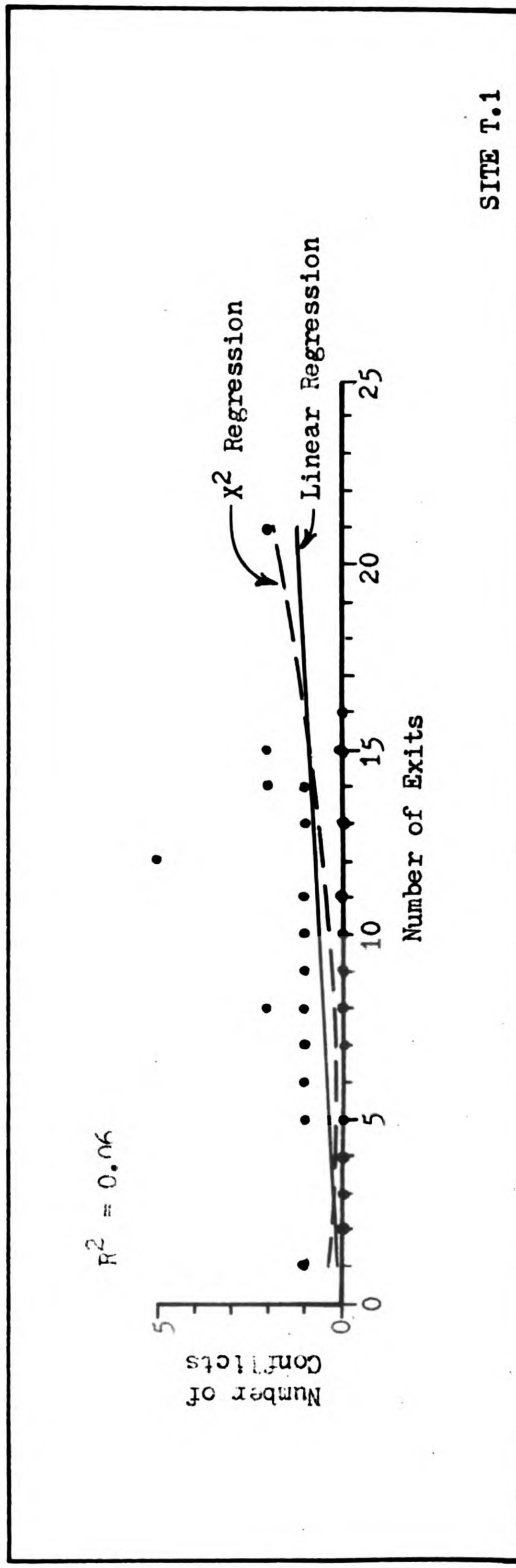


Figure 8
Data Points for Site T.1

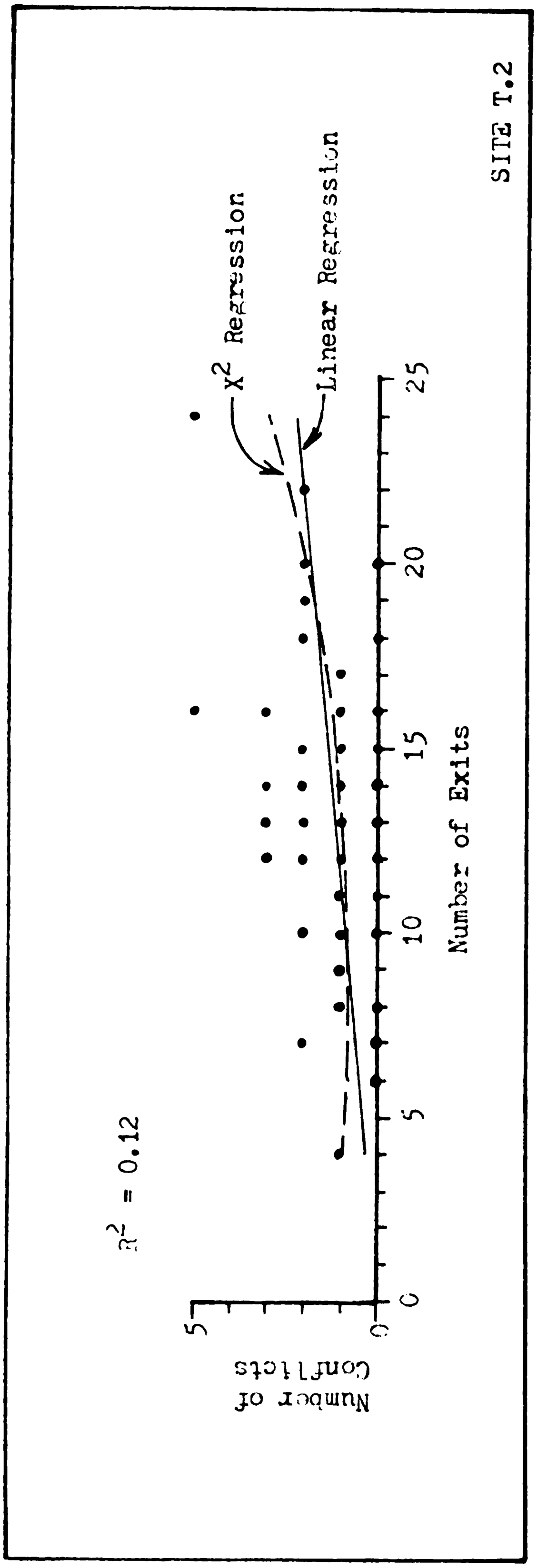


Figure 9
Data Points for Site T.2

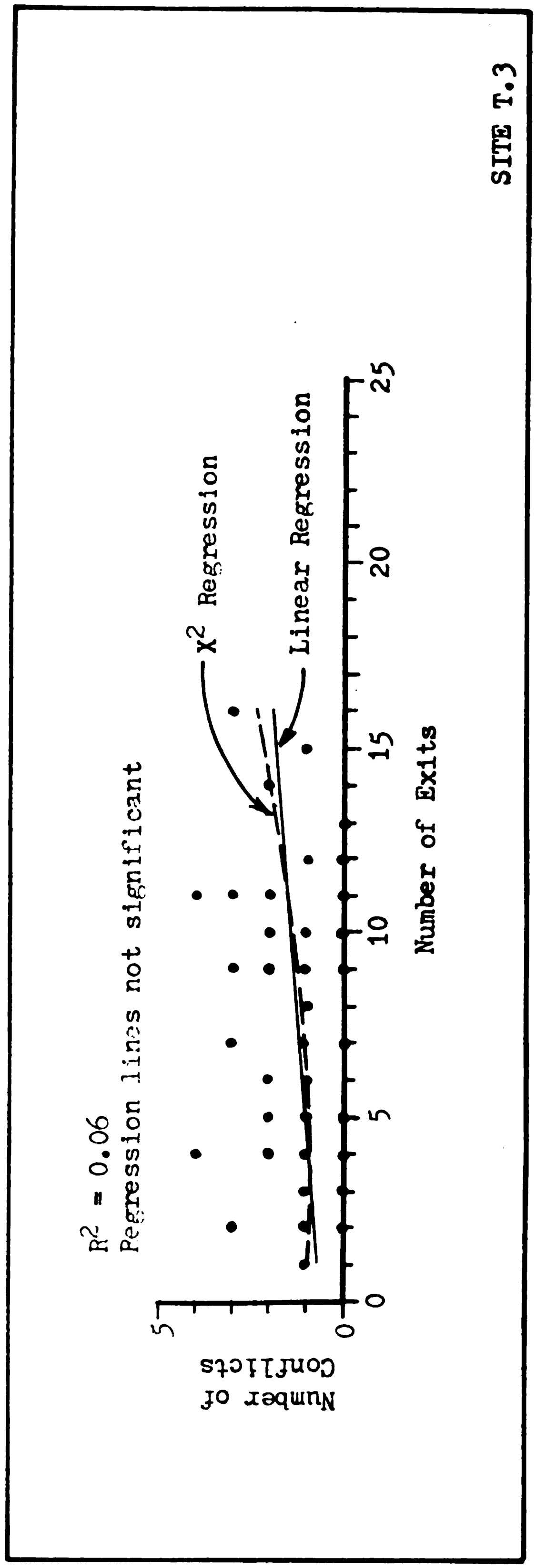


Figure 10
Data Points for Site T.3

CHAPTER 6

CONCLUSIONS

Based on the results obtained in this work, it is concluded that the Traffic Conflicts Technique is 94 percent reliable in measuring the quality of operation of urban off-ramps. The technique can therefore be used whenever a level of significance of 94 percent or less is acceptable.

The linear equation for accidents as a function of Conflicts per Minute determined from this work is a poor predictor. Thus, while the results show that (at 94 percent significance) a ramp with a conflict rate greater than that of another ramp will have a higher accident rate, the equation will give only a poor estimate of the numerical values of those accident rates.

This work found that generally the number of conflicts per minute increases as the various flow rates increase. The linear models for these relationships are poor predictors of the conflict rates, however.

This work was unable to detect any difference in operation between the three parallel design ramps and the three taper design ramps that can be attributed to the design type.

This work was also unable to detect any correlation between the frequencies of occurrence of the different types of conflicts and the types of accidents that occurred at the ramps.

CHAPTER 7

RECOMMENDED PROCEDURE

When the Traffic Conflicts Technique is used in future observations at urban off ramps, the following points should be considered:

1. If a significance level of 94 percent or less is acceptable, a significant difference in mean conflicts per minute can be regarded as evidence of a significant difference in the quality of operation.

   To establish a significant difference in means, longer testing periods should be used. The length needed depends on the mathematical difference in means, the variances of the samples, and the level of confidence desired. For this work, a typical variance found was 1.5. Using that value, the following sample sizes, in minutes, would be needed:

Difference in Means	Confidence Level 90%	94%	95%	99%
0.1	800	1060	1150	2200
0.2	200	270	300	500
0.3	90	120	130	220
0.4	50	70	70	120
0.5	30	50	50	80

   Once a significant difference is found, the investigator must determine what causative factors are involved. While such a difference might well be due to the drivers' ability to negotiate the two types of ramps, other factors must be considered, including horizontal and vertical alignments, signing, and volume/capacity ratio. A large number of sites may be needed to adequately compensate for the factors not being analyzed.

2. The data collection form developed for this work proved adequate and can be used in future observations. This form is shown in Appendix 3. If the form is reprinted, a few minor changes are suggested. The column for feet can be eliminated. The initial intent was to mark the location of the conflict, but that proved to be impractical. The three columns for Slow-Vehicle can be reduced to two, so that only one column need be checked for each Slow-Vehicle Conflict. Actually, nearly all such conflicts involved an exiting vehicle. A third column for Weaves, from Lane 2 to Lane 3, would be helpful for freeways having more than two thru lanes.

3. The use of video tape for obtaining data is recommended over the use of observers at the site. The video tape has two distinct advantages; it provides a more accurate count of the number of conflicts, and it provides the opportunity to review a sequence of events to determine exactly what conflicts occurred.

4. Additional conflict and accident data, especially for ramps with either low or high conflict rates, may produce a higher level of significance and more representative regression line. In this work, four of the six ramps had conflict rates near the mean; this resulted in the wide range in the confidence interval shown in Figure 4. More values on the extremes of the conflict rates would narrow that interval.

A P P E N D I C E S

A P P E N D I X 1

Conflict and Flow Data

SITE P.1
I 496 EB to US 127 NB
Lansing

Test Dates:			
Test P.1.1	Tuesday, September 12, 1972	3:30 - 4:00 pm
Test P.1.3	Wednesday, October 4, 1972	4:15 - 4:45 pm
Test P.1.4	Friday, August 31, 1973	4:15 - 4:45 pm

DISTRIBUTION OF CONFLICTS

Frequency Per Min	Slow Vehicle	Rear-End Conflict	Wrong-Lane Conflict	Late-Exit Conflict	Weave	Total Conflicts
0	49	50	43	80	83	16
1	23	18	36	8	5	22
2	12	12	8	2	2	11
3	4	5	2			15
4	0	3	1			8
5	2	1				10
6		1				4
7						2
8						0
9						0
10						0
11						1
12						1
Totals	69	80	62	12	9	232
Mean	0.77	0.89	0.69	0.13	0.10	2.58
Stand Dev	1.08	1.29	0.82	0.40	0.37	2.34

FLOW RATES
70 Minutes

Flow Rate in vpm	Inside Lane	Outside Lane	Exits	Total Thru	Outside Thru	Total Flow
Total	711	1694	1273	1132	421	2405
Mean	10.2	24.2	18.2	16.2	6.0	34.4
Stand Dev	6.5	7.1	6.2			12.5

Site P.1

REGRESSION EQUATIONS
all values per minute

		Stand Err	R^2
	Total Conflicts = f(Exits, Total Thru)		
(1 min)*	T Confl = 0.15 Ex + 0.076 T Th - 1.49	2.12	0.27
(2 min)	-Neither variable significant-		
(3 min)	-Neither variable significant-		
	Total Conflicts = f(Exits, Outside Thru)		
(1 min)	-Outside Thru not significant-		
(2 min)	-Outside Thru not significant-		
(3 min)	-Outside Thru not significant-		
	Total Conflicts = f(Exits)		
(1 min)	T Confl = 0.195 x Ex - 1.08	2.19	0.23
(2 min)	T Confl = 0.204 x Ex - 1.18	1.93	0.24
(3 min)	T Confl = 0.218 x Ex - 1.47	1.70	0.29
	Total Conflicts = f(Total Flow)		
(1 min)	T Confl = 0.104 x T Fl - 1.12	2.13	0.28
(2 min)	T Confl = 0.100 x T Fl - 0.87	1.86	0.29
(3 min)	T Confl = 0.103 x T Fl - 1.02	1.60	0.37
	Total Conflicts = f($Exits^2$, Exits)		
(1 min)	T Confl = 0.0162 x Ex^2 - 0.0476 x Ex + 5.13		
	Exits = f(Total Flow)		
(1 min)	Ex = 0.384 x T Fl + 4.98	3.85	0.61
(2 min)	Ex = 0.379 x T Fl + 5.21	2.69	0.74
(3 min)	Ex = 0.380 x T Fl + 5.16	2.10	0.83
	Exits = f(Outside Lane Flow		
(1 min)	Ex = 0.695 x OL + 1.36	3.70	0.64
(2 min)	Ex = 0.776 x OL - 0.59	2.28	0.81
(3 min)	Ex = 0.792 x OL - 0.97	1.59	0.90

* Time increment used for computations

SITE P.2
I 496 EB to Walnut St
Lansing

Test Dates:			
	Test P.2.1	Thursday, September 21, 1972	7:25 - 7:55 am
	Test P.2.2	Wednesday, September 26, 1972	3:15 - 3:45 pm
	Test P.2.3	Wednesday, October 9, 1973	7:25 - 7:55 am

DISTRIBUTION OF CONFLICTS

Frequency Per Min	Slow Vehicle	Rear-End Conflict	Wrong-Lane Conflict	Late-Exit Conflict	Weave	Total Conflicts
0	76	48	79	88	78	30
1	11	23	11	1	12	29
2	3	14		1		18
3		4				8
4		1				4
5						
Totals	17	67	11	3	12	110
Mean	0.19	0.74	0.12	0.03	0.13	1.22
Stand Dev	0.47	0.95	0.33	0.23	0.34	1.20

FLOW RATES
90 Minutes

Flow Rate in vpm	Inside Lane	Outside Lane	Exits	Total Thru	Outside Thru	Total Flow
Total	1538	2025	1388	2175	817	3563
Mean	17.09	22.50	15.42	24.17	9.08	39.59
Stand Dev	9.90	9.10	9.23			18.05

SITE P.2

REGRESSION EQUATIONS
all values per minute

		Stand Err	R^2
	Total Conflicts = f(Exits, Total Thru)		
(1 min)*	-Total Thru not significant-		
(2 min)	-Total Thru not significant-		
(3 min)	-Total Thru not significant-		
	Total Conflicts = f(Exits, Outside Thru)		
(1 min)	-Outside Thru not significant-		
(2 min)	T Confl = 0.0817 x Ex + 0.0754 x O Th - 1.14	1.38	0.50
(3 min)	T Confl = 0.0810 x Ex + 0.102 x O Th - 1.12	0.94	0.55
	Total Conflicts = f(Exits)		
(1 min)	T Confl = 0.0809 x Ex - 0.03	0.94	0.39
(2 min)	T Confl = 0.0714 x Ex + 0.12	0.73	0.46
(3 min)	T Confl = 0.0723 x Ex + 0.11	0.68	0.49
	Total Conflicts = f(Total Flow)		
(1 min)	T Confl = 0.0362 x T Fl - 0.21	1.01	0.30
(2 min)	T Confl = 0.0385 x T Fl - 0.30	0.71	0.49
(3 min)	T Confl = 0.0388 x T Fl - 0.31	0.64	0.55
	Total Conflicts = f($Exits^2$, Exits)		
(1 min)	T Confl = -0.0002 x Ex^2 + 0.0879 Ex - 0.05		
	Exits = f(Total Flow)		
(1 min)	Ex = 0.466 x T Fl - 3.04	3.79	0.83
(2 min)	Ex = 0.469 x T Fl - 3.14	4.21	0.80
(3 min)	Ex = 0.475 x T Fl - 3.38	3.29	0.87
	Exits = f(Outside Lane Flow)		
(1 min)	Ex = 0.959 x OL - 6.16	2.99	0.90
(2 min)	Ex = 0.986 x OL - 6.76	3.63	0.85
(3 min)	Ex = 0.990 x OL - 6.86	2.83	0.91

* Time increment used for computations

SITE T.2
US 10 NB to Meyers
Detroit

Test Dates:			
	Test T.2.1	Thursday, September 28, 1972	5:00 - 5:30 pm
	Test T.2.2	Thursday, September 20, 1973	2:00 - 2:30 pm
	Test T.2.3	Thursday, September 20, 1973	5:15 - 5:45 pm

DISTRIBUTION OF CONFLICTS

Frequency Per Min	Slow Vehicle	Rear-End Conflict	Wrong-Lane Conflict	Late-Exit Conflict	Weave	Total Conflicts
0	82	54	66	89	65	27
1	7	22	24	1	19	25
2	1	9			5	21
3		4			1	14
4		0				1
5	—	1	—	—	—	2
Totals	9	57	24	1	32	123
Mean	0.10	0.63	0.27	0.01	0.36	1.37
Stand Dev	0.34	0.97	0.44	0.10	0.64	1.22

FLOW RATES
70 Minutes

Flow Rate in vpm	Inside Lanes	Outside Lane	Exits	Total Thru	Outside Thru	Total Flow
Total	3459	1714	923	7250	791	5173
Mean	49.41	24.49	13.19	60.71	11.30	73.90
Stand Dev	18.01	6.87	3.94			24.14

SITE P.3

REGRESSION EQUATIONS
all values per minute

		Stand Err	R^2
	Total Conflicts = f(Exits, Total Thru)		
(1 min)*	-Neither variable significant-		
(2 min)	-Neither variable significant-		
	Total Conflicts = f(Exits, Outside Thru)		
(1 min)	-Neither variable significant-		
(2 min)	-Neither variable significant-		
	Total Conflicts = f(Exits)		
(1 min)	-R^2 less than minimum for significance-		0.00
(2 min)	-R^2 less than minimum for significance-		0.03
(3 min)	-R^2 less than minimum for significance-		0.05
	Total Conflicts = f(Total Flow)		
(1 min)	-R^2 less than minimum for significance-		0.02
(2 min)	-R^2 less than minimum for significance-		0.05
(3 min)	-R^2 less than minimum for significance-		0.06
	Total Conflicts = f(Exits^2, Exits)		
(1 min)	T Confl = 0.0089 x Ex^2 - 0.173 x Ex + 1.90		
	Exits = f(Total Flow)		
(1 min)	Ex = 0.173 x T Flow + 5.09	2.65	0.15
(2 min)	-R^2 less than minimum for significance-		0.08
(3 min)	Ex = 0.192 x T Flow + 4.72	1.59	0.28
	Exits = f(Outside Lane Flow)		
(1 min)	Ex = 0.340 x OL + 3.69	2.45	0.27
(2 min)	Ex = 0.214 x OL + 5.44	1.80	0.15
(3 min)	Ex = 0.289 x OL + 4.39	1.55	0.31

* Time increment used for computations

SITE T.1
I 75 NB to 7-Mile
Detroit

Test Dates:			
	Test T.1.2	Thursday, September 21, 1972	5:00 - 5:15 pm
	Test T.1.3	Wednesday, July 25, 1973	11:30 - 12:00 noon
	Test T.1.4	Tuesday, October 9, 1973	5:45 - 6:15 pm

DISTRIBUTION OF CONFLICTS

Frequency Per Min	Slow Vehicle	Rear-End Conflict	Wrong-Lane Conflict	Late-Exit Conflict	Weave	Total Conflicts
0	83	69	85	90	80	57
1	7	17	5		10	22
2		3				9
3		1				1
4						0
5	—	—	—	—	—	1
Totals	7	26	5	0	10	48
Mean	0.08	0.29	0.06		0.11	0.53
Stand Dev	0.27	0.59	0.23		0.32	0.86

FLOW RATES
70 Minutes

Flow Rate in vpm	Inside Lanes	Outside Lane	Exits	Total Thru	Outside Thru	Total Flow
Total	4615	1348	609	5371	739	5980
Mean	65.93	19.26	8.70	76.73	10.56	85.43
Stand Dev	24.15	6.01	3.58			29.38

SITE T.1

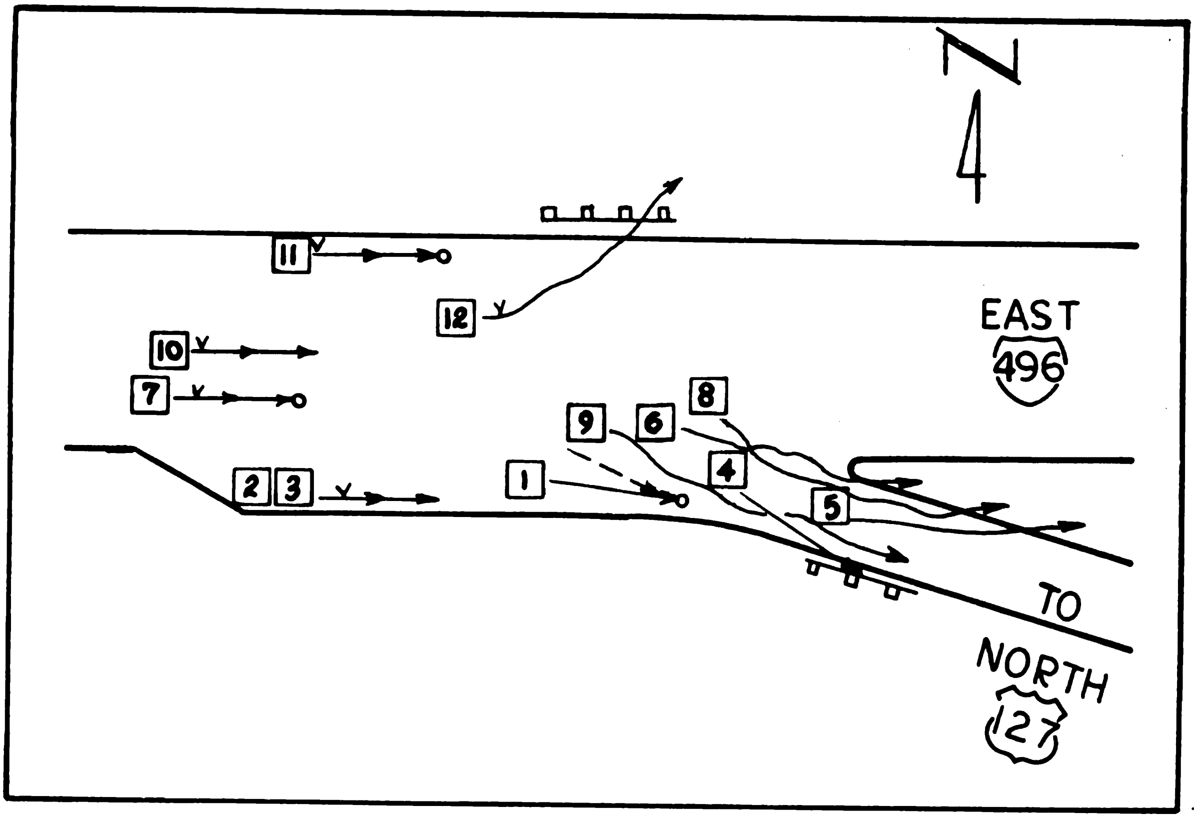
REGRESSION EQUATIONS
all values per minute

		Stand Err	R^2
	Total Conflicts = f(Exits, Total Thru)		
(1 min)*	-Exits not significant-		
(2 min)	-Exits not significant-		
(3 min)	-Neither significant-		
	Total Conflicts = f(Exits, Outside Thru)		
(1 min)	T Confl = 0.0536 x Ex + 0.0671 x 0 Th - 0.70	0.75	0.18
(2 min)	-Exits not significant-		
(3 min)	-Outside Thru not significant-		
	Total Conflicts = f(Exits)		
(1 min)	T Confl = 0.0585 x Ex - 0.04	0.81	0.06
(2 min)	-R^2 less than minimum for significance-		0.08
(3 min)	T Confl = 0.148 x Ex - 0.82	0.52	0.34
	Total Conflicts = f(Total Flow)		
(1 min)	T Confl = 0.0106 x T Fl - 0.44	0.77	0.14
(2 min)	T Confl = 0.0112 x T Fl - 0.48	0.56	0.26
(3 min)	T Confl = 0.0117 x T Fl - 0.53	0.54	0.28
	Total Conflicts = f($Exits^2$, Exits)		
(1 min)	T Confl = $0.0052 \times Ex^2 - 0.0415 \times Ex + 0.38$		
	Exits = f(Total Flow)		
(1 min)	Ex = 0.0584 x T Flow + 3.71	3.17	0.23
(2 min)	Ex = 0.0576 x T Flow + 3.78	2.33	0.34
(3 min)	Ex = 0.0568 x T Flow + 3.85	1.86	0.43
	Exits = f(Outside Lane Flow)		
(1 min)	Ex = 0.382 x OL + 1.35	2.78	0.41
(2 min)	Ex = 0.354 x OL + 1.88	2.11	0.46
(3 min)	Ex = 0.330 x OL + 2.33	1.71	0.52

* Time increment used for computations

ACCIDENT DATA
SITE P.1

COLLISION DIAGRAM

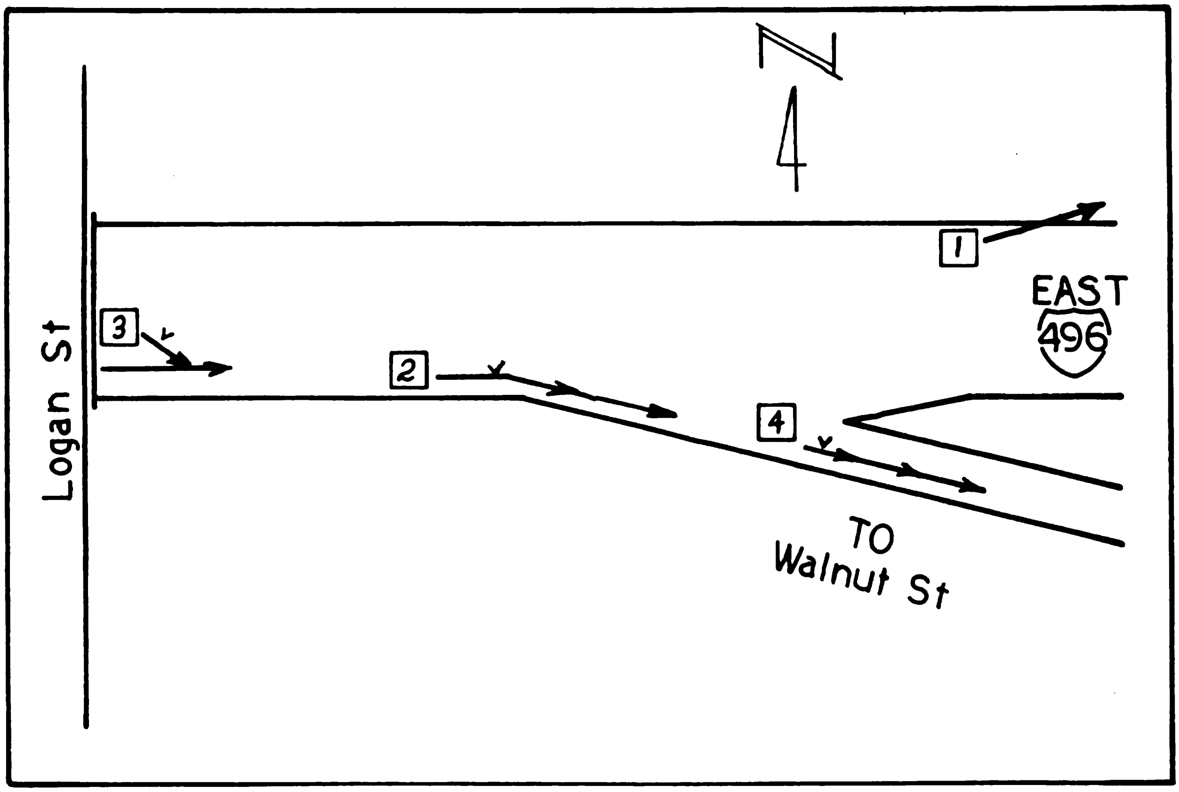


DETAILS OF ACCIDENTS

Acc No.	Date	Time	Severity F	PI	PD	Weather	Light Cond.	Pavt	Stated Reason for Accident
	1971								
1	9-18	0235		X		Clear	Dark	Dry	Avoid Vehicle
2	9-27	0730			X	Rain	Light	Wet	Too Close
3	10-22	1710			X	Rain	Dusk	Wet	Failure to Stop
	1972								
4	4-19	1405			X	Clear	Light	Dry	Avoid Vehicle
5	5-13	1820			X	Rain	Light	Wet	Speed
6	6-14	0900			X	Cloudy	Light	Dry	Avoid Vehicle
7	6-29	1640		X		Clear	Light	Dry	Failure to Stop
8	7-22	2153			X	Clear	Dark	Dry	Avoid Vehicle
9	9-18	0730			X	Rain	Light	Wet	Speed
10	10- 7	1150			X	Clear	Light	Dry	Too Close
11	10- 7	1150		X		Clear	Light	Dry	Failure to Stop
12	10-30	1500			X	Clear	Light	Dry	Speed

ACCIDENT DATA
SITE P.2

COLLISION DIAGRAM

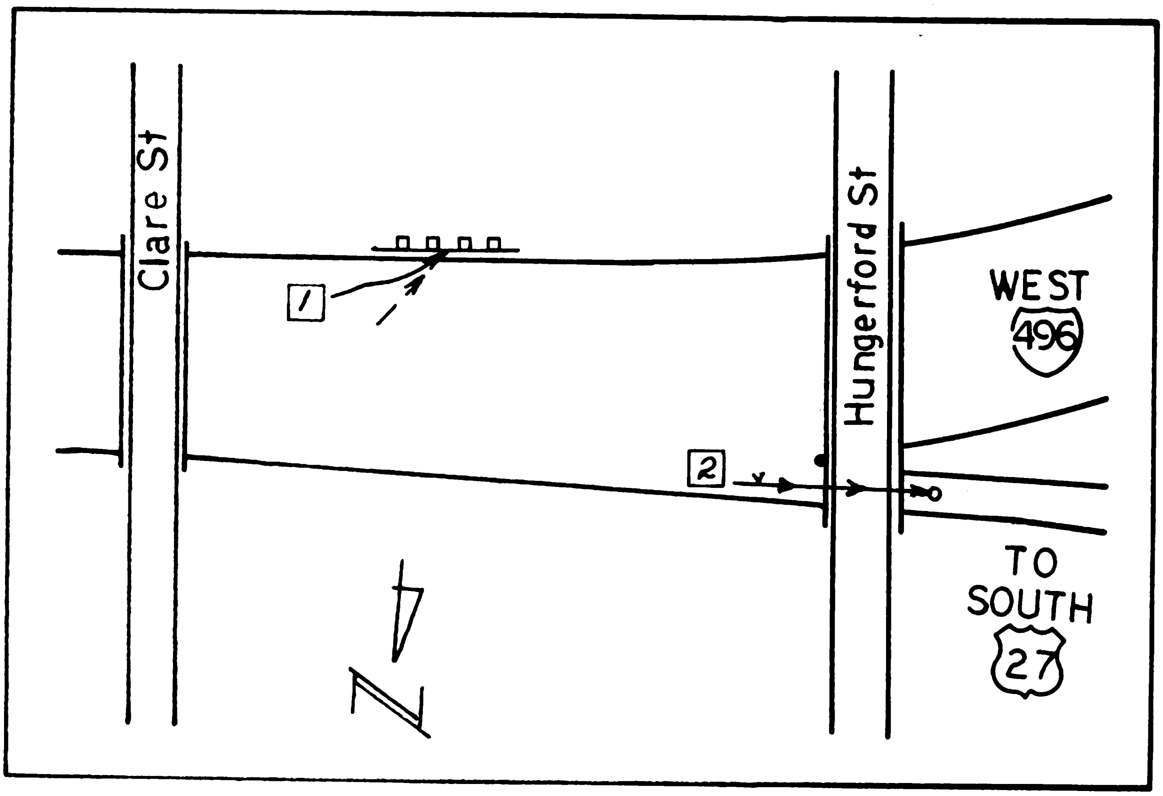


DETAILS OF ACCIDENTS

Acc No.	Date	Time	Severity F	PI	PD	Weather	Light Cond.	Pavt	Stated Reason for Accident
	1972								
1	2-6	1005			X	Snow	Light	Snow	Speed
2	2-8	0802			X	Clear	Light	Dry	Failure to Stop
3	8-25	0040			X	Clear	Dark	Dry	Improper Lane Usage
4	11-30	0813			X	Clear	Light	Dry	Speed

ACCIDENT DATA
SITE P.3

COLLISION DIAGRAM

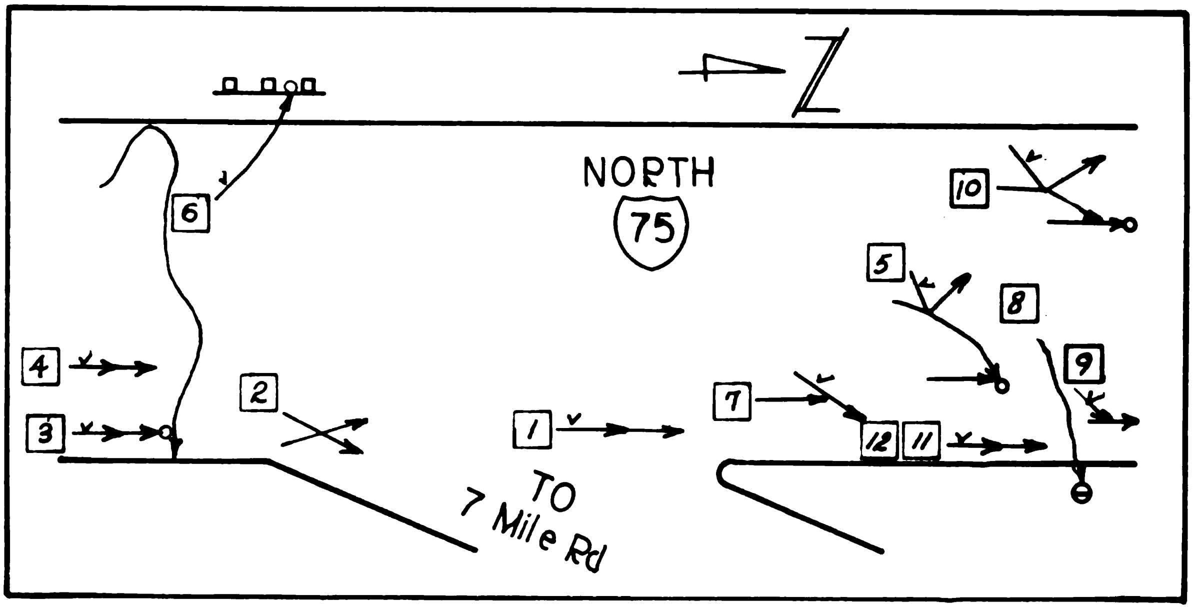


DETAILS OF ACCIDENTS

Acc No.	Date	Time	Severity F	Severity PI	Severity PD	Weather	Light Cond.	Pavt	Stated Reason for Accident
	1971								
1	3-2	2340			X	Cloudy	Dark	Dry	Avoid Vehicle Drinking
	1972								
2	8-22	1700		X		Rain	Light	Wet	Failure to Stop

ACCIDENT DATA
SITE T.1

COLLISION DIAGRAM

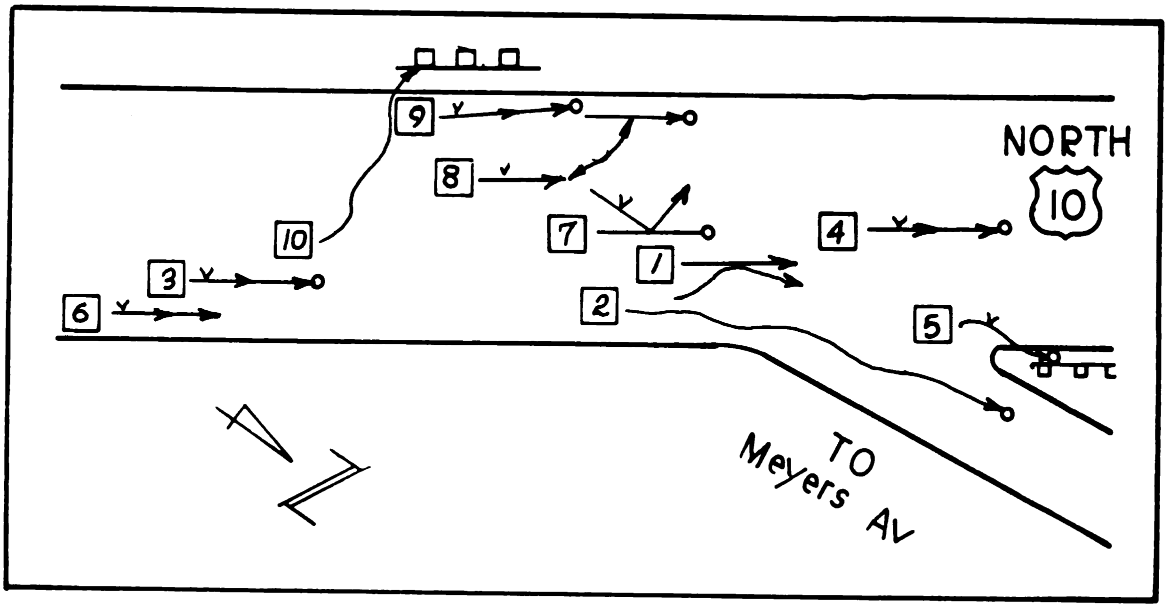


DETAILS OF ACCIDENTS

Acc No.	Date	Time	Severity F	PI	PD	Weather	Light Cond.	Pavt	Stated Reason for Accident
	1971								
1	1-14	0337			X	Clear	Dark	Icy	Failure to Stop
2	4-30	1750			X	Clear	Dusk	Dry	Improper Lane Usage
3	9-20	0630		X		Rain	Light	Wet	Lost Control, Too Close
4	10-26	0012			X	Clear	Dawn	Dry	Failure to Stop
	1972								
5	1-11	1045		X		Clear	Light	Dry	Improper Lane Usage
6	2-6	0400		X		Snow	Dark	Snow	Lost Control
7	10-9	0830			X	Clear	Light	Dry	Improper Lane Usage
8	12-6	0930			X	Cloudy	Light	Wet	Lost Control
9	12-23	0225			X	Cloudy	Dark	Dry	Failure to Stop
10	12-23	0230		X		Cloudy	Dark	Dry	Improper Lane Usage
11	12-23	0230			X	Cloudy	Dark	Dry	Too Close
12	12-23	0230			X	Cloudy	Dark	Dry	Failure to Stop

ACCIDENT DATA
SITE T.2

COLLISION DIAGRAM

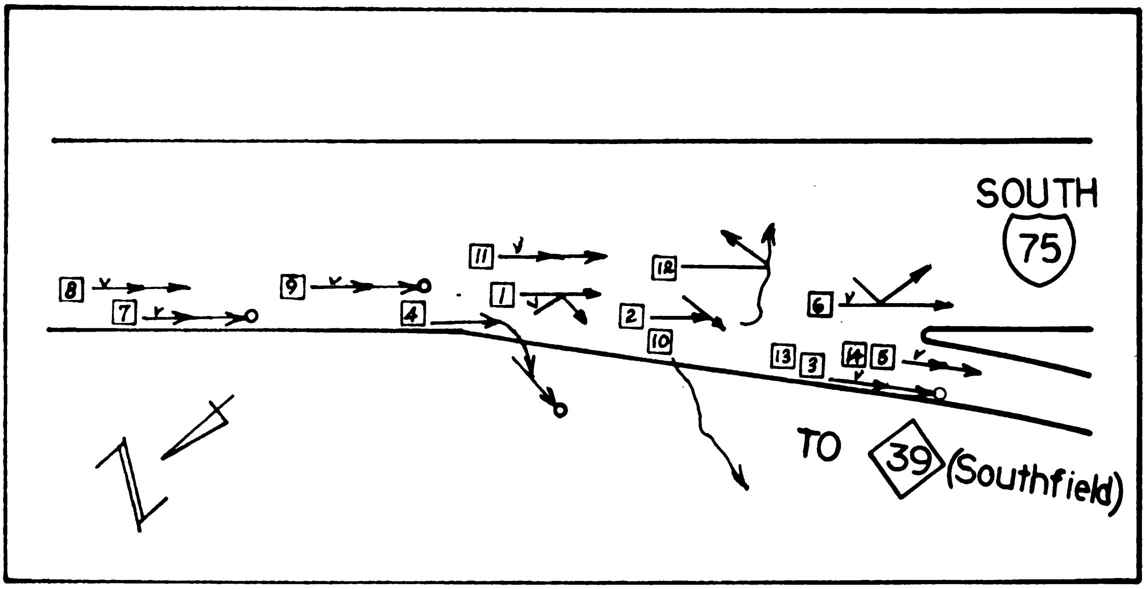


DETAILS OF ACCIDENTS

Acc No.	Date	Time	Severity F	PI	PD	Weather	Light Cond.	Pavt	Stated Reason for Accident
	1971								
1	3-10	1230			X	Rain	Light	Wet	Lost Control
2	9-8	0700		X		Clear	Light	Dry	Lost Control
3	10-12	2045		X		Clear	Dark	Dry	Failure to Stop
4	12-6			X		Rain	Dusk	Wet	Too Close
	1972								
5	6-16	0115		X		Clear	Dark	Dry	Lost Control
6	7-3	0010			X	Rain	Dark	Wet	Failure to Stop
7	10-29	1105		X		Rain	Dark	Wet	Improper Lane Usage
8	10-29	0115		X		Rain	Dark	Wet	Failure to Stop
9	10-29	0120		X		Rain	Dark	Wet	Failure to Stop
10	11-4	2200			X	Cloudy	Dark	Dry	Avoid Vehicle

ACCIDENT DATA
SITE T.3

COLLISION DIAGRAM



DETAILS OF ACCIDENTS

Acc No.	Date	Time	Severity F	PI	PD	Weather	Light Cond.	Pavt	Stated Reason for Accident
	1971								
1	6-7	1530			X	Cloudy	Light	Dry	Improper Lane Usage
2	6-11	2255			X	Clear	Dark	Dry	Improper Lane Usage
3	2-11	1545		X		Clear	Light	Dry	Failure to Stop
4	12-7	1800		X		Clear	Dark	Dry	Speed
	1972								
5	1-10	1300			X	Cloudy	Light	Dry	Improper Backing
6	1-13	1425			X	Snow	Light	Snow	Improper Lane Usage
7	4-25	1535		X		Clear	Light	Dry	Speed
8	4-27	1730			X	Clear	Light	Dry	Speed
9	9-13	1730		X		Cloudy	Light	Dry	Too Close
10	9-20	0905			X	Clear	Dark	Dry	Lost Control
11	10-24	1745			X	Clear	Dusk	Dry	Failure to Stop
12	11-20	1825		X		Cloudy	Dark	Dry.	Avoid Vehicle
13	12-1	1930		X		Clear	Light	Dry	Speed
14	12-26	0130			X	Snow	Dark	Snow	Failure to Stop

ACCIDENT RATES

Accidents per 100-Million Vehicles

$$\text{Accident Rate} = \frac{(\text{Acc in 2 years}) \ \text{x} \ (100\ 000\ 000)}{2 \text{ yr x } 365 \text{ day/yr x } (\frac{1}{2} \text{ Average Daily Traffic})}$$

$$= \frac{(\text{Acc in 2 years})}{(\text{Average Daily Traffic})} \text{ x } 273\ 973.$$

Ramp	1971 & 1972 Accidents	1971 ADT	Acc/100 MV
P.1	12	34 000	96.70
P.2	4	32 500	33.72
P.3	2	36 000	15.22
T.1	12	98 500	33.38
T.2	10	108 000	25.37
T.3	14	63 000	60.88

TEST FOR LEVEL OF SIGNIFICANCE

Accident Rate as a Function of Total Conflicts per Minute

Regression Equation:

$$Acc/100\ MV = 36.0435 \times (T\ Confl) - 5.5680$$

Null Hypothesis: H_o: $b = b_o$

$$t = \frac{(b - b_o)}{Se} \times \sqrt{\frac{Sxx}{n}}$$

$b = 36.0435$
$b_o = 0$
$Se = 19.8750$
$Sxx = 13.2809$
$n = 6$

$t = 2.698$

The confidence level for t = 2.7 w/4 DF is approximately 94.5%.

A P P E N D I X 3

Data Collection Form

STATE OF MICHIGAN
DEPARTMENT OF STATE HIGHWAYS
TRAFFIC & SAFETY DIVISION

URBAN RAMP STUDY

CONFLICTS

Location ______________________

Date __________ Time __________

Observer __________ Live ___ TV ___

Time	ft	Slow Veh Exit Yes No	Rear End Confl	Wrong-Lane Exit La 4	La 3	La 2	Conflict Cong Weave	Late Exit Confl	Weave La 1 Ramp La 2 La 1	Weave Confl	Drift	

R E F E R E N C E S

REFERENCES

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