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DEVELOPMENT AND USE OF A
COMPUTER SIMULATION MODEL FOR
THE EVALUATION OF DESIGN AND
CONTROL ALTERNATIVES FOR
INTERSECTIONS OF MINOR ROADS
WITH MULTI-LANE RURAL HIGHWAYS:
FIELD STUDIES AND MODEL VALIDATION

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PURDUE UNIVERSITY
INDIANA STATE HIGHWAY COMMISSION

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Interim Report

DEVELOPMENT AND USE OF A COMPUTER SIMULATION MODEL FOR THE EVALUATION OF DESIGN AND CONTROL ALTERNATIVES FOR INTERSECTIONS OF MINOR ROADS WITH MULTI-LANE RURAL HIGHWAYS: FIELD STUDIES AND MODEL VALIDATION

TO: H. L. Michael, Director
Joint Highway Research Project

May 15, 1979

Project: C-36-59X

FROM: K. C. Sinha, Research Engineer
Joint Highway Research Project

File: 8-5-24

Attached is an Interim Report on the Indiana HPR Part II study entitled "Evaluation of Design and Control Alternatives to Improve Safety of Intersections at Multi-Lane Rural Highways." The title of the present report is "Development and Use of a Computer Simulation Model for the Evaluation of Design and Control Alternatives for Intersections of Minor Roads with Multi-Lane Rural Highways: Field Studies and Model Validation." It has been authored by Ahmed Essam Radwan, Kumares C. Sinha and Harold L. Michael.

This part of the simulation phase (portion of Task 10 of approved proposal) presents the details of the field studies conducted at a group of intersections in Indiana in order to obtain data on relevant traffic characteristics for use in the simulation models. The report also includes the results of the validation of a modified version of the computer simulation model.

The report is forwarded for review, comment and acceptance as partial fulfillment of the objectives of the research.

Respectfully submitted,

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Interim Report

DEVELOPMENT AND USE OF A COMPUTER SIMULATION MODEL
FOR THE EVALUATION OF DESIGN AND CONTROL ALTERNATIVES
FOR INTERSECTIONS OF MINOR ROADS WITH MULTI-LANE
RURAL HIGHWAYS: FIELD STUDIES AND MODEL VALIDATION

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The contents of this report reflect the views of the author who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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16. Abstract <p>This report presents the details of the field studies conducted at a group of intersections in Indiana to obtain the necessary data for the modification and subsequent validation of the computer simulation model. Appropriate field studies were conducted to estimate gap acceptance distribution of minor road drivers to cross or merge with major road traffic at stop controlled intersections. Additional field studies were performed at signalized and unsignalized intersections to investigate the correlation between traffic volume, traffic conflicts and accidents, and to determine the effectiveness of the traffic conflict technique as a measure of safety at intersections.</p> <p>The simulation package, developed under the auspices of the Federal Highway Administration, called UTCS-1 (known now as NETSIM) has been selected for use in this study. Several modification, however, were made to the computer model to incorporate the estimation of average delay per vehicle, fuel consumption per vehicle, and number of traffic conflicts. The results of the field studies were further used to substantiate the model's validity.</p>			
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DRIVER CHARACTERISTICS FIELD STUDIES STUDY PROCEDURE

Site Selection

The selection of suitable study sites involved the consideration of several factors. The following considerations were made regarding the selection of study locations in order to control several roadway and traffic variables which could influence the study results.

1. The intersection should be located in rural areas or urban-sururban areas with rural design characteristics.
2. To obtain a random sample of gaps in the main traffic stream, the intersections were located at least 0.25 mile from any traffic-control device on both major and minor roads.
3. The ADT values on the major and minor roads were in excess of 4,000 and 2,000 vpd, respectively.
4. The intersections studied were very similar with regard to their geometry, consisting of four-lanes on the major divided by a median of at least 30 feet in width, and two-lane minor road. Sight distance conditions were about equal on all approaches, and the major road was approximately the same at all intersections.
5. All intersections were controlled by stop sign or flasher on the minor road.
6. Posted speed limits on the major roads were 55 pmh, except for one intersection which was 45 mph, and minor roads speed

limits were 40 mph except one intersection which had a 35 mph speed limit.

The limitation narrowed the number of intersections down to six. A brief description of each intersection is given in Table 1.

Use and Placement of Equipment

Several means of recording gaps and lags were considered. It was decided to use one of the following methods and to determine which would be the most suitable for the purpose of this study:

1. Multi-pen recorder.
2. Time-lapse photography.

Preliminary study was conducted at an intersection in Lafayette, Indiana by using a multi-pen recorder. This unit was used in conjunction with pneumatic road tubes. A series of adjacent courses of pneumatic tubes were used near the intersection to obtain the time spacing between vehicles. A special survey device consisting of a group of buttons - microswitches electrically connected to the recorder - enabled the observer to measure up to 20 variables. The switches were manually actuated to indicate:

1. Arrival of side street vehicles classified by direction and vehicle type.
2. Complete entry into the intersection of side street vehicles classified by direction and turning maneuver.
3. Left turning vehicles from major highway to minor road.

Several difficulties were encountered during the preliminary study and it was decided to disregard the multi-pen recorder as a tool for this study. The reasons for this decision were:

1. The high speed of traffic on the major road did not allow the pneumatic tubes to stay stretched across the lanes of traffic.

Table 1. Summary of Gap Acceptance Study Locations in Indiana

<u>Inter- section</u>	<u>Minor Road</u>	<u>Major Road</u>	<u>County</u>	<u>Grade At Intersection</u>	<u>Sight Conditions</u>	<u>Major Speed Limit</u>	<u>Minor Speed Limit</u>	<u>Major ADT**</u>	<u>Minor ADT</u>
A	SR-14	US-31	Fulton	Level	Adequate	45 mph	40 mph	4,650 Vpd	2,675 Vpd
B	SR-18	US-31	Miami	Level	Adequate	55 mph	40 mph	15,400 Vpd	2,000 Vpd
C	SR-38	US-31	Hamilton	Mostly Level	Adequate	55 mph	40 mph	12,200 Vpd	2,440 Vpd
D	Burr St.	US-30	Lake	Level	Adequate	55 mph	40 mph	19,600 Vpd	3,000 Vpd
E	*SR-51	US-30	Lake	Level	Adequate	55 mph	40 mph	21,150 Vpd	3,175 Vpd
F	9th St. Road	US-52 Bypass in Lafayette	Tippe- canoe	Level	Adequate	55 mph	35 mph	16,400 Vpd	2,440 Vpd

*SR-51 and US-30 Intersection was operating under flashing operation during the study period.

**The ADT Figures are extracted from the 1975 A.D.T. map.

2. A team of observers is required to secure the necessary data.
3. The interpretation of the recorder chart is not simple. In addition, the accuracy of the time spacing between vehicles is measured to the nearest one second only.
4. Human errors anticipated due to the manual actuation of the electronical switches.

Time-motion picture was then chosen for this study. The camera used was a 16mm type specially equipped with a zooming and wide-angle lens. A 12-volt battery, working through an inverter, powered the electric motorized drive unit which operated the camera. The drive unit connected to the camera mounted on a tripod was geared for a speed of 4 frames per second. Therefore the elapsed time intervals were measured to the nearest 0.25 second.

Data Collection

Data collection was performed with the same procedure at all study sites. At each intersection, the camera tripod was mounted on a plywood base specially attached to the roof of a car. The car was parked at the minor approach with a distance of 100 to 150 feet away from the intersection corner. The zooming lens enabled the observer to overcome the problem created by parking away from the intersection. The parked car was located at a spot in which it was relatively as inconspicuous to the passing traffic as possible. Two typical field installations are shown in Figure 1.

Data were collected randomly throughout the seven days of the week during peak periods as well as off-peak periods. Field studies were performed only when the weather was clear and the pavement surface was dry. The speed of the camera was frequently calibrated. Passenger cars as well as trucks were considered on both minor and major approaches.

FIGURE 1 TWO TYPICAL FIELD SETUP

The camera was started whenever a vehicle approached the intersection from the minor road and stopped for the stop sign. After the minor road driver had accepted a time gap, the camera was stopped unless there was another vehicle at the opposite approach, or a queue of more than one vehicle was formed at the minor approach. The maximum gap size accepted by the minor road driver was chosen to be 15 seconds, and the camera was stopped if the observer felt that the time gap between vehicles on the major road was longer than this limit. A total of 25 films, 100 feet each, were exposed at the six intersections.

The developed films were viewed by a time-motion - study projector. The projector had the ability to advance the film one frame at a time. The observer started counting frames as the minor road vehicle stopped at the stop sign, and stopped counting as the major road vehicle arrived at the intersection. The difference between the two counts, divided by 4, gave the time-lag in seconds. A time-gap interval was recorded as the difference in frame numbers, between two successive major road vehicles passing through the intersection divided by the speed of the camera.

Data Analysis

The first step in the statistical analysis was to determine the minimum sample size required. A minimum value was established at early stages of data collection. Two types of errors were covered. Type I error or α error is defined as the hypothesis as stated is true but the analysis indicates it is false. Type II error or β error is the hypothesis as stated is false but the analysis indicates it is true.

A maximum value of 0.05 was assumed for both errors α and β . The maximum acceptable standard deviation, S , was assumed to be 1.0 second.

It was also decided that differences in headways of .75 seconds should be detected. According to the minimum sample size tables (1), results indicated that a combined sample size of at least 40 was required in order to have the errors within the desired range.

Gap Acceptance Distributions

The acceptance and rejection values for the gaps and lags were tabulated and classified according to the following maneuvers:

1. Right turn maneuver from major road to minor road.
2. Through movement crossing the major road without stopping at the median. It was called crossing in one maneuver.
3. Through movement crossing the major road with stopping at the median. It was called crossing in two maneuvers because the driver crossed two lanes at a time.
4. Left turn from major road to minor road. It was called left turn in one maneuver because the driver performed the crossing of two major lanes and merged with the third lane in one movement.
5. Left turn from median to major road. It was called left turn in two maneuvers. The crossing of the two lanes of the major road to get to the median was classified under through movement in two maneuvers.

The classification of a maneuver to the above categories followed some logical considerations. In the case of crossing the major road, the driver may try to cross the four lanes in one maneuver. The lags available on both near and far sides have to be checked. If the traffic conditions at the moment enable the driver to perform this maneuver, he takes the necessary actions to do so. On the other hand, if the lags do not allow him to do so, he performs the maneuver in two steps.

As for the left turning maneuver, it follows the same logic mentioned above. The gap acceptance and rejection values for truck drivers were tabulated separately from passenger cars.

The second step in the statistical analysis was to combine gaps and lags data from different intersections. The Kruskal-Wallis one-way analysis of variance (2) was chosen as the most suitable nonparametric test for this study. This test is another alternative to the parametric F-test since the F-test is sensitive to deviations from normality.

Kruskall-Wallis Nonparametric Test

To perform the Kruskal-Wallis Test, all cases from the n intersections are ranked in a single series. If the n populations are the same, whether they are normal or nonnormal, no group should be very much different from another in terms of the ranks within each group. The rank sum is computed for each group. From these, the Kruskal-Wallis H statistic is computed, which has a chi-square distribution. The null hypothesis is:

H_0 : The medians of the K populations are equal.

The alternative hypothesis is:

H_1 : At least one of the populations has a median value different from the others.

It was noticed from the gap and lag acceptance values that the figures reported at the intersection of SR 14 and US 31 were way above the average. The high values of gap acceptance may be explained by the very low ADT at that particular intersection. In addition, the amount of data at this intersection was found to be insufficient, hence, it was decided to disregard this intersection in the Kruskal-Wallis test. Also, it was assumed that there is no difference between the observed values of lags and gaps and they could be combined.

The Kruskal-Wallis test was performed on the five intersections as one group and for each maneuver separately. It was found that the medians of the five intersections, for four maneuvers out of the five, were not equal at level of significance $\alpha = .05$ and therefore it was thought undesirable to combine the five intersections. This result was as expected because of the variation in traffic volume from one intersection to the other. As the traffic increases on the major road, the gap sizes decrease and the drivers of the minor roads tend to accept smaller gaps. Also the increase of traffic volume on the minor road creates some sort of pressure of traffic demand which force drivers to accept smaller gaps.

One good measure of pressure of traffic demand is gap rejection. This phenomenon was realized earlier by several investigators. It was decided to use the number of gap rejections as the best measure to eliminate the variations between intersections. For each lag or gap acceptance, the number of lag or gap rejections were counted. For each maneuver at each intersection the total number of gap and lag rejections were calculated. Each lag and gap acceptance value was then adjusted by a factor calculated from Equation (1):

Gap acceptance value for a driver (seconds) = (Gap acceptance value)X

$$\left(1.00 + \frac{\text{number of gaps rejected by a driver}}{\text{total number of gaps rejected by all drivers}}\right) \quad (1)$$

The Kruskal-Wallis test was performed again on the transformed data and it was found that the medians of the five intersections were equal at level of significance $\alpha = .10$. The results of the test are

shown in Table 2. Before accepting the null hypothesis, one must consider the probability of committing type II error (β value). The true value of β is impossible to calculate unless a specific alternative hypothesis is specified. Closer look at the results of the Kruskal-Wallis test results, shown in Table 2 reveals that the significance level α acceptance for the last two maneuvers are relatively high. So there is no reason to believe that the data of the two left-turn maneuvers are significantly different for the five intersections. Therefore, their data can be combined. As for the other maneuvers, it was desired to detect differences in the true population by a value of 2.0 seconds. Knowing the α level, the sample size and the difference in the true population, β values were calculated and their values were found to be less than .10 for the three cases. Hence, the data of the five intersections for the first three maneuvers was also combined. The Kruskal-Wallis test was then performed on all the maneuvers and it was found that there is a significant difference between the five maneuvers at level of significance $\alpha = .05$. Hence, there was no reason to believe that the data of the five maneuvers can be combined. The results of this test are shown in Table 3.

The same procedure was performed on the gap and lag acceptance values involving truck drivers. The number of data points observed for truck drivers for each maneuver was very small. Therefore, the data points of through one maneuver were combined with the data points of through two maneuvers and the same process was repeated for the left turn maneuvers. The Kruskal-Wallis test was then run for the three maneuvers separately for the transformed gap acceptance data. From the

Table 2. Kruskal-Wallis Test Results of Passenger Cars for Each Maneuver Separately

Type of Maneuver	Original Gap Acceptance Data			Transformed Data		
	No. of Cases	Chi-Square	Significance	No. of Cases	Chi-Square	Significance
Right Turn	220	17.9516	.0013	220	5.7971	.2148
Through in One Maneuver	73	9.9067	.0194	73	5.8640	.1184
Through in Two Maneuvers	312	12.6981	.0053	312	4.7879	.1880
Left Turn in One Maneuver	167	.9339	.6269	167	3.7514	.4407
Left Turn in Two Maneuvers	154	19.2513	.0007	154	2.5822	.6300

Table 3. Kruskal-Wallis Test Results of Passenger Cars for All Maneuvers Combined

<u>No. of Cases</u>	<u>Chi-Square</u>	<u>Significance Level</u>
926	57.1713	.00001

test results, it was found that there was no significance difference between the five intersections for level of significance $\alpha = .05$. The results of this test are shown in Table 4. The Kruskal-Wallis test was performed again on all the maneuvers combined. The test showed that there was no significance difference between the three maneuvers for level of significance $\alpha = .05$. The results are shown in Table 5.

In order to differentiate the method discussed in this section from other methods of analysis discussed later, this particular method will be referred to as the "Adjusted Gap Acceptance" method.

Distributions of the "Adjusted Gap Acceptance" Method

Frequency tables were calculated for the five maneuvers of passenger cars as well as the three maneuvers of trucks. To develop the real gap acceptance distributions, it was necessary to divide those values by some factor to bring it back to the original figures which is shown in Equation 2.

$$F_j = \left(1 + \frac{\sum_i \left(\frac{\text{total number of gaps accepted by all drivers}}{\text{total number of gaps rejected by all drivers}} \right)}{n_i} \right) \quad (2)$$

where: F_j = Factor for the j th maneuver.

n_i = Number of intersections for each maneuver.

The cumulative acceptance percentages were calculated for the adjusted gap acceptance values and it was plotted versus the gap acceptance size. The distribution of the all maneuvers are shown in Figures 2, 3, 4, 5, 6, and 7, assuming the 50th percentile of the cumulative gap acceptance distribution represents the average gap acceptance value for all drivers. The average gap acceptance values of the five maneuvers for the Adjusted Gap Acceptance method are shown in Table 6.

Table 4. Kruskal-Wallis Results of Trucks for each Maneuver Separately

<u>Maneuver Type</u>	<u>No. of Cases</u>	<u>Chi-Square</u>	<u>Significance Level</u>
Right Turn	34	.0360	.8494
Through	75	.3051	.8585
Left Turn	43	1.2633	.2610

Table 5. Kruskal-Wallis Results of Trucks for All Maneuvers Combined

<u>No. of Cases</u>	<u>Chi-Square</u>	<u>Significance Level</u>
152	.6766	.7130

FIGURE 2 GAP ACCEPTANCE DISTRIBUTION OF PASSENGER CARS
FOR RIGHT TURN MANEUVER

FIGURE 3 GAP ACCEPTANCE DISTRIBUTION OF PASSENGER
CARS FOR THROUGH MOVEMENT IN ONE MANEUVER

FIGURE 4 GAP ACCEPTANCE DISTRIBUTION OF PASSENGER
CARS FOR THROUGH MOVEMENT IN TWO MANEUVERS

FIGURE 5 GAP ACCEPTANCE DISTRIBUTION OF PASSENGER CARS FOR LEFT TURN MOVEMENT IN ONE MANEUVER

FIGURE 6 GAP ACCEPTANCE DISTRIBUTION OF PASSENGER CARS FOR LEFT TURN MOVEMENT IN TWO MANEUVERS

FIGURE 7 GAP ACCEPTANCE DISTRIBUTION OF
TRUCKS FOR ALL MANEUVERS COMBINED

Table 6. Average Gap Acceptance Values of the "Adjusted Gap Acceptance" Method for Passenger Cars

<u>Maneuver</u>	<u>Median Value (Seconds)</u>
Right Turn	5.80
Through (One Maneuver)	7.50
Through (Two Maneuvers)	6.50
Left Turn (One Maneuver)	6.25
Left Turn (Two Maneuvers)	5.70

The right turn average gap acceptance was found to be very close to the left turn average gap acceptance in two maneuvers. This result is as expected because the two maneuvers are similar. In the case of right turn maneuver, the driver has to find an acceptable gap in the first lane. In the case of left turn from median, the driver will stay in the median as long as an acceptable gap in the first lane does not occur. For both cases the driver is trying to find an acceptable gap in one lane whether it is on the left or the right side from the driver.

The average gap acceptance for through movement in two maneuvers was noticed to be higher than both cases right turn and left turn in two maneuvers. This can be explained by the fact that drivers need longer time to cross two lanes rather than to merge right or left with one lane.

The average gap acceptance of the through movement in one maneuver was found to be higher than the average gap acceptance of through movement in two maneuvers. A driver performing a through movement in one maneuver has to find an acceptable gap in four lanes to finish the crossing safely. However, the difference of 1.00 second between the two averages is not a large value, probably because drivers who cross the four lanes in one maneuver are mostly aggressive drivers. Similarly, the left turn movement in one maneuver is probably performed by large numbers of aggressive drivers who are willing to find suitable gap in three lanes.

Raff and Hart Critical Lag Acceptance

All the acceptance and rejection lags were tabulated for all maneuvers. The cumulative number of accepted lags were plotted versus the lag size measured in seconds. On the same plot, the cumulative number of rejected lags was plotted and the intersection of the two cumulative curves was defined to be the critical lag acceptance. The plots of the five maneuvers of passenger cars and the combined maneuvers of trucks are shown in Figures 8, 9, 10, 11, 12, and 13.

It was noticed that the critical lag acceptance values derived from this method (Raff and Hart) were greater than the values developed previously. It was believed that the small number of lags and the exclusion of time-gap values in the analysis were the main reasons for these variations. Hence, it was decided to use the gap acceptance values in another analysis to remove the bias resulting from the previous procedures. The method of analysis was similar to Blunden's method with some differences. This method was therefore called here the "Modified Blunden" method.

Modified Blunden Method

The gap and lag acceptance and rejection values for all the studied intersections were combined. Frequency tables were developed for each maneuver separately in which the lower limit of the gap size was chosen to be 0.125 second, the upper limit was 12.125 seconds, and the gap size increment was 0.500 second. For each gap size, the number of gap acceptances and gap rejections were recorded and the percentage acceptance for this particular gap size was calculated. The percentage

FIGURE 8. DISTRIBUTION OF ACCEPTED AND REJECTED
LAGS OF RIGHT TURN MANEUVER FOR PASSENGER CARS
(Raff and Hart method)

FIGURE 9. DISTRIBUTION OF ACCEPTED AND REJECTED LAGS OF
THROUGH MOVEMENT IN ONE MANEUVER FOR PASSENGER CARS
(Raff and Hart method)

FIGURE 10. DISTRIBUTION OF ACCEPTED AND REJECTED LAGS OF
THROUGH MOVEMENT IN TWO MANEUVERS FOR PASSENGER CARS
(Raff and Hart method)

FIGURE 11. DISTRIBUTION OF ACCEPTED AND REJECTED LAGS

FOR LEFT TURN IN ONE MANEUVER FOR PASSENGER CARS

(Raff and Hart method)

FIGURE 12. DISTRIBUTION OF ACCEPTED AND REJECTED LAGS OF LEFT TURN MANEUVER FROM MEDIAN FOR PASSENGER CARS (Raff and Hart method)

FIGURE 13. DISTRIBUTION OF ACCEPTED AND REJECTED
LAGS OF ALL MANEUVERS COMBINED FOR TRUCKS

(Raff and Hart method)

acceptance values were plotted versus the gap size and a logit model of "S" type curve was fitted. The fitted models of the five maneuvers for passenger cars as well as the combined maneuvers for trucks are shown in Figures 14, 15, 16, 17, 18, and 19.

The usual tests which are appropriate in a linear model case are, in general, not appropriate when the model is nonlinear (3). In order to test the significance of the logit model, the observed and predicted gap acceptance values were plotted and the slope and correlation coefficient of the line were checked. The results of this procedure are shown in Table 7. It can be noted in Table 7 that the slope of the linear fitting is very close to unity, the correlation coefficients are reasonably high and the intercept portion is very small. These results would indicate that the data does fit to the logit model well.

The main difference between the method suggested by Blunden (4) and the "Modified Blunden" method used here is that in Blunden's method the number of rejections of each gap size is proportionately reduced so that the total number of rejections is equal to the total number of acceptances. However, in the modified approach no such adjustment is used.

Comparison of Results

The results of the average gap acceptance values for the three methods are shown in Table 8. The "Adjusted Gap Acceptance" method results were found to be similar and consistent with the results of the "Modified Blunden" method. The difference between the values of the two methods were found to be as low as .07 second for the left turn without stop at median and as high as 1.29 seconds for trucks. The

FIGURE 14, GAP ACCEPTANCE DISTRIBUTION OF
PASSENGER CARS FOR RIGHT TURN MANEUVER
(Modified Blunden method)

FIGURE 15. GAP ACCEPTANCE DISTRIBUTION OF PASSENGER CARS FOR THROUGH MOVEMENT IN ONE MANEUVER
(Modified Blunden method)

FIGURE 16. GAP ACCEPTANCE DISTRIBUTION OF PASSENGER
CARS FOR THROUGH MOVEMENT IN TWO MANEUVERS
(Modified Blunden method)

FIGURE 17. ACCEPTANCE DISTRIBUTION OF PASSENGER
CARS FOR LEFT TURN IN ONE MANEUVER
(Modified Blunden method)

FIGURE 18. GAP ACCEPTANCE DISTRIBUTION OF PASSENGER CARS FOR LEFT TURN IN TWO MANEUVERS
(Modified Blunden method)

FIGURE 19. GAP ACCEPTANCE DISTRIBUTION OF
TRUCKS FOR ALL MANEUVERS COMBINED
(Modified Blunden method)

Table 7. Observed and Predicted Gap Acceptance Linear Fitting Results
(Modified Blunden Method)

<u>Maneuver Type</u>	<u>Intercept Portion</u>	<u>Slope of Line</u>	<u>Corr. Coeff. (R)</u>
Right Turn	-.024	1.0024	.986
Through (One maneuver)	-.028	1.0535	.962
Through (Two maneuvers)	-.024	1.02641	.982
Left Turn (One maneuver)	-.007	.98947	.912
Left Turn (Two maneuvers)	-.003	1.00885	.965
Trucks (All maneuvers)	-.072	1.05450	.954

Table 8. Average Gap Acceptance Values for the Three Methods of Analysis

<u>Type of Maneuver</u>	<u>Adjusted Gap Acceptance Method</u>	<u>Raff and Hart Method</u>	<u>Modified Blunden Method</u>
Right Turn Passenger Cars	5.80	7.10	6.73
Through Without Stop at Median Passenger Cars	7.50	7.25	7.90
Through With Stop at Median Passenger Cars	6.50	7.60	7.21
Left Turn Without Stop at Median Passenger Cars	6.25	6.55	6.32
Left Turn From Median Passenger Cars	5.70	6.70	6.60
All Maneuvers Combined for Trucks	9.65	9.50	8.36

results of the "Modified Blunden" method was chosen to be used as the gap acceptance distributions in the computer simulation model to analyze traffic flow through intersections of four-lane divided major highway with two-lane minor roads.

The gap acceptance distributions developed by Wagner (5) for the intersections of four-lane undivided state road with two-lane city streets showed that the 50th percentile of the peak period gap acceptance values for the left turn, right turn, and through maneuvers were found to be 6.25, 6.50, and 7.25 seconds respectively. According to "Modified Blunden" method results, these values were found to be 6.32, 6.73, and 7.90 seconds. It can be concluded that the average gap acceptance values for any maneuver developed by "Modified Blunden" method for this research were slightly more than the average gap acceptance values for the same maneuver developed by Wagner. The variation is possibly explained by the difference in vehicle speeds and the volume of traffic.

TRAFFIC CONFLICT STUDIES

The review studies involving safety measurements indicated that the traffic conflict technique is the best available procedure to measure safety at intersections, and it was decided that this procedure would be used for this research. The reasons for selecting the traffic conflict technique are listed below:

1. Reliable measure for predicting accident potential.
2. Conflicts are not rare events like traffic accidents.
3. Conflicts are easier to be calculated from traffic simulation model than accidents.

To prove the validity of the approach, it was decided to conduct appropriate traffic conflict studies at unsignalized divided intersections as well as at signalized divided intersections.

The traffic conflicts technique is a systematic method of observing traffic at an intersection in order to determine any existing potential for traffic accidents. For the purposes of this research, a traffic conflict is defined as any potential accident situation and may be identified by observation of: (1) evasive actions of drivers to avoid a collision, and (2) traffic violations. Evasive actions are evidenced by vehicle braking or weaving and are denoted through observation of brake lights or lane changes. A traffic violation is viewed as a conflict, even though only one vehicle is involved.

In this project, an evasive action of drivers to avoid collision was adopted as the conflict measure at unsignalized intersections and

a traffic violation at signalized intersections. These definitions are justified because of the specific nature of traffic flow at signalized and unsignalized intersections.

In the case of unsignalized intersections, five types of conflicts were observed:

1. Cross traffic conflict observed by brake light of the major highway vehicles due to minor road vehicles.
2. Major highway vehicle braking due to right turn minor road vehicles.
3. Major highway vehicle braking due to left turn minor road vehicles.
4. Major highway vehicle weaving due to right turn minor road vehicles.
5. Major highway vehicle weaving due to left turn minor road vehicles.

The five types of conflicts observed at unsignalized intersections are schematically shown in Figures 20, 21, and 22.

Traffic violations observed at signalized intersections were of the following type:

1. A minor road vehicle entered the intersection after the amber phase and the signal turned red before the vehicle entered the intersection.
2. A minor road vehicle ran a red signal.

Study Sites

Five unsignalized intersections controlled by stop sign on the minor road and five intersections controlled by fixed time signal were chosen. Listing of the study sites are shown in Table 9.

Fig.20 Cross traffic conflict

Fig. 21 Braking & weaving due to right turning vehicle

Fig. 22 ,Braking & weaving due to left turning vehicle

Table 9. Study Site Listing of Signalized and Unsignalized Intersections

<u>Unsignalized Intersections</u>		<u>Signalized Intersections</u>	
<u>Major Road</u>	<u>Minor Road</u>	<u>Major Road</u>	<u>Minor Road</u>
US-30	Burr St.	US-31 *	SR-28
US-31	SR-14	US-31 *	Kern Road ¹
US-31	SR-18	US-24	Getz Road ²
US-31	SR-38	US-31	National Ave. ³
SR-63	SR-163	SR-67	Franklin Rd. ⁴

¹South Bend By-Pass

²Fort Wayne By-Pass

³South side of Indianapolis

⁴South-west side of Indianapolis

* Operating as fixed-time signal during data collection.

Data Recording

Traffic conflict data was recorded by a two-man team utilizing hand counters and previously-prepared data forms. The counts were made from a vehicle located approximately 300 ft. from the intersection corner and parked in such a position as to be as inconspicuous as possible. Because of the fewer number of conflict types to be recorded, it was decided to make the two observers count traffic conflicts and other traffic movements simultaneously.

Three hours of conflict counts were performed at each unsignalized intersection between 8:00 a.m. and 12:00 p.m. on the first day and three hours between 1:00 p.m. and 5:00 p.m. on the second except for the intersection of State Road 63 and State Road 163 which had only the afternoon hours of observations. To minimize possible errors in the observation of conflict situation, the locations of the observers were switched during the second day. As for signalized intersections, one day of conflict study was considered. Most of the conflict studies at both types of intersections were performed during the weekdays of Tuesday, Wednesday, and Thursday. Hauer (6) showed in his study that the marginal increase in estimation accuracy diminishes rapidly as conflict counting period increases, and there is a little to be gained by counting conflicts longer than 3 days.

Unsignalized Intersections Results

The stepwise regression analysis was used as a tool to check the possible correlation between traffic volume and traffic conflicts. The dependent variable of these equations was number of conflicts and the independent variables were classified into passenger car and truck

volumes. The following variables were then considered for each volume classification:

1. Major road volume per hour.
2. Minor road volume per hour for right-turning maneuver.
3. Minor road volume per hour for through maneuver.
4. Minor road volume per hour for left turning maneuver.

Because the conflict rate had no replicates, normality of data and homogeneity of variances could not be tested (7). Therefore, for the purpose of the analysis these conditions were assumed to have been met.

Three sets of regression equations were developed. The first equation was fitted for cross traffic conflict data (type 1 conflict), the second equation was fitted for weaving and braking conflicts due to right turning vehicles (type 2, and type 4 conflicts combined), and the third equation was fitted for weaving and braking conflicts due to left turning vehicles (type 3, and type 5 conflicts combined). From the correlation matrix between the dependent and independent variables, the highest positive correlated independent variables were chosen and forced into the regression equations. Equations 3, 4, and 5 were then attained.

$$Y1 = -21.96 + .011 \sqrt{(X1) \cdot (X2)} + 1.812 \sqrt{X2} + 1.845 \sqrt{X5} + .000239(X1) \cdot (X5)$$

$$\text{Coefficient of determination } (R^2) = .488 \quad (3)$$

$$Y2 = -1.0367 + .00327(X3) \cdot (X5) + .0033(X1) \quad (4)$$

$$\text{Coefficient of determination } (R^2) = .582$$

$$Y3 = -2.1585 + .0121(X1) + .3793(X8) \quad (5)$$

$$\text{Coefficient of determination } (R^2) = .564$$

where: Y1 = Number of cross traffic conflicts (conf./hr.)
 Y2 = Number of right turning conflicts (conf./hr.)
 Y3 = Number of left turning conflicts (conf./hr.)
 X1 = Number of passenger cars on major road (veh./hr.)
 X2 = Number of passenger cars going through on minor road
 (veh./hr.)
 X3 = Number of passenger cars turning right from minor road
 (veh./hr.)
 X5 = Number of trucks on major road (veh./hr.)
 X8 = Number of trucks turning left from minor road (veh./hr.)

The three regression equations can be interpreted as discussed below:

1. The coefficient of determination values for the three equations were not high enough to allow the conclusion that conflicts are highly correlated with volumes.
2. The product of the square root of the major and minor passenger cars was the highest correlated independent variable for equation 3.
3. The product of the number of passenger cars turning right from the minor road and the number of trucks on major road was found to be the highest correlated independent variable for equation 4.
4. The number of passenger cars on the major road was the highest correlated independent variable for equation 5.

The summary of the average values of the three hour counts are shown in Table 10.

Table 10. Data Summary for Unsignalized Intersections

Intersection	Observations-Time	Average Conflicts			Average Conflicts			Total Conflicts Per Approach (conflicts/hr)	Average	
		Type 1	Type 2	Type 3	Type 4	Type 5	Type 5		Major Road Volumes (veh/hr)	Minor Road Volumes (veh/hr)
Burt St. x US 36	First day-morning	3.90	9.83	2.33	2.00	2.84	2.84	11.00	579.	102.
	Second day-afternoon	6.66	2.00	4.67	.83	4.34	4.34	18.50	741.	222.
SR 14 x US 31	First day-morning	.66	.17	.17	1.00	0.00	0.00	2.00	223.	100.
	Second day-afternoon	1.67	0.00	0.00	.33	.50	.50	2.50	175.	123.
SR 18 x US 31	First day-morning	1.50	.17	1.00	0.00	1.66	1.66	4.33	476.	98.
	Second day-afternoon	.83	.67	.66	2.00	2.34	2.34	6.50	333.	77.
SR 38 x US 31	First day-morning	2.50	.67	.67	6.50	.66	.66	11.00	431.	100.
	Second day-afternoon	2.83	.33	.83	5.67	1.34	1.34	11.00	363.	116.
SR 63 x SR 153	Afternoon hours	1.00	.25	.50	1.00	.75	.75	3.50	152.	84.

Correlation Between Conflicts and Accidents

The number of accidents were extracted from the accident reports summary provided by the Indiana State Highway Commission (ISHC). The average number of annual accidents for the 3 year period, 1974, 1975, and 1976, were calculated for each intersection separately. The accident averages were calculated for right angle accidents as well as for total accidents. The ADT values were provided by the planning division of the Indiana State Highway Commission in Indianapolis for the same three years and for both minor and major approaches. The 3 year average ADT values were then calculated for each intersection.

To determine the existence of correlation among volume of traffic, conflicts, and accidents the rank correlation technique (non parametric test) was considered (8).

Kendall Rank Correlation Method

To study the correlation among traffic volumes, traffic conflicts, and reported accidents the intersections were ranked in a descending order by these three variables. The purpose of the rank correlation test is to know if the rankings are substantially in agreement with one another. Kendall (9) has proposed a measure known as the "coefficient of concordance", W, to perform this test and this measure is defined by equation 6.

$$W = \frac{12S}{m^2 (n^3 - n)} \quad (6)$$

where: S = Sum of the squares of the deviations from the mean.

n = Number of intersections to be ranked.

m = Number of variables according to which intersections are ranked.

$m(n+1)/2$ = Mean value of the totals of ranks.

W varies from -1 to +1, -1 representing no community of preference, while +1 represents perfect agreement.

The procedure suggested by Kendall was applied for total conflicts as well as cross traffic conflicts and the results are shown in Table 11. The calculations of total conflicts showed that $S = 292$, and $W = .81$. The critical value for S , obtained from the appropriate tables in Kendall (9), was 176.1. Thus, it was concluded that there was no reason to reject the hypothesis that there exists a correlation among conflicts, traffic volume, and accidents, for level of significance .01. The cross traffic conflicts calculations yielded values of $S = 312.5$, and $W = .868$. The same conclusion, attained for the total conflicts test, was also reached for the cross traffic conflicts test for the same level of significance.

Conflict and Accident Rates

Conflict rates in term of conflicts per 1,000 opportunities were calculated by dividing average total conflicts per hour by average volume per hour on the minor approaches. The overall average of the conflict rate was calculated for the 5 intersections and it showed a value of 67 conflicts per 1,000 opportunities. The average accident rates per 1,000 vehicles were calculated for the five unsignalized intersections considering that the yearly traffic opportunities are the result of 365 days multiplied by the ADT values for the minor road. The overall average accident rates showed a value of 0.0528 accidents per 1,000 opportunities. The average conflict to accident ratio was found to be 1269.

Table 11. Kendall Test Results for Unsignalized Intersections Data

Conflict Class	Intersection	Average Conflicts Per Approach (conflict/hr)	Average Accidents (acc/year)	Major Road Volume (veh/hr)	(Major Road Volume × Minor Road Volume) 10^4 (veh/hr) ²	Average A.D.T. of Major Road	(Major Road A.D.T. × Minor Road A.D.T.) 10^7 (veh/yr) ²	Total Ranks
Total Conflicts	Burr x US 30	14.75 (1)*	11.00 (2)	719.00 (1)	8.52 (1)	22249. (1)	5.079 (1)	(7)
	SR 14 x US 31	2.25 (5)	9.00 (4)	180.60 (4)	1.09 (5)	4021. (4)	.678 (5)	(27)
	SR 18 x US 31	5.42 (3)	9.00 (4)	429.00 (2)	2.56 (3)	14068. (2)	3.256 (3)	(17)
	SR 38 x US 31	11.00 (2)	12.66 (1)	397.50 (3)	2.87 (2)	13407. (3)	3.88 (2)	(13)
	SR 163 x SR 63	3.50 (4)	9.00 (4)	151.80 (5)	1.26 (4)	3308. (5)	.781 (4)	(26)
Cross Traffic Conflicts	Burr x US 30	4.83 (1)	6.67 (2)	659.0 (1)	11.16 (1)	22249. (1)	5.079 (1)	(7)
	SR 14 x US 31	1.00 (4.5)	4.67 (5)	199.0 (4)	2.21 (4)	4021. (4)	.678 (5)	(26.5)
	SR 18 x US 31	1.16 (3)	5.33 (3)	404.0 (2)	3.65 (3)	14068. (2)	3.256 (3)	(16)
	SR 38 x US 31	2.66 (2)	7.00 (1)	396.0 (3)	4.26 (2)	13407. (3)	3.88 (2)	(13)
	SR 63 x SR 163	1.00 (4.5)	4.00 (4)	152.0 (5)	1.27 (5)	3308. (5)	.781 (4)	(27.5)

*Ranking values are shown in parentheses.

Signalized Intersections Results

One day of conflict study was performed at each signalized intersection. The average conflict values per hour and the average minor volume (vehicles per hour) were recorded from the field study. Reported accidents for the years 1974, 1975, and 1976 as well as the ADT values were obtained from the ISHC. The 3-year average ADT values for minor roads and the 3-year average accidents occurring due to running red light were calculated for the studied intersections. The Kendall rank correlation technique was applied for the signalized intersections data and the results of the test are shown in Table 12.

The test results showed that conflicts observed at signalized intersections, traffic volumes, and reported accidents at the same intersections were highly correlated at a level of significance $\alpha = .01$.

The results obtained by the Kendall correlation test for signalized and unsignalized intersections supported the findings of previous studies. It also showed with some confidence that the selection of the conflict technique as a measure of effectiveness for safety at intersection is a reasonable choice.

Purpose, Structure, and Operation of NETSIM

The simulation model is designed primarily to serve as a vehicle for testing relatively complex network control strategies under conditions of heavy traffic flow. It is particularly appropriate to the analysis of dynamically-controlled traffic signal systems based upon real-time surveillance of network traffic movements. It may also be used, however, to address a variety of other simpler problems, including

Table 12. Kendall Test Results for Signalized Intersections

<u>Intersections</u>	<u>Average Conflicts (Conflict/hr)</u>	<u>Average Accidents (Acc./year)</u>	<u>Average Minor Volume (Vehicle/hr)</u>	<u>Average Minor Road ADT</u>	<u>Total Ranks</u>
SR 28 x US 31	1.25 (5)*	1.00 (5)	109.0 (5)	1086.0 (5)	18
Kern Road x US 31	2.50 (3)	4.33 (2)	104.0 (4)	3933.0 (3)	12
Getz Road x US 24	3.50 (2)	3.00 (3)	170.0 (2)	4898.0 (2)	9
National Ave. x US 31	1.50 (4)	2.00 (4)	98.0 (5)	2179.0 (4)	17
Franklin Rd. x SR 67	6.00 (1)	11.33 (1)	309.0 (1)	15634.0 (1)	4

Sum of squares (S) = 134, Coefficient of Concordance (W) = .837

Critical Sum of squares = 176

*Ranking values are shown in parentheses.

the effectiveness of conventional traffic engineering measures, bus priority systems, and a full range of yield sign, stop sign, standard fixed-time, and actuated signal control strategies.

In order to run the model, an urban street network is first broken down into a set of uni-directional links and nodes. One link would represent a particular direction of travel along a single street between two adjacent intersections, and may contain up to five moving lanes. Mid-block changes in geometry may be accommodated by breaking a single block down into two or more successive links. Data needed to describe the network geometry are input on a link by link basis and include the length, grade, upstream, and downstream node numbers of each link. They also include a description of input traffic conditions, including input-flow-rates on each entry link, traffic composition, intersection turning movements, pedestrian traffic flows, and "source"/"sink" flows for each link. In addition, this category of inputs includes the type of control for each link, the percentages of bus as well as trucks, and information relative to short and long term events such as lane blockages that are to occur during the period to be simulated. The list also includes specification of the duration of the simulation sub-interval, a maximum value for network-load time, specification of output options desired and identification of clock-time value at the start of the first simulation sub-interval.

In addition to the exogenously supplied input data there is a set of embedded inputs specifying the values of several parameters to be used within one or more of the main simulation routines. The embedded data inputs include distribution of "leading" and "lagging" left turn probabilities, distribution of amber phase response, distribution of

acceptable gaps for traffic discharging from stop sign, maximum number of links, mean effective vehicle lengths by vehicle type, and other data. This data may be changed at the user's request.

The model proceeds in one second time steps. Vehicles are emitted onto the network via entry nodes on the periphery or source nodes within the network proper. Each vehicle has associated with it a set of performance characteristics which are assigned to it stochastically at the time it enters the network. These characteristics include designation of the "vehicle type", cumulative trip-time and travel-distance, cumulative delay and number of stops incurred to date, position within the network, projected turning movement at the next intersection, and driver category. The vehicle array is updated once every time step. Random code is used to access various distributions such as the driver's amber phase response, vehicle desired free-flow speed, vehicle queue discharge headways, duration of short-term events, and gap acceptance.

Standard statistical output report is generated at the end of each simulation sub-interval. The output parameters are shown in Table 13. The number of intermediate outputs as well as the elapsed time between successive intermediate printouts are included in the input stream according to the user desires. A sample of the intermediate output parameters are shown in Table 14.

Previous Modifications for the NETSIM Model

A recent research project conducted by Hall (12) was directed for the purpose of evaluating energy, air pollution, delay, and safety aspects of nonsignalized control at low volume intersections. The UTCS-1S version, used by Hall is a small version of the FHWA NETSIM

Table 13. Standard NETSIM Statistical Output

- Link identification
- Total vehicle miles of travel
- Average travel time per vehicle
- Average travel time per vehicle mile
- Count of vehicles discharged from the link.
- Average delay time per vehicle.
- Average delay per vehicle mile
- Total vehicle minutes of travel time
- Average traffic speed
- Average number of stops per vehicle
- Total delay time
- Ratio of moving to total travel time
- Average link saturation percentages
- Number of cycle failures
- Average link occupancy
- Total travel time to date for link

Table 14. Intermediate Link Statistics Output

- Link identification
- Occupancy
- Total cumulative vehicle discharge
- Total, cumulative left turning, through, and right-turning traffic flows
- Queue length by lane
- Average delay per vehicle
- Percent stopped delay
- Number of cycle failures
- Number of rare event occurrences
- Current status of channelization
- Cumulative number of stops
- Signal phasing at the downstream end of

package without the Post-Processor overlay. In this package all statistical testing must be completed exogenously to the simulation runs. Removal of this overlay was but the first of several subsequent changes made to the UTCS-1S.

The second change was made to permit close study of the velocity patterns created by automobiles traversing an intersection from the controlled (signed) approaches. Utilizing an adapted version of the EPA's Automotive Exhaust Emission Modal Analysis Model (13), the UTCS-1S output was used as an input to the EPA model to examine gasoline consumption and air pollution resulting from different control strategies across a range of traffic volumes.

The third change in the model involved the estimation of the noise level generated by traffic in the vicinity of stop and yield control intersections.

New Modifications to the Simulation Model

The main advantage of the UTCS-1S model is that it is a microscopic model which simulates the traffic flow of an urban street network in detail. It is designed as a laboratory tool for testing alternative network control strategies with particular emphasis on relatively sophisticated signal control schemes. Stop and yield signs are also handled by this model, however, they are not given emphasis as signal control. Furthermore, urban street traffic has different characteristics compared to suburban and rural traffic due to volume and speed differences. Consequently it was decided to re-evaluate and calibrate the simulation model for the suburban and rural traffic environment.

In order to simulate traffic at the intersection of four-lane major divided highways and two-lane minor highways in the UTCS-1S, the intersection was treated as a combination of two separate intersections with one way traffic in each direction and separated by a very short link which represents the median. The network simulating this intersection controlled by stop sign on minor road and stop or yield signs at the median is shown in Figure 23. Several points were considered in modifying the logic of the UTCS-1S to simulate traffic at this type of intersections, as discussed below:

1. Field observations showed that, in spite of the adequacy of the median width for a car to stop, many drivers tend to make their through or left maneuver in one move.
2. The decile distribution of acceptable gaps for a vehicle at a stop-line facing a stop sign, stored in the UTCS-1S as embedded data, provides acceptable gaps for near-side (or one-way) cross street traffic. An additive time factor is applied for gaps in far-side cross street traffic to account for the time required for the discharging vehicle to reach the far side. The additive time factor is a function of the number of lanes crossed at an acceleration of 8 feet per second per second. It was decided to eliminate this gap acceptance decile distribution from the model and replace it with the appropriate distributions because the existing algorithm does not consider in detail the different maneuvers at an intersection of the type under consideration in this study.
3. The logic of the traffic simulation model is designed so that traffic flows through intersections safely without encountering collisions and the model is self adjusted for preventing accidents.

FIGURE 23, A NETWORK SIMULATING THE INTERSECTION OF FOUR LANE DIVIDED HIGHWAY AND TWO-LANE MINOR ROAD

Minor modifications were implemented to several subprograms and the major changes were incorporated in six routines. The functions of those modified routines are:

1. STPSN: This routine tests the gaps in cross traffic to determine whether a vehicle can discharge after encountering a STOP or YIELD sign.
2. TSIG: This routine tests the signal for a vehicle approaching an intersection. Vehicles continue through the intersection if the signal is green, however if the signal is amber, a decision of stopping or continuing is given to the driver based on an acceptable deceleration rate.
3. TRVL: This routine represents the non-queued vehicle specified as moved along the link it occupies in response to traffic conditions in its immediate vicinity. One of a set of scenarios based upon car-following logic is adopted for this vehicle; its choice depends upon known traffic conditions which prevail at this time. A typical speed profile and a stochastically determined desired speed provide the bases for the vehicles trajectory. The vehicle could switch lanes if conditions indicate it can also discharge into another link.
4. LFTRN: This routine processes a vehicle which is attempting an unprotected turning maneuver upon discharge from a link. It tests the availability of a sufficient gap in on-coming traffic to determine whether a turn can be successfully negotiated.

5. CLRAL: This routine clears all appropriate COMMON storage arrays and primes storage with the embedded calibration data.
6. SIGACT: This routine updates all signals at a traffic actuated intersection. Various timers are updated in the process.

The details of these modifications are shown in Appendix B.

VALIDATION OF THE SIMULATION MODEL

The NETSIM model has been calibrated against data collected during previous studies; however, most of the testings have considered signal-controlled traffic networks. No study documents the validity of the simulation for single, rural, unsignalized intersection with provision of median on the major road.

This chapter discusses the validation of the NETSIM model conducted for the purpose of the present study. The first phase was to validate delay calculated by the model and then use the model to compare average delay per vehicle between divided and undivided intersections under a wide range of traffic volumes. In addition, the simulated delay was compared against delay calculated from a queuing model.

The second phase was to validate the model for the purpose of detecting conflicts at unsignalized intersections. The final phase dealt with the validation of conflicts at signalized intersections which were observed from the simulation model.

Validation of the NETSIM

The simulation model NETSIM was developed in two phases. Both phases were calibrated and validated for a 16-block grid network located in downtown Washington, D. C. Separate comparisons between field data and a series of model runs were made for morning peak and morning off peak conditions (14). The results of the statistical

analysis showed that the model accurately replicated the observed traffic performance on the network under morning peak traffic conditions; however, the comparison was less satisfactory under off peak traffic conditions. The validation results indicated that the model replicates the specific details of traffic behavior more accurately under conditions of relatively disciplined as opposed to undisciplined flow.

The model was also tested in a more complex application for a large network in San Jose, California (15). The purpose of this test was to illustrate the adaptability of the model to network and traffic conditions which differed significantly from those for which it was originally validated in Washington, D. C. A secondary objective was to use the model to evaluate two different signal optimization procedures "SIGOP" and "TRANSYT". The results of these tests indicated that the NETSIM can be used satisfactorily as a tool in evaluating various signal timing plans.

Delay Validation at Stop Controlled Intersections

In order to use the modified version of NETSIM for simulating traffic at isolated major divided highway intersections located in rural areas, it was necessary to conduct a calibration procedure which would reflect the real life conditions. The field traffic data was recorded by a movie camera. A 100 foot film was used for the purpose. This film was taken on a weekday with clear weather and dry pavement conditions for the intersection of State Road 18 and U.S. 31 north of Kokomo. The film was examined frame-by-frame to observe the following:

1. Delay time for each minor road vehicle from the time it stops at the minor approach to the time it crosses or merges with the major approach traffic.
2. The number of vehicles on each of the four approaches of the intersection and their turning percentages.
3. Composition of traffic.
4. Total time of observations in seconds.

Table 15 presents the delay values procured from the film.

Volume and geometric data descriptive of this intersection were fed to the simulation model. The desired free-flow speed for the major road approach was set to be 55 mph, however, the desired speed for the minor road did not have a fixed value. Several runs were performed using different free-flow speeds and it was found that the following figures represent the field conditions:

1. Desired free-flow speed of 45 mph for minor road links upstream of stop lines.
2. Desired free-flow speed of 10 mph for vehicles discharging from the minor road to the intersection median.
3. Desired free-flow speed of 25 mph for vehicles discharging from the intersection median to the minor road.

Intermittent links were inserted between source nodes and minor road approaches to permit vehicles to reach desired free-flow speed before beginning the deceleration process.

One problem inherent in the calculation of delay in the simulation model is that the total delay per vehicle is calculated per link. This means that the delay experienced by each turning vehicle is not counted

Table 15. SR 18 - US 31 Two Way Stop Observed Delay in Seconds

<u>SR 18 East Approach</u>			<u>SR 18 West Approach</u>		
<u>Right</u>	<u>Through</u>	<u>Left</u>	<u>Right</u>	<u>Through</u>	<u>Left</u>
5.00	17.25	9.75	6.25	34.75	23.00
8.25	23.25	10.00	2.75	14.75	13.25
4.00	17.50	19.75	4.75	39.50	12.75
14.50	23.25	16.00	5.25	20.00	29.75
14.50	22.75	15.00		7.50	20.75
17.50	11.50	12.50		35.00	17.00
4.50	25.50	18.00		39.50	17.00
13.00	22.25	6.50		33.25	20.75
5.00	8.50	7.50		22.00	5.50
	16.75	26.50		7.50	
	15.75	4.25		15.25	
	26.00	5.00		8.75	
	7.25	14.25		6.75	
	21.75	10.00		16.75	
		15.00		24.00	
		12.75		22.50	
				20.50	
				14.75	

Average = 15.853 $\pm$ 8.808

separately, therefore, it was decided to combine all the turning vehicle delays with the through movement vehicle delays. Since major road vehicles are not expected to experience stop or queue delay, it was assumed that the average queue delay per vehicle in seconds for the two minor road approaches calculated from the simulation should be checked against the observed delay data. Based on the variation of data within mean from the first five replications, it was determined that five more replicates would be sufficient to test the hypothesis of equality of means at significance levels of $\alpha = .05$ and $\beta = .05$.

The student "t" test was chosen for testing the significance between the observed and simulated data. This test requires the assumptions of normal distribution and equal variances.

A natural logarithmic transformation made both simulated and field observations to be normal. A homogeneity test of variances was conducted for the transformed data and it was found that the two variances came from the same distribution. The statistical comparison, presented in Table 16 indicated that there was no significant difference between vehicle queue delay calculated by the simulation model and the field measurements.

Verification of Delay Results

After the model was validated for traffic delay it was decided to verify the delay results with a theoretical method developed for the Swedish Highway Capacity Manual (16). The method is based on a queuing model in which each lane was considered as a service station. A group of charts was developed to represent the waiting time of minor road

Table 16. Statistical Comparison Between Observed and Simulated Delay Data

Null Hypothesis

Alternative Hypothesis

$$H_0: \mu_{sim} = \mu_{obs}$$

$$H_1: \mu_{sim} \neq \mu_{obs}$$

$$\alpha = .05$$

$$t = \frac{\bar{X}_{sim} - \bar{X}_{obs}}{Sp \sqrt{\frac{1}{n_{sim}} + \frac{1}{n_{obs}}}}$$

where

$$Sp = \sqrt{\frac{(n_{obs} - 1) S_{obs}^2 + (n_{sim} - 1) S_{sim}^2}{n_{obs} + n_{sim} - 2}}$$

$$t = \frac{2.63 - 2.59}{.27 \sqrt{\frac{1}{700} + \frac{1}{70}}} = 1.182$$

$$t_{769, .05} = 1.645$$

vehicles as a function of the primary flow which occupied two lanes per direction. The waiting delay observed from these charts considered the provision of an intersection median between the two directions. Traffic flow rates, observed from the 100 foot film exposed at the intersection of SR 18 and US 31, were used to calculate the average waiting time using the theoretical method. This method yielded an average waiting time of 13.00 seconds, which is less than the average observed delay of 15.85 seconds and the simulated value of 16.60 seconds. The difference can be attributed to the value of running delay not calculated by the queuing theory method.

Delay-Volume Relationships

Several computer runs were performed for both divided and undivided highways under different flow rates to study the relationship between the delay per vehicle calculated from the simulation model and the different flow rates. Five flow rates of 200, 350, 500, 650 and 800 vehicles per hour for major roads and two flow rates of 50 and 100 vehicles per hour for minor roads were studied. For each minor-major flow rate combination, three replications of 600 seconds of simulation time were considered. The 800 vph major road volume and 100 vph minor road volume were thought to be the maximum flow rates at stop-controlled intersections beyond which traffic signals are warranted. Foster-Burr test of homogeneity of variance (Q test) was conducted for both divided and undivided highway intersections and for the minor road flow rates. The results of the test showed that the variances of all cells for major-minor volume combinations were equal for the level of significance

$\alpha = .05$. Test of normality (W test) was satisfied for all data cells. A linear regression was tried to fit the relationship between average delay per vehicle as dependent variable and major road volumes as independent variables. The fitted lines are presented in Figure 24. To test the adequacy of the linear fit for the delay data, the pure error for the repeated measures were calculated and tests of lack of fit were determined. The results of these tests, shown in Tables 17 and 18, indicated that there is no reason to doubt the adequacy of the models at level $\alpha = .05$ and both pure error and lack of fit mean squares can be used as estimates of σ^2 .

The delay-flow rate relationship was noticed to be positively correlated and the slope of the fitted line for the undivided highways case was found to be much higher than for the case of divided highways. This result was expected because the median permits drivers to perform their through maneuver in two steps and consequently they experience less delay. Another phenomenon worth mentioning is that the delay for undivided highways was found to be less than the delay for divided highways as long as the major flow rates are less than 290 and 315 vehicles per hour at minor flow rates of 100 and 50 vehicles per hour respectively.

Traffic Conflicts at Stop Controlled Intersections

The accident history of unsignalized intersections with wide median on the major highway showed that the majority of reported accidents were caused by minor road vehicles. These accidents are of two types: (1) rear-end accidents and (2) right-angle accidents.

FIGURE 24, DELAY FLOW RATE LINEAR FITTING

Table 17. Lack of Fit Test Results for Undivided Highway Intersections

Case 1: Minor Volume = 50 vph

<u>Analysis of Variance</u>	<u>Degrees of Freedom</u>	<u>Sum of Squares</u>	<u>Mean Squares</u>	<u>F</u>	<u>Significance F</u>
Regression	1	581.7684	581.7684	14.8915	0.002
Residual	13	507.8703	39.0669		
Lack of Fit	3	72.0903	24.0301	<1	
Pure Error	10	435.7800	43.578		

Case 2: Minor Volume = 100 vph

Regression	1	17190.3565	17190.3565	27.6949	0.000
Residual	13	8069.1545	620.7041		
Lack of Fit	3	3510.1488	1170.0496	2.56	0.105
Pure Error	10	4559.0057	455.9005		

Table 18. Lack of Fit Test Results for Divided Highway Intersections

Case 1: Minor Volume = 50 vph

Analysis of Variance	Degrees of Freedom	Sum of Squares	Mean Squares	F	Significance F
Regression	1	9.2740	9.2740	12.6608	0.004
Residual	13	9.5224	0.7325		
Lack of Fit	3	2.5329	0.8443	1.2080	0.375
Pure Error	10	6.9894	0.6989		

Case 2: Minor Volume = 100 vph

Regression	1	47.9561	47.9561	13.8680	0.003
Residual	13	44.9544	3.4580		
Lack of Fit	3	21.0070	7.0023	2.9240	0.085
Pure Error	10	23.9474	2.3947		

These two types can be represented by observing the following type of conflicts:

1. Cross traffic conflicts.
2. Braking and lane changing conflicts caused by right turning minor road vehicles.
3. Braking and lane changing conflicts caused by left turning minor road vehicles.

Subroutine STPSN in the simulation model was modified to count the number of cross traffic conflicts occurring at stop-controlled intersections. Minor road drivers accepting small gaps in the major flow was the main cause of cross traffic conflicts. A critical gap acceptance of 5.00 seconds, below which cross traffic conflicts occur, was assumed. The dimension of the vehicle array in the simulation model, which contains vehicle specific data, was increased in size to make it possible for the computer program to keep track of the minor road vehicles in the network and to store codes in order to prevent conflicts double counting.

Once a vehicle is discharged from subroutine STPSN, a turning code is stored in its vehicle array. The second step was modifying subroutine TRVL to detect the other two types of conflicts. This subroutine is called every time increment (1 second) to move a vehicle along the link it occupies in response to traffic conditions in its immediate vicinity. The vehicle could switch lanes if conditions indicate. Once a spacing between two vehicles is spotted to be less than 6 feet and the leading vehicle is a minor road vehicle, two cases are considered:

1. Check the possibility of changing lanes. If traffic conditions permit that, a lane switch conflict is counted.
2. If the condition of traffic does not permit lane changing, a brake conflict type is recorded.

The conflict data collected earlier from the field study at five stop controlled intersections was used to check the validity of the conflict logic. The average 15 minutes conflict counts were calculated for each intersection for the period of study. Ten traffic simulation runs, each of 900 seconds, were studied for every intersection. It was noticed that the conflict logic tends to under-count the number of conflicts occurring at low volume rates on the minor road and over-count them at high volume rates. Therefore it was decided to check the validity of that logic for the average conditions of the studied intersections. Traffic flow rates, truck percentages, and turning movement percentages were averaged for the five studied intersections and then ten simulation runs were performed for the average data. The average number of conflicts, predicted from the simulation runs, were compared with the average number of conflicts observed from the field study and it was proved that the two values were equal at level of significance $\alpha = .05$. This conclusion proved that: on the average, the conflict logic of the model for unsignalized intersections works well.

To predict the number of conflicts occurring at any intersection from the simulation model, it was decided to fit a linear regression model between the ratio of observed and simulated conflicts as the dependent variable and the minor flow rate as the independent variable. The results of the regression analysis showed a correlation coefficient

r of 0.952 and a regression significance F value of 0.012. The linear model is:

$$Y = 2.374 - 0.01852 X$$

where: Y = Ratio of simulated and observed conflicts.

X = Flow rate on the minor road in vehicles per hour.

Using this equation, one is then able to estimate the number of conflicts at unsignalized intersection from the simulated figures.

Traffic Conflicts at Signalized Intersections

The decision of proceeding through the intersection or anticipating a change of a traffic signal to an amber phase is sometimes referred to as the "dilemma zone". Inappropriate decision by some drivers result in numerous rear-end and right-angle collisions at intersections where the flow of traffic is at a fairly high speed. High speeds of approaching vehicles at rural intersections leads to insufficient time for perception, reaction, and braking. In similar situations, there have been attempts to decrease the number of rear-end and right-angle collisions by installing green-phase extension systems (GES systems) (17).

Zegeer (17) studied the effectiveness of the green-extension system (GES) for reducing the dilemma-zone problem associated with the amber phase of traffic signals at high-speed intersections. Conflict, volume, delay, and speed data were taken before and after GES installation at two sites in Kentucky. The conflicts used in this study were revisions of the General Motors method (18) and were adapted to the dilemma-zone

problem. The observed conflicts were: (1) run red light, (2) abrupt stop, (3) swerve-to-avoid collision, (4) vehicle skidded, (5) acceleration through yellow, and (6) brakes applied before passing through. Two days of conflict observations were considered for each site and the hourly traffic volumes were recorded too. Signal timing and a detailed diagram were fully documented for both intersections. The intersection of US 23 and Hoods Creek Pike in Ashland, Kentucky was chosen to be used as a sample for validating conflicts calculated from the simulation model for signalized intersections. Traffic volume and conflict data before CES installation as well as signal timing are shown in Table 19. Diagram of the intersection of interest is shown in Figure 25.

To determine the length of the dilemma zone in Zegeer's study, driver responses were recorded at nine high-speed intersections in Lexington and Louisville, Kentucky. All intersections were on four-lane, divided arterials. Two observers were used to record the data: one monitored the speed of each vehicle approaching the intersection, and the other watched for the yellow indication. Responses of about 2,100 drivers to the yellow phase were recorded. A set of curves for speeds of 35 to 55 mph was drawn from the data to represent relationship between the distance back from the stop bar at which the signal phase amber was observed and the percent probability of stopping. Assuming that Indiana drivers have the same response for amber phase signals as Kentucky drivers, a value of 308 feet for distance back from the stop bar was chosen to represent the stopping distance for the average driver at vehicle speed of 55 mph. Knowing the initial speed of vehicle and the safe stopping distance, the rate of deceleration was calculated. Using

Table 19. Traffic Volume, Conflicts, and Signal Timing Data for Ashland Before GES Installation

(Source: Ref. 68)

<u>Time of Day</u>	<u>Traffic Volumes (Two-Directional)</u>			<u>No. of Conflicts Per Hour</u>
	<u>Cars</u>	<u>Trucks</u>	<u>Total</u>	
8:00 to 9:00 a.m.	932	95	1,027	4
9:00 to 10:00	853	85	938	11
10:00 to 11:00	956	94	1,050	4
11:00 to 12:00	1,222	84	1,306	21
12:00 to 1:00 p.m.	1,320	83	1,403	26
1:00 to 2:00	1,390	90	1,480	25
2:00 to 3:00	1,548	96	1,644	23
3:00 to 4:00	1,607	97	1,704	19
4:00 to 5:00	1,431	75	1,506	32
5:00 to 6:00	1,867	56	1,923	25

Phase A (US 23)

Maximum Green = 35 seconds

Amber = 4 seconds

Phase B (Hoods Creek Pike)

Initial Interval = 5 seconds

Vehicle Interval = 5 seconds

Maximum Extension = 20 seconds

Amber = 4 seconds

FIGURE 25. DIAGRAM OF INTERSECTION OF US 23 AND HOODS CREEK PIKE IN ASHLAND, KENTUCKY

the distribution of driver's type embedded in the simulation model for the amber phase response, a new decile statistical distribution was developed to be used in the model for the purpose of calculating conflicts at signalized intersections.

To determine the statistical reliability that the simulation model represents the real world conflict data, a mean difference test (t-test) was used. The sampling periods were the 15-minute intervals for recording conflicts and volumes. The sample size n , for the field data was 29 time intervals. A sample size of 10 time intervals was used to represent simulated data. Averaging the ten hourly volume counts, shown in Table 19, and using the geometrical design data together with the signal timing of this intersection, ten computer runs each of 15-minute intervals were run. The mean conflicts per 15-minute period were 4.30 and 4.34 for the observed and simulated cases, respectively. The results of the statistical test are shown in Table 20. Since it is desired to fail to reject the null hypothesis, one must consider the probability of committing a type II error - accepting the null hypothesis, H_0 , when it is false (β). The true value of β , type II error, is impossible to calculate unless a specific alternative hypothesis is specified. It was desired to detect differences in the true population of the conflicts occurred by a value of 2.0. Given this value, one could then calculate the value of β at an α level of 0.40. Since an increase in the level of significance generally results in a decrease in the probability of committing a type II error, the tests were conducted at a relatively high α . The calculated value β was found to be .08. From the results of the t-test and the calculated value for type II error, it was concluded that the simulation model can be used with high confidence in detecting conflicts created at signalized intersections.

Table 20. Results of T-Test for Traffic Conflicts

$$H_0: \mu_{\text{obs}} = \mu_{\text{sim}}$$

$$H_1: \mu_{\text{obs}} \neq \mu_{\text{sim}}$$

$$\alpha = .40$$

	<u>Observed</u>	<u>Simulated</u>
n	10	29
mean	4.34	4.30
S _d	3.59	4.69
*Sp		3.88
**t		.028
t _{37, α/2}		.255
d.f.		37

$$* \text{ Sp} = \sqrt{\frac{(n_{\text{obs}} - 1) S_{\text{obs}}^2 + (n_{\text{sim}} - 1) S_{\text{sim}}^2}{n_{\text{sim}} + n_{\text{obs}} - 2}}$$

$$**t = \frac{X_{\text{obs}} - X_{\text{sim}}}{\sqrt{\text{Sp} \left(\frac{1}{n_{\text{obs}}} + \frac{1}{n_{\text{sim}}} \right)}}$$

CONCLUSIONS

A gap acceptance field study was conducted to evaluate driver characteristics at stop controlled intersections of a minor road with a multi-lane, divided rural highway. In addition, traffic conflict studies were performed at both signalized and unsignalized intersections. A computer simulation package UTCS-1 (known now as NETSIM) was modified and validated. The following is a brief summary of the significant findings of this phase of the study:

1. The average passenger car driver stopped at a stop sign on the minor road of a multi-lane highway intersection required an average time gap of 6.73 seconds to perform a right turn maneuver. Those gap acceptance values were found to be 7.90 and 6.32 seconds for through, and left turn maneuvers without stopping at the intersection median, respectively. In addition, the required gap acceptance figures for through and left turn maneuvers from the median location, for the case of stopping at the intersection median, amounted to 7.21, and 6.60 seconds. The average truck driver accepted an average gap of 8.36 seconds to perform a turning or crossing maneuver.
2. Traffic conflict studies conducted at unsignalized divided intersections showed a very high correlation rate among observed traffic conflicts, traffic volumes, and reported traffic accidents. The average traffic conflict rate was found to be 67 conflicts per 1,000 opportunities. In addition, the average

accident rate amounted to 0.0528 accidents per 1,000 opportunities; therefore, the average conflict to accident ratio was 1269. Similarly, traffic conflict studies at signalized intersections concluded that traffic conflicts, traffic volumes, and reported accidents were correlated. The results of the conflict studies at both types of intersections indicated, with some confidence, that the number of traffic conflicts is an effective measure of safety at intersections.

3. The modified version of UTCS-1 can be used, with some confidence, to estimate traffic conflicts and delay at intersections of a minor road with multi-lane, divided rural highways.

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APPENDIX A

TABLE A-1-1 ADJUSTED METHOD DATA FOR RIGHT TURN MANEUVER

GAP ACCEPTANCE VALUES WEIGHTED BY NO. OF		REQ. FOR		RIGHT TURN			
5.151	6.250	8.863	6.945	7.000	9.159	9.750	10.500
12.000	12.500	12.500	12.250	12.621	11.750	13.393	13.242
14.318	5.305	5.500	5.471	5.000	6.776	6.447	12.750
6.636	6.315	6.500	6.773	6.963	7.147	8.421	6.315
8.678	7.913	8.678	8.157	8.768	8.934	9.250	8.336
9.026	9.500	9.899	9.500	9.000	10.681	10.250	9.799
9.750	10.863	12.368	12.500	12.121	13.389	12.750	10.500
14.250	13.750	4.083	4.500	4.000	5.500	5.250	14.000
5.552	6.000	6.000	6.500	6.375	6.500	5.670	5.208
7.031	7.000	7.000	6.670	7.635	7.000	7.437	7.401
7.750	8.000	8.166	8.000	7.989	8.750	9.330	8.500
9.000	9.330	9.031	10.625	10.208	10.760	10.000	9.031
11.000	11.000	11.333	11.000	11.000	12.500	12.156	11.000
13.000	13.270	14.291	14.291	14.000	5.028	5.555	13.500
5.500	6.500	6.500	6.536	6.285	6.500	6.033	5.624
7.542	6.646	7.039	7.750	8.536	8.225	8.740	7.250
8.548	8.090	7.750	8.090	9.101	9.607	8.848	8.000
10.250	10.169	10.169	10.307	9.750	10.395	10.000	8.389
10.000	10.388	10.553	10.338	11.000	11.500	10.225	9.000
11.807	12.000	12.338	12.711	12.000	12.000	11.759	9.805
13.614	13.000	13.000	13.293	13.073	13.073	12.641	11.435
14.237	14.158	14.000	14.079	14.000	14.079	12.894	12.197
						4.657	14.000
						5.250	7.697
						6.368	
						10.045	
						15.363	
						6.250	
						8.239	
						9.600	
						10.357	
						12.750	
						5.500	
						6.461	
						7.583	
						9.000	
						10.625	
						12.000	
						5.279	
						7.754	
						8.389	
						9.254	
						10.395	
						11.186	
						12.570	
						14.079	
						6.368	

TABLE A-1-2 ADJUSTED METHOD DATA FOR THROUGH ONE MANUEVER

CAP ACCEPTANCE VALUES WT. BY NO. OF										REJ. FOR THROUGH MOVEMENT										IN ONE MANUEV.																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
11.754	11.000	11.970	11.500	11.911	13.058	13.000	8.229	9.364	9.330	10.783	10.000	13.364	12.912	13.250	13.351	13.690	15.283	16.851	15.283	17.048	4.000	5.500	5.000	5.279	5.000	5.110	5.781	6.571	6.500	7.011	7.490	6.899	7.011	9.657	10.165	10.589	10.868	12.066	14.879	6.175	7.554	8.686	9.056	9.556	8.750	10.399	10.516	11.332	12.350	13.598	14.000	14.965	15.000	5.024	7.537	7.750	8.000	8.836	9.000	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
6.066	6.823	6.250	6.535	5.908	5.877	7.011	7.490	6.899	7.011	7.750	8.500	8.295	8.000	6.687	6.593	8.546	8.500	8.044	9.657	10.165	10.589	10.868	12.066	14.879	6.175	7.554	8.686	9.056	9.556	8.750	10.399	10.516	11.332	12.350	13.598	14.000	14.965	15.000	5.024	7.537	7.750	8.000	8.836	9.000	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
9.198	9.352	9.657	9.604	9.505	9.000	9.709	9.750	10.589	10.165	10.589	10.868	12.066	14.879	6.175	7.554	8.686	9.056	9.556	8.750	10.399	10.516	11.332	12.350	13.598	14.000	14.965	15.000	5.024	7.537	7.750	8.000	8.836	9.000	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
10.616	10.419	10.674	9.803	10.000	10.476	9.857	11.250	11.060	10.868	12.066	14.879	6.175	7.554	8.686	9.056	9.556	8.750	10.399	10.516	11.332	12.350	13.598	14.000	14.965	15.000	5.024	7.537	7.750	8.000	8.836	9.000	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
11.563	11.250	11.243	10.750	11.312	10.809	11.563	12.009	12.009	12.066	14.879	6.175	7.554	8.686	9.056	9.556	8.750	10.399	10.516	11.332	12.350	13.598	14.000	14.965	15.000	5.024	7.537	7.750	8.000	8.836	9.000	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
12.000	13.071	13.574	14.171	13.872	13.762	13.689	12.750	14.486	14.879	6.175	7.554	8.686	9.056	9.556	8.750	10.399	10.516	11.332	12.350	13.598	14.000	14.965	15.000	5.024	7.537	7.750	8.000	8.836	9.000	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
13.825	14.801	14.564	14.000	14.000	5.166	5.250	5.500	6.295	6.175	7.554	8.686	9.056	9.556	8.750	10.399	10.516	11.332	12.350	13.598	14.000	14.965	15.000	5.024	7.537	7.750	8.000	8.836	9.000	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
6.500	6.341	6.000	6.341	7.500	7.000	7.051	6.848	6.799	7.554	8.686	9.056	9.556	8.750	10.399	10.516	11.332	12.350	13.598	14.000	14.965	15.000	5.024	7.537	7.750	8.000	8.836	9.000	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
7.051	8.500	8.250	8.430	8.058	8.291	8.291	8.000	7.750	8.686	9.056	9.556	8.750	10.399	10.516	11.332	12.350	13.598	14.000	14.965	15.000	5.024	7.537	7.750	8.000	8.836	9.000	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
9.250	9.250	9.065	9.569	9.000	9.385	9.394	9.452	8.750	9.056	9.556	8.750	10.399	10.516	11.332	12.350	13.598	14.000	14.965	15.000	5.024	7.537	7.750	8.000	8.836	9.000	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
9.520	10.000	10.218	10.576	10.399	10.000	10.500	10.399	10.516	11.332	12.350	13.598	14.000	14.965	15.000	5.024	7.537	7.750	8.000	8.836	9.000	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
11.000	11.080	12.339	11.332	12.000	12.500	12.591	12.591	12.350	12.339	12.350	13.598	14.000	14.965	15.000	5.024	7.537	7.750	8.000	8.836	9.000	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
12.428	12.875	12.250	12.250	12.175	13.000	13.000	13.250	13.500	13.598	14.000	14.965	15.000	5.024	7.537	7.750	8.000	8.836	9.000	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
13.284	13.830	13.598	14.000	16.087	15.180	14.750	14.857	14.965	15.000	5.024	7.537	7.750	8.000	8.836	9.000	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
14.750	14.750	5.554	4.797	5.500	5.123	5.500	5.327	5.356	5.024	7.537	7.750	8.000	8.836	9.000	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
6.029	6.000	7.014	5.698	6.392	7.250	7.648	7.648	7.000	7.537	7.750	8.000	8.836	9.000	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
7.500	7.648	7.438	8.584	8.000	8.752	8.000	8.039	8.000	7.750	8.000	8.836	9.000	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																										
8.500	8.158	8.371	8.000	8.079	8.277	9.500	9.991	8.836	9.000	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
8.750	9.376	9.376	8.755	9.000	8.841	9.330	10.148	10.099	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
10.603	10.346	9.750	10.759	10.198	11.500	11.417	10.803	10.803	11.500	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																
11.599	11.305	10.722	11.326	10.934	11.386	11.498	12.561	12.475	12.685	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
12.995	11.866	12.118	12.178	11.785	12.391	11.727	11.843	12.939	13.257	12.939	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
13.193	13.450	13.461	14.000	14.815	4.516	4.532	5.559	5.330	5.363	5.943	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
6.608	6.250	6.150	6.250	6.107	5.730	5.852	7.351	7.075	6.943	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
6.870	7.301	6.750	7.431	7.795	7.500	7.150	8.560	8.652	8.086	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																							
8.258	7.972	8.397	8.086	7.670	8.688	9.316	9.129	9.500	9.000	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
9.602	9.349	9.747	8.969	9.354	9.283	9.250	9.500	8.732	8.701	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
9.998	10.035	9.784	10.500	9.854	10.286	9.750	10.000	11.157	11.500																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
11.500	10.827																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			

TABLE A-1-3 ADJUSTED METHOD DATA FOR THROUGH TWO MANEUVERS

GAP	ACCEPTANCE	VALUES	WT. BY	NO. OF	REJ. FOR	THROUGH	MOVEMENT
10.065	7.750	7.750	9.736	8.670	9.330	10.721	
8.822	11.348	11.059	10.844	11.410	12.125	13.861	
12.719	12.572	11.750	12.679	13.333	13.789	14.029	
7.250	6.750	10.291	9.000	11.331	11.375	10.250	
11.250	13.000	13.020	12.000	12.750	12.250	14.062	
14.354	14.354	13.500	16.312	15.104	14.500	7.250	
7.500	6.750	9.043	8.347	10.255	9.750	10.000	
10.500	11.500	11.750	11.500	15.710	13.043	12.516	
12.521	14.086	12.516	12.000	13.500	13.581	14.500	
14.500	14.000	7.941	8.500	8.500	10.897	10.617	
10.338	10.058	9.529					

TABLE A-1-4 ADJUSTED METHOD DATA FOR LEFT TURN ONE MANEUVER

GAP ACCEPTANCE VALUES WT. BY NO. OF REJ. FOR LEFT TURN IN ONE MANEU.									
11.788	10.865	11.540	11.000	11.250	10.822	12.337	12.215	11.750	12.426
12.771	11.670	13.000	13.596	12.795	13.500	14.301	14.750	13.848	13.986
43.750	12.428	8.750	9.500	9.000	9.583	10.476	9.750	10.250	9.750
12.857	11.250	12.500	14.880	12.750	16.000	13.750	6.000	6.666	7.500
7.000	7.649	7.749	7.200	8.500	8.008	8.524	8.250	9.041	9.816
10.133	9.500	9.500	9.558	9.000	9.625	9.500	10.400	9.750	11.108
11.250	11.500	12.500	12.000	12.000	2.141	13.250	13.000	14.575	13.250
14.300	14.208	5.500	5.500	5.000	6.206	5.948	6.500	7.172	6.439
7.000	7.374	7.500	7.500	7.330	7.362	8.793	8.250	8.750	9.775
9.000	9.330	10.500	10.000	10.517	10.889	11.042	10.000	10.686	10.333
11.000	13.448	12.500	12.715	12.000	12.000	12.206	12.000	12.620	12.000
11.871	11.670	13.000	13.224	15.206	14.000	14.500	14.241	14.482	14.000
4.500	4.551	4.000	6.000	6.000	7.500	8.425	8.500	7.839	7.750
8.463	9.356	10.034	9.413	9.827	10.741	10.336	11.000	12.896	11.460
12.000	13.241	13.000	13.500	13.000	13.000	13.000	13.636	14.000	14.000
14.482	14.000	14.659	6.710	6.330	7.052	6.851	8.046	8.000	8.000
9.018	8.831	9.061	10.154	9.500	9.500	8.930			

TABLE A-1-5 ' ADJUSTED METHOD DATA FOR LEFT TURN TWO MANEUVERS

GAP ACCEPTANCE VALUES WT. BY NO. OF REJ. FOR LEFT TURN IN TWO MANUV.									
10.500	10.000	9.969	10.578	10.000	10.736	10.327	11.000	10.830	10.750
11.248	11.750	13.424	13.293	15.225	12.860	11.933	14.783	14.609	
15.111	6.250	6.044	6.500	8.269	8.846	7.884	7.993	8.250	8.346
10.544	11.038	10.512	10.500	13.666	12.750	13.730	4.932	4.230	6.500
5.971	6.370	5.860	6.000	7.139	7.788	8.076	7.644	8.076	7.009
8.250	8.250	8.942	8.826	7.899	9.579	9.000	9.000	9.427	10.500
10.644	12.461	12.956	14.019	13.500	13.000	13.759	13.759	4.063	5.079
5.476	5.158	5.500	6.530	6.530	6.000	6.285	6.000	6.731	6.670
7.000	6.670	6.750	8.250	10.142	9.571	9.500	10.821	10.000	10.158
10.476	10.214	11.500	11.330	11.174	11.174	12.380	15.269	13.825	14.222
3.769	4.707	5.500	4.823	6.092	5.076	5.986	6.461	5.750	6.092
6.799	6.092	7.250	7.500	8.500	9.153	8.984	9.250	8.884	8.750
9.750	10.461	10.000	14.807	12.750	13.000	13.000	13.880	3.559	4.759
4.634	4.268	5.250	6.303	7.166	5.995	6.435	6.330	6.025	7.621
7.182	7.000	7.705	7.392	8.426	7.816	8.681	7.846	8.320	8.579
8.102	8.787	9.865	8.670						

TABLE A-2 RAFF METHOD DATA

GAP ACCEPTANCE DATA FOR RAFF METHOD (PASS. CARS ONLY)											
GAP RANGE	RIGHT		THROUGH ONE		THROUGH TWO		LEFT ONE		LEFT TWO		
	A	R	A	R	A	R	A	R	A	R	
<3.0	-	99	-	27	-	213	-	40	-	102	
3.0-4.0	3	31	-	11	1	52	1	16	-	30	
4.0-5.0	3	33	-	5	2	51	3	17	-	18	
5.0-6.0	10	20	-	7	9	38	4	6	8	17	
6.0-7.0	14	15	2	1	8	12	4	2	10	3	
7.0-8.0	9	9	3	-	11	11	9	2	2	-	
8.0-9.0	8	3	6	1	9	9	10	1	8	6	
9.0-10.0	14	3	6	1	15	2	14	-	5	1	
10.0-11.0	7	1	5	-	7	-	8	1	2	-	
11.0-12.0	10	-	8	-	8	-	12	-	4	-	
12.0-13.0	13	-	4	-	6	-	10	-	4	-	
13.0-14.0	22	-	15	-	40	-	20	-	4	-	
> 14.0	48	-	18	-	50	-	28	-	15	-	
TOTAL	161	215	67	53	166	388	123	86	62	177	

APPENDIX B

STPSN Modifications

The stop sign routine "STPSN" was rewritten to take into consideration the existence of median, and different gap acceptance distributions and conflicts developed by minor street vehicles. The flow chart of STPSN is shown in Figure B-1.

Two types of conflicts were detected in this subroutine and they are:

1. Cross traffic conflicts caused by minor road vehicles crossing or merging left with major road traffic. An arbitrary gap value of 5.00 second was assumed to be the minimum threshold below which conflicts occur. The reason for this assumption was: major road driver needed one second to react for the crossing vehicle plus at least 4 seconds to make him slow down from 55 mph to 30 mph at a deceleration rate of 9 feet per second square.
2. Shadow factor conflicts created at divided major highway intersections and caused by left turning vehicle stopped at left turn pocket lane. A diagram showing the shadow area is presented in Figure B-2. Closer look at the diagram reveals that the average distance of shadow area is a function of median width, median opening, and length of left turning vehicle. These three variables are input to the simulation model.

TSIG Modifications

This subroutine was modified to count the number of vehicles that stop or continue through the intersection during the amber phase of a

Figure B-1. STPSN Flow Chart

Figure B-1. Continued

Figure B-1. Continued

Figure B-1. Continued

FIGURE B-2. SHADOW FACTOR AREA DIAGRAM FOR DIVIDED MAJOR HIGHWAY INTERSECTION

fixed time signal control. This incident is detected whenever the signal turns amber at the major road approach. A random deceleration rate is assigned to the vehicle which approaches the intersection at this critical instant and the deceleration rate required to stop the vehicle safely is calculated. If the required rate is higher than the rate assigned to the driver, the vehicle will continue without stopping and the number of conflicts caused by going through the intersection during the amber phase is increased by one. If the required rate is smaller than the rate assigned to the driver, the vehicle will come to a complete stop and the number of the second type of traffic conflicts is increased by one.

TRVL Modifications

Since this subroutine represents the non-queued vehicle specified as moving along the link it occupies in response to traffic conditions in its immediate vicinity, it was decided to modify its logic for the purpose of detecting conflicts created on the major road and caused by minor road vehicles and count the number of vehicles that bypassed the stopping point at minor intersection approaches. Subroutine TRVL is called every 1 second and the location, speed, acceleration rate, deceleration rate, and the identification numbers of both leader and follower vehicles are updated. The distance between every vehicle and its leader is checked and, if this spacing is found to be 6 feet, the leader vehicle is a minor road vehicle which previously joined the major road traffic and the follower vehicle is a major road vehicle, the follower vehicle is moved to switch lanes to avoid collision with the leader. If the lane switching process succeeded, the number of switching

lane conflicts is increased by one. If the traffic condition in the adjacent lanes does not permit the follower vehicle to switch lane, the number of braking conflicts is increased by one. The modified part of the flow chart for this subroutine is shown in Figure B-3.

LFTRN Modifications

This subroutine was modified to detect the following conflicts:

(a) possible conflicts created by the shadow factor of left turning vehicles from the major divided highway to minor roads, (b) conflicts created by the shadow factor area effect for the case of undivided major highway, and (c) number of left turning conflicts.

The median width and median opening are fed to the simulation model to represent the geometrical design aspects of an intersection with divided major highway. The turning codes of vehicles are checked and if a vehicle is found to be a left turner, the opposing link is determined. A shadow factor conflict cannot happen unless two left turning vehicles are performing their maneuver simultaneously from a left turn pocket and one vehicle is blocking the vision of the other vehicle. Once this condition is detected, the boundaries of the shadow area are determined in the simulation model by means of the two lengths X and Y shown in Figure B-4. The location of each major road vehicle is checked and if one or more of these vehicles are detected within the shadow area, a shadow factor conflict counter is increased by one. The same logic is applied for the case of undivided major highway by making the median width equal to zero.

Left turning conflicts occur when a left turning vehicle accepts a small gap in the opposing traffic stream causing oncoming vehicles

Figure B-3. Modified Section of TRVL Subroutine

Figure B-3. Continued

Figure B-3. Continued

Figure B-3. Continued.

SHADOW FACTOR FOR DIVIDED HIGHWAY

SHADOW FACTOR FOR UNDIVIDED HIGHWAY

FIGURE B-4. DIAGRAM OF SHADOW FACTOR
CONFLICTS AT INTERSECTIONS

to slow down or brake. This critical gap size was investigated by numerous researchers and it was found, in general, that its value varied from driver to driver depending on driver type, speed of approaching vehicle and several other factors. It was assumed that the driver of the oncoming vehicle needs 1.5 second to percept and react for the existing stimulus, and 4.00 seconds to reduce the speed of his vehicle from 55 mph to 30 mph at a deceleration rate of 9 feet per second per second, therefore a total of 5.5 seconds was assumed to be the maximum value below which left turn conflicts occur.

CLRALL Modifications

The gap acceptance distributions developed from the field study were included in the model for the case in which the median is wide enough for a vehicle to stop without causing interruption to the through traffic on the major road. The average gap acceptance values mentioned in Chapter 2 for the distributions associated with the five different maneuvers together with the distribution of driver types in the embedded data of the simulation model were used to generate new decile statistical distributions for all possible maneuvers. The resulting distributions are shown in Table B-5.

Since the field study dealt only with divided major highway intersections, it was necessary to consult the literature to find gap acceptance distributions for the undivided case. Gap acceptance distributions for all possible maneuvers, developed by Wagner (5) for intersections without median on major roads, were utilized at this stage. The 50th percentile values for right turn, through, left turn

Table B-5. Gap Acceptance Distributions Used in the Simulation Model for the Case of Intersection with Median

Right & Left Turn Maneuver From Median LSLAG (10)	Through Maneuver In One Movement LSTHO (10)		Through Maneuver In Two Movements LSTHT (10)		Left Turn Maneuver In One Movement LSTLO (10)		Truck Maneuvers LSTRUK (10)	
	(1)=106*	(1)=115	(1)=106	(1)=93	(1)=115	(1)=115	(1)=115	(1)=115
"	(2)= 95	" (2)=104	" (2)= 95	" (2)=83	" (2)=110	" (2)=110	" (2)=110	" (2)=110
"	(3)= 87	" (3)= 96	" (3)= 87	" (3)=77	" (3)=101	" (3)=101	" (3)=101	" (3)=101
"	(4)= 79	" (4)= 87	" (4)= 80	" (4)=70	" (4)= 92	" (4)= 92	" (4)= 92	" (4)= 92
"	(5)= 73	" (5)= 81	" (5)= 74	" (5)=65	" (5)= 86	" (5)= 86	" (5)= 86	" (5)= 86
"	(6)= 70	" (6)= 77	" (6)= 70	" (6)=62	" (6)= 81	" (6)= 81	" (6)= 81	" (6)= 81
"	(7)= 64	" (7)= 71	" (7)= 65	" (7)=57	" (7)= 75	" (7)= 75	" (7)= 75	" (7)= 75
"	(8)= 57	" (8)= 62	" (8)= 57	" (8)=50	" (8)= 66	" (8)= 66	" (8)= 66	" (8)= 66
"	(9)= 49	" (9)= 54	" (9)= 49	" (9)=43	" (9)= 57	" (9)= 57	" (9)= 57	" (9)= 57
"	(10)= 38	" (10)= 42	" (10)= 38	" (10)=33	" (10)= 44	" (10)= 44	" (10)= 44	" (10)= 44

*Time lags are shown in units of tenth of a second.

maneuvers, and trucks were used to generate the necessary decile statistical distributions to be used in the computer model. The four distributions are presented in Table B-6.

SIGACT Modifications

This routine is called from the main program every one second to update all signals at a traffic actuated intersection. Detection of amber phase occurrence at major road approaches from this routine made the conflict counts at signalized intersections possible. The logic of the modification can be seen in the flow chart of Figure B-7.

Table B-6. Gap Acceptance Distributions Used in the Simulation Model for the Case of Intersection Without Median

Right Turn Maneuver <u>LSRT (10)</u>	Through Maneuver <u>LSTH (10)</u>	Left Turn Maneuver <u>LSLT (10)</u>	Truck Maneuver <u>LSTRK (10)</u>
LSRT (1)= 96*	LSTH (1)=107	LSLT (1)= 92	LSTRK (1)=115
" (2)= 86	" (2)= 95	" (2)= 82	" (2)=105
" (3)= 79	" (3)= 88	" (3)= 76	" (3)= 97
" (4)= 72	" (4)= 80	" (4)= 69	" (4)= 88
" (5)= 67	" (5)= 74	" (5)= 64	" (5)= 82
" (6)= 63	" (6)= 71	" (6)= 61	" (6)= 78
" (7)= 58	" (7)= 65	" (7)= 56	" (7)= 72
" (8)= 51	" (8)= 57	" (8)= 49	" (8)= 63
" (9)= 44	" (9)= 50	" (9)= 43	" (9)= 55
" (10)= 34	" (10)= 38	" (10)= 33	" (10)= 42

*Time lags are shown in units of tenth of a second.

Figure B-7. Modified Section of SIGACT Subroutine

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