JOINT HIGHWAY RESEARCH PROJECT

JHRP-74-8

THE EFFECT OF COMMERCIAL
VEHICLES ON INTERSECTION
CAPACITY AND DELAY

Thomas H. Yurysta

Digitized by the Internet Archive
in 2011 with funding from
LYRASIS members and Sloan Foundation; Indiana Department of Transportation

Final Report
THE EFFECT OF COMMERCIAL VEHICLES ON INTERSECTION
CAPACITY AND DELAY

by

Thomas H. Yurysta
Graduate Assistant in Research

Joint Highway Research Project

Project No.: C-36-17MM

File No.: 8-4-39

Prepared as Part of an Investigation

Conducted by

Joint Highway Research Project
Engineering Experiment Station
Purdue University

in cooperation with the

Indiana State Highway Commission

Purdue University
West Lafayette, Indiana
June 19, 1974

Final Report
THE EFFECT OF COMMERCIAL VEHICLES ON INTERSECTION
CAPACITY AND DELAY

TO: J. F. McLaughlin, Director June 19, 1974
Joint Highway Research Project
FROM: H. L. Michael, Associate Director Project: C-36-17MM
Joint Highway Research Project File: 8-4-39

Attached is a Final Report on a JHRP Research Study titled "The Effect of Commercial Vehicles on Intersection Capacity and Delay". The research was performed and the report written by Mr. Thomas H. Yurysta, Graduate Assistant in Research on our staff. The research was directed by Professor H. L. Michael.

The research reports findings relative to the equivalency value in passenger cars of a commercial vehicle at signalized intersections, the travel time delay caused by commercial vehicles at signalized intersections and the optimum corner radii to accommodate with minimum detrimental effects commercial vehicle turning movements.

The Report is presented as fulfillment of the objectives of the research. The results will be useful in the proper timing of traffic signals by providing important information as to equivalency factors for commercial vehicles and in the proper design of intersections relative to corner radii.

Respectfully submitted,

Harold L. Michael
Associate Director

HLM:ms

cc: W. L. Dolch	M. L. Hayes	C. F. Scholer
R. L. Eskew	C. W. Lovell	M. B. Scott
G. D. Gibson	G. W. Marks	J. A. Spooner
W. H. Goetz	R. F. Marsh	H. R. J. Walsh
M. J. Gutzwiller	R. D. Miles	E. J. Yoder
G. K. Hallock	G. T. Satterly	S. R. Yoder

ACKNOWLEDGEMENTS

The author expresses his gratitude to the Directors of the Joint Highway Research Project and to the Indiana State Highway Commission for enabling him to further his education through their financial support.

Acknowledgement in the highest regard is given to Professor Harold L. Michael, Head of Transportation and Urban Engineering, School of Civil Engineering, for his guidance in the selection of the study topic, for his constructive comments throughout the study, and for his critical review of the manuscript. Acknowledgement is also given to Dr. Virgil L. Anderson, Department of Statistics, for his critical review of the statistical analyses and review of the manuscript, and to Dr. Gilbert T. Satterly Jr., Professor of Transportation Engineering, for his review of the manuscript.

Thanks go to the author's colleagues and friends that rendered help during the course of the study. Special thanks are extended to Mr. Douglas Harwood and Mr. John Pavlovich for their consistent help in the data collection.

A thank-you goes to Mr. George Stafford of the School of Civil Engineering for his assistance in the data collection and to Mr. Dennis Fuddy of the Statistics Department for his critical comments of the statistical analyses.

Appreciation is due to Mrs. Martha Moll for her last minute typing of the manuscript.

No amount of appreciation can ever equal the motivation given by my parents to whom I am very grateful.

TABLE OF CONTENTS

	Page
LIST OF TABLES.....	vi
LIST OF FIGURES.....	vii
ABSTRACT.....	viii
INTRODUCTION.....	1
LITERATURE REVIEW.....	4
Passenger Car Equivalency of Trucks.....	6
Capacity Measurements.....	8
Commercial Vehicle Delay.....	10
Curb Radii at Intersections.....	11
PURPOSE.....	12
DATA COLLECTION.....	13
Selection of Signalized Intersections.....	13
Data Collection and Calculation of Equivalency Factors.....	13
Data Collection for Commercial Vehicle Delay.....	18
Data Collection for Right Turn Study.....	22
ANALYSIS OF DATA.....	24
Capacity Analysis.....	24
Statistical Analysis on Equivalency Factors.....	25
Regression Analysis on Equivalency Factors..	31
Comparing Headway Analysis to Capacity Analysis.....	42
Pilot Study for Commercial Vehicle Delay....	45
Commercial Vehicle Delay.....	47
Statistical Analysis on Commercial Vehicle Delay.....	48

TABLE OF CONTENTS (continued)

	Page
Developing A Model for Predicting Commercial Vehicle Delay.....	50
Improving the Model.....	53
Testing the Model.....	61
Right Turn Study.....	63
SUMMARY OF RESULTS.....	78
Capacity Analysis.....	78
Delay Analysis.....	80
Right Turn Analysis.....	82
CONCLUSION AND RECOMMENDATIONS	
General Conclusion.....	84
Recommendations.....	85
RECOMMENDATIONS FOR FURTHER RESEARCH.....	87
BIBLIOGRAPHY.....	89
APPENDICES.....	93
Appendix A.....	93
Appendix B.....	94
Appendix C.....	99
Appendix D.....	100
Appendix E.....	101

LIST OF TABLES

Table	Page
1. Equivalency Factors Found in the Different Studies.....	7
2. Equivalent Passenger Cars for Each Truck Type at a Signalized Intersection.....	26
3. Summary of Results from the 'W' Test.....	28
4. Range of Variables Affecting Truck Equivalency Factors.....	30
5. A Comparison of Truck Equivalency Factors Determined From the Headway Analysis and Capacity Analysis.....	43
6. Results From Commercial Vehicle Delay Pilot Study.....	46
7. Summary of Data for Commercial Vehicle Delay Study.....	49
8. Range of Variables Affecting Commercial Vehicle Delay.....	51
9. Significant Predictor Variables for Truck Delay.....	55
10. Summary of Data for Right Turn Study.....	65
11. Right Turn Vehicular Speeds for 30 to 50 foot Curb Radii.....	66

LIST OF FIGURES

Figure	Page
1. Growth of Passenger Car and Commercial Vehicle Registrations from 1952 to 1972...	5
2. Metropolitan Areas and Their 1970 Population.....	14
3. Frequency of Residuals for Truck Delay Model.....	57
4. Right Turn Speed of Passenger Cars as a Function of Curb Radius.....	67
5. Right Turn Speed of Single Unit Trucks as a Function of Curb Radius.....	62
6. Right Turn Speed of Truck Combinations as a Function of Curb Radius.....	69
7. Right Turn Speeds as a Function of Curb Radius.....	70
8. Right Turn Speed, in Miles Per Hour, for Passenger Cars.....	74.
9. Right Turn Speed, in Miles Per Hour, for Truck Combinations.....	78
10. Right Turn Speed, in Miles Per Hour, for Commercial Vehicles.....	77
Appendix	
Figure	
E1. Truck Delay as a Function of Speed Limit on Approach.....	101
E2. Truck Delay as a Function of Degree of Right Turn.....	102

ABSTRACT

Yurysta, Thomas H. M.S.C.E., Purdue University, May 1974.
The Effect of Commercial Vehicles on Intersection Capacity
and Delay. Major Professor: Harold L. Michael.

The general purpose of this research project was to study the effects commercial vehicles have on intersection capacity and delay. This general purpose was divided into the following three objectives:

1. Determine the equivalency factor of passenger cars to commercial vehicle at a signalized intersection, with respect to the type of commercial vehicle and the type of intersection approach.
2. Determine the travel time delay caused by commercial vehicles at signalized intersections.
3. Determine the effect of intersection corner radii on commercial vehicle turning movements.

Twenty intersection approaches were studied for equivalency factors of passenger cars to commercial vehicles. Equivalency factors were determined for a single unit truck, truck combination, and commercial vehicle, for a three-lane approach, two-lane approach, through and left turn lane, through lane, and through and right turn lane. Stepwise multiple regression analysis was employed to

develop regression equations that would predict commercial vehicle equivalency factors for any given intersection approach. Both regression equations that were developed explained over 90% of the variation in the original 16 commercial vehicle equivalency factors.

Twenty-three signalized intersection approaches were studied for delay caused by commercial vehicles. It was found that a passenger car's average running travel time through an intersection was increased from 39.8 seconds to 49.4 seconds, when one or more commercial vehicles were traveling ahead of it in the same platoon of vehicles. It was also found that the presence of trucks at a signalized intersection does not significantly increase or decrease the average stop time for a passenger car.

A regression model was developed to predict the average commercial vehicle delay at any given signalized intersection. This model explained 87.1% of the variation in the average commercial vehicle delay at the 23 approaches studied. The factors that produced a significant effect on increasing commercial vehicle delay were peak hour volume, percent of commercial vehicles, the presence of a left turn green phase, the presence of a right turn only lane, and the approach width. The factors that produced a significant effect on reducing delay were the presence of a left turn only lane, percent of right turns, the right turn curb radius, metropolitan area population, and the presence of curbing on the approach.

Nineteen corner radii were studied, and the relationships of passenger car, single unit truck, truck combination, and commercial vehicle right turn speeds to corner radii, curbing on approach, approach turning width, and cross street turning width were determined through regression analysis.

It was found that for a curb radii range of 30 to 50 feet, a 30 foot curb radius caused the least delay for a passenger car following a single unit truck, and a 50 foot curb radius caused the least delay for a passenger car following a truck combination. It was also found that from a 60 foot to 90 foot curb radius, the increase in speed for a single unit truck or truck combination was less than 0.5 miles per hour.

The presence of curbing at a signalized intersection approach was found to decrease the right turn speed of passenger cars by 0.7 miles per hour, to decrease the right turn speed of truck combinations by .9 miles per hour, and to have no effect on the right turn speed of single unit trucks.

INTRODUCTION

The control of vehicular traffic at an intersection is of critical importance to the Traffic Engineer. As this is the point in the traffic stream where travel time delays and accidents are at a maximum. The intersection must efficiently serve the demands of competing traffic streams during the peak traffic periods or else a bottleneck will develop that will result in excessive travel time delays to the driver and an increase in operating cost to the vehicular owner. Studies have been made to try to quantitize travel time delays in terms of dollars per unit of travel time (8, 11, 13, 16). These have found that the average operating cost for a passenger car amounts to 6 cents per minute and an average person values his time at least at 5 cents per vehicle-minute (see Appendix A for calculations). As an example, if 3 seconds of travel time could be saved for each vehicle using a heavily traveled intersection during each peak period, a total savings of \$11,440.00 would result during one year to the vehicle drivers and owners. (This calculation is based on an intersection peak period volume of 4,000 vehicles and 10 peak periods a week.)

It is easy to see that the intersection is the critical point in minimizing both travel time delays and accidents. O. K. Norman, to whom the 1965 edition of the Highway Capacity Manual is dedicated, states:

There is little doubt that the improvement of the efficiency of traffic movement at intersections is one of the more important, if not the most important, urban transportation problems (28).

Norman goes on to state:

There still remain for analysis several variables that have an extremely important effect on intersection capacities: These include right turns, left turns, commercial vehicles, ... (28).

With regards to commercial vehicles, in another article Norman and W. P. Walker state:

The presence of commercial vehicles tends to reduce intersection capacities in terms of the total number of vehicles because their acceleration rates are lower and they occupy more road space than passenger cars (29).

C. T. Kope states that trucks will never have the acceleration or gradeability to match passenger cars (23).

Kope suggests that the solution to congestion lies in upgrading the highway system and traffic control methods. This solution is especially applicable to intersection congestion where the optimal cycle times, phase splits, approach widths, and turning radii are critical to minimizing travel time delay and maximizing vehicular capacity.

Special consideration has to be given to commercial vehicles when selecting phase splits, approach widths, and turning radii at intersections. An equivalency factor of passenger cars to commercial vehicles is used to equate the two classes of vehicles and thus arrive at a single vehicular volume expressed in passenger car units. This procedure is desirable to compute phase splits for traffic signals (31). Approach widths are designed to meet optimum operational performance for a specified truck class; and curb radii at intersections are usually designed to accomodate turning movements of the longer trucks without encroachment on adjacent lanes or the curb (21).

Proper consideration of commercial vehicles in the design and operation of the intersection results in reducing travel time delay and increasing vehicular capacity.

LITERATURE REVIEW

"One factor that is now frequently considered as influencing the quality of traffic flow is the number of trucks or slow moving vehicles or both in the traffic stream (18)." As previously stated, trucks reduce intersection capacity and increase intersection delay.

Motor vehicle registration records indicate that the number of registered trucks has increased steadily in recent decades. However, from 1971 to 1972, there was an increase of 7.2% in registered trucks, as opposed to an increase of 4.5% in passenger cars (20). Figure 1 depicts the growth of registered trucks and passenger cars from 1952 to 1972 (3, 4, 20). Further analysis shows that the number of larger trucks being sold in the United States is increasing twice as fast as the total motor trucks and buses sold. From 1960 to 1970, sales of trucks with six wheels and three axles increased 310%, while sales of total motor trucks and buses increased 160% (3). Existing intersections are not only carrying higher traffic volumes but also are carrying a proportionally higher volume of trucks. This situation causes intersection capacity and delay to be of critical importance.

FIGURE 1. GROWTH OF PASSENGER CAR AND
COMMERCIAL VEHICLE REGISTRATIONS
FROM 1952 TO 1972

There has been a limited number of studies on the effect of trucks on intersection capacity and delay with most of these studies concentrating on intersection capacity.

Passenger Car Equivalency of Trucks

The vehicular capacity of an intersection is frequently specified in terms of passenger car units. This term is used so that an allowance for differences in the types of vehicles can be made. Since trucks have a greater interference on other traffic than passenger cars, an equivalency factor is used to equate trucks to passenger car units.

The first study on truck equivalency factors was made by O. K. Norman in 1949. He stated, "On the average, one commercial vehicle, not including those that stop to pick up or discharge passengers or goods, is equivalent in an intersection capacity sense to two passenger cars" (29). Today, the 1965 edition of the Highway Capacity Manual basically repeats this statement, "one truck can be considered as equivalent to a minimum of two passenger cars at intersections, even under the best conditions" (17).

Between 1949 and 1965 there were studies to arrive at a more exact truck equivalency factor. The results of these studies have not been consistent. The equivalency factors ranged from 1.3 to 1.75. Table 1 lists the equivalency factors that were found in the different studies.

TABLE 1
EQUIVALENCY FACTORS FOUND IN THE DIFFERENT STUDIES

Truck Equivalency Factor	Source
1.3	K. V. M. Koeford (22)
1.5 (10 foot approach)	Road Research Laboratory (33)
1.7 (12 foot approach)	
1.70	Sidney, Australia Study (25)
1.75	E. V. Webster and J. G. Wardrop (37)
1.75	R. Lane (24)
1.70 (short green phases)	Zurich, Switzerland Study (38)
2.0 (long green phases)	

No studies were found that determined commercial vehicle equivalency factors at intersections on a lane basis for the variable movements to be made. Also, few studies were found that subdivided commercial vehicles into more than one category.

The following equivalency factors were found for subdivided commercial vehicles:

F. V. Webster and J. G. Wardrop Study (37)

1.75 for 1 heavy or medium commercial vehicle

2.25 for 1 bus

2.5 for 1 tram

1.0 for 1 light goods vehicle

R. Lane Study (24)

1.0 for cars and vans

1.75 for commercial vehicles over 30 cwt

2.25 for buses and coaches

No studies were found that subdivided commercial vehicles into truck combinations and single unit trucks even though this is the most common subdivision of commercial vehicles.

Capacity Measurements

One measurement of capacity at an intersection is saturation flow. Saturation flow is the maximum number of vehicles that can be accommodated at an intersection approach given the characteristics unique to that intersection approach. H. J. W. Leong found that each 1% of commercial vehicles at an intersection approach reduces

saturation flow by 0.7% where the range of commercial vehicles is between 0 and 19% (25). The 1965 edition of the Highway Capacity Manual uses 1.00 as the intersection capacity adjustment for each percent of trucks and through buses in the intersection (17). The Highway Capacity Manual defines intersection capacity as "the maximum number of vehicles that can be accommodated given the particular geometrics, environment, and traffic characteristics and controls" (17). This definition of intersection capacity is similar to that of saturation flow.

Another measurement of capacity is headway or the interval in time between individual vehicles measured from a stationary point. One method of determining equivalency factors at intersections is by comparing headway times. The Road Research Laboratory used this method in their study entitled "The Passenger Car Unit Equivalent of a 'Heavy' Vehicle in Single Lane Flow at Traffic Signals" (33). B. R. Wildermuth studied headways at signalized intersections and found that each percent of heavy vehicles increased the average headway by about 0.7% for green intervals between 10 and 30 seconds, and for green intervals between 30 and 50 seconds, the increase caused by each percent of heavy vehicles was about 1% (38). These results on headways are similar to those on saturation flow previously mentioned.

Commercial Vehicle Delay

Another method of measuring intersection performance is vehicular delay. Intersection delay is the difference between the actual travel time through an intersection and the travel time through the intersection at normal roadway speed without deceleration, stopping, and acceleration. "Some investigators in the field of signalized intersections have recommended that average individual delay be used as an indicator of the level of service offered by a signalized intersection" (19). In a recent article entitled "Evaluation of Intersection-Delay Measurement Techniques", the authors conducted a literature review on the subject of intersection capacity and performance, and concluded that the majority of authors preferred delay as the most desirable and tangible measure of intersection performance (15).

The problem with using delay as a measure of intersection performance is the difficulty in the measurement of the actual delay. The only consistent methods of measurement have been those that record the delay to every vehicle using the intersection during the study period. These methods, however, are costly and require sophisticated equipment and/or large amounts of man-power.

No studies were found that specifically evaluate the delays caused by commercial vehicles at intersections. It is believed that the difficulty in measuring delay, as previously stated, has caused the lack of research in the area.

Curb Radii at Intersections

One of the largest single vehicular delays at an intersection may be caused by a long truck negotiating a right turn. Many intersections within urban areas are not able to accomodate turning movements of truck combinations without encroachment on adjacent lanes. Often, one large truck combination will delay a lane of traffic for an entire cycle because of its inability to negotiate a right turn without encroachment on the opposing and/or adjacent lane on the cross street.

It is of obvious critical importance, with respect to delay, that curb radii accomodate turning movements of truck combinations and buses without encroachment, especially where turning movements of these vehicles is large in number. A recent Institute of Traffic Engineers subcommittee report recommends 30 to 50 foot curb radii for trucks at intersecting streets on major streets carrying heavy traffic volumes (21). The American Association of State Highway and Transportation Officials also recommends 30 to 50 feet corner radii for most trucks and buses on arterial streets carrying heavy traffic volumes (1).

PURPOSE

The general purpose of this research project was to analyze the effects commercial vehicles have on intersection capacity and delay as an effort to improve traffic operations at signalized intersections. The Literature Review revealed voids and conflicts of opinions on the effects of commercial vehicles on intersection capacity and delay.

The specific purpose of this research was to attain the following three objectives:

1. Determine the equivalency factor of passenger cars to commercial vehicle at signalized intersections, with respect to the type of commercial vehicle and intersection approach.
2. Determine the travel time delay caused by commercial vehicles at signalized intersections; and then generate, through regression analysis, a general delay equation that will predict average travel time delay caused by commercial vehicles.
3. Determine the effect of intersection corner radii on commercial vehicle turning movements.

DATA COLLECTION

Selection of Signalized Intersections

Two general criteria for intersection selection were to select intersections from different size metropolitan areas and to select intersections that would give a wide percentage range of commercial vehicles. Specific criteria for selection were to limit the location of study intersections to fringe areas and outlying business districts of metropolitan areas. These two areas contain most of the commercial vehicle movements in a metropolitan area and exhibit intersections that have a wide percentage range of commercial vehicles. A final criteria of economy limited the location of metropolitan areas to within a reasonable travel time from Purdue University. Economy refers to conserving the limited resources of time and money. Figure 2 shows the metropolitan areas chosen and their respective location and population (9).

Data Collection and Calculation of Equivalency Factors

The effect of commercial vehicles on intersection capacity is greatest during the peak traffic periods. Approach volumes often reach the capacity of the intersection during

FIGURE 2. METROPOLITAN AREAS AND THEIR 1970 POPULATION

these periods, and the presence of commercial vehicles will have their greatest effect on intersection capacity.

The method used to determine the effect of commercial vehicles on intersection capacity was to study loaded signal phases during the peak periods. A loaded signal phase was defined as a green phase on an approach that had a continuous flow of vehicles during the entire phase, with no stoppages or long spacings between vehicles occurring at any time. This condition necessitates that vehicles are ready to enter the intersection when the signal turns green. From the study of loaded phases, the average number of vehicles in a loaded phase when commercial vehicles were present and when they were not present was determined. Subtracting these two averages and dividing by the average number of commercial vehicles gave the average number of passenger cars that were displaced by a commercial vehicle. (A commercial vehicle was defined as any vehicle that had at least 6 tires and two or more axles, including any passenger car towing a trailer.) For example, if a peak period had loaded phases on an approach with the following characteristics:

- a) average vehicles per loaded phase without commercial vehicles present of 10.0,
- b) average vehicles per loaded phase with commercial vehicles present of 8.0,

- c) and average number of commercial vehicles in a loaded phase that had commercial vehicles of 2.0;

the displacement factor would be $(10.0-8.0)/2.0$ or 1.0.

To get the equivalency factor of passenger cars to a commercial vehicle, 1 is added to the displacement factor.

In the above example, the equivalency factor would be 2.0.

A loaded phase represents the critical number of vehicles that can be accommodated with respect to the intersection and environmental conditions, traffic characteristics, and control measures. In this research, all factors affecting the amount and size of a loaded phase were held constant or measured and accounted for in the data analysis for each intersection studied. The following factors were held constant for each intersection studied:

1. Traffic signal at intersection
2. Location within metropolitan area
3. Clear weather.

The following factors were not held constant and thus were measured or calculated, for each intersection approach studied:

1. Approach volume per lane for each green phase
2. Width of each approach lane
3. Parking conditions
4. One-way or two-way street
5. Number of loaded phases per approach lane
6. Type of traffic signal
7. Metropolitan area population
8. Right turn volume for each green phase
9. Left turn volume for each green phase
10. Commercial vehicle volume for each green phase
11. Curb conditions
12. Curb radius
13. Speed limit on approach
14. Length of green phase
15. Degree of right turn at cross street
- 16.. Peak hour factor.

A copy of each of the data collection sheets is presented in Appendix B.

Vehicle types and their respective turning movements were counted for each intersection approach lane. After each green phase, the observer totaled the counts from the preceding green phase and indicated which lanes had had

a loaded green phase. Intersection approaches were studied for at least an hour during the peak traffic period or until a sufficient number of loaded phases had been counted. Usually this required 10 loaded phases without commercial vehicles and 10 loaded phases with commercial vehicles. In some cases, a larger number of loaded phases were counted because the number of vehicles in a loaded phase varied considerably, making a good statistical average nonevident.

Data Collection for Commercial Vehicle Delay

In this research, commercial vehicle delay is defined as the average travel time delay for each vehicle at an intersection, caused by the presence of commercial vehicles. Commercial vehicle delay is primarily caused from the slower deceleration and acceleration rates of commercial vehicles.

The first step in determining commercial vehicle delay was to define the roadway distance that is affected by the presence of a signalized intersection. This roadway distance originates at a point before an intersection where the average running speed on the roadway is reduced because of the presence of the intersection. The distance terminates after the intersection at a point where the average running speed on the roadway is continued. For this research, the average running speed for roadways with a design speed of 30 miles per hour was used as the

criterion for average running speed. This design speed is typical on most roadways located in fringe areas and outlying business districts of metropolitan areas. Table II-6 in A Policy of Geometric Design of Rural Highways, 1965 edition, indicates an average running speed of 25 miles per hour for a roadway with a design speed of 30 miles per hour, operating under traffic conditions approaching possible capacity (2). Figure 2.14 in the Traffic Engineering Handbook indicates a deceleration length of 500 feet is required for an average running speed of 25 miles per hour (5). Assuming an average maximum queue length of 250 feet at an intersection approach during a peak period, a vehicle will start decelerating at a distance of 750 feet before the intersection (500 feet + 250 feet). This distance represents the average maximum distance from the intersection at which a driver will begin decelerating. The average maximum distance required to accelerate to 25 miles per hour from a stop is determined by a heavy commercial vehicle. It has been determined that a mean semi-trailer requires 500 feet to accelerate to an average running speed of 25 miles per hour (14). Thus, to determine the commercial vehicle delay, vehicular movements were studied from 750 feet before an intersection to 500 feet after the intersection.

The present practical methods of determining general intersection delay from field measurements are the

Sagi-Campbell Method (10), the Berry-Van Til Sampling Method (6), Time-Lapse Photography Method (36), Traffic Flow Meter Method (12), and a Floating Car Method. The Sagi-Campbell Method, Berry-Van Til Sampling Method, and Traffic Flow Meter Method are not practically applicable to measuring commercial vehicle delay. The Time-Lapse Photography Method is the best method available for measuring commercial vehicle delay; however, for reasons of economy and time this method was not chosen. The Floating Car Method is the second best method for measuring commercial vehicle delay. This method was chosen because it best fit the conditions of economy and time.

The Floating Car Method requires a test car to repeatedly, randomly enter a platoon of vehicles approaching an intersection and to remain within the platoon until a point beyond the intersection, where the average running speed of the roadway is again reached. Each run is timed from a set of reference points, and the number of runs required is statistically determined. In this research project, the reference points used were stationary points located 750 feet before the intersection and 500 feet after the intersection. Besides timing each run, the following factors affecting commercial vehicle delay were determined by the observer in the test car:

1. Position of test car in the platoon of vehicles
2. Approach lane occupied by the test car

3. Number of commercial vehicles ahead of the test car in the platoon of vehicles
4. Stop time caused by the red signal (if applicable)
5. Pedestrian delay time (if applicable)
6. Delay caused by a slow right-turning vehicle (if applicable)

The following additional factors were counted by a stationary observer:

1. Approach volume per lane
2. Number of loaded phases
3. Right turn volume
4. Left turn volume
5. Commercial vehicle volume.

The following measurements were made at each intersection approach studied:

1. Approach width
2. Parking conditions
3. Number of approach lanes
4. One-way or two-way street
5. Metropolitan area population
6. Curb parking on approach
7. Type of traffic signal
8. Curb radius
9. Speed limit on approach
10. Degree of right turn at cross street
11. Length of green phase

12. Curbing on approach

13. Exclusive turning lanes.

These factors that were counted and measured were factors that might affect commercial vehicle delay.

Data Collection for Right Turn Study

The best method of increasing capacity and reducing delay from right turn movements at intersections is to increase the curb or corner radius.

Vehicular speed was chosen as the criterion for measuring the effects of curb radius on capacity and delay from right turn movements. In this research, vehicles were timed as they negotiated right turns at several signalized intersection approaches. The approaches were chosen such that a wide range of curb radii was investigated. Vehicles were subdivided into passenger cars, single unit trucks and through buses, and truck combinations. Times were repeatedly taken for each vehicle sub-class at each approach studied, until a good statistical average was obtained.

The same measuring distance was used for each curb radius studied. This distance was based on the largest, probable curb radius that would be studied. This gave assurance that the full turning maneuver would be timed. Since the intersection locations would be limited to fringe areas and outlying business districts, a 60 foot radius was assumed to be the maximum that would be encountered, and was thus chosen as the measuring distance.

Criteria for timing a vehicle was that the vehicle be moving before entering the beginning reference point. The beginning reference point was established by measuring 60 feet along the front tangent of the curve from an initial measuring point located at the point of intersection of the curve tangents. The vehicle was also required to be moving in free flow the entire timing distance. This meant that the vehicle's turning maneuver could not be influenced by the movements of other vehicles in front or in back of it. The timing distance ended at the end reference point, which was established by measuring 60 feet along the back tangent from the point of intersection of the curve tangents. The last criteria was that curb radii from only right angle intersections be studied.

ANALYSIS OF DATA

Capacity Analysis

The Capacity Analysis was performed at 20 intersection approaches. Fifteen of these approaches are located in Indianapolis; three are located in East Chicago; one is in Hammond; and one is in Lafayette.

Truck equivalency factors were determined on an approach and on a lane basis. The equivalency factors were determined for a two-lane approach with through and right turning movements on the right lane and through and left turning movements on the left lane; and for a three-lane approach with through and right turning movements on the right lane, through movements on the center lane, and left turning movements on the left lane. Equivalency factors were also determined for an approach lane with through and right turning movements; for an approach lane with through movements; and for an approach lane with through and left turning movements.

The data that was used for determining truck equivalency factors for all commercial vehicles was then reanalyzed for loaded phases that contained only single unit trucks, through buses, and passenger cars; and for loaded phases

that contained only truck combinations and passenger cars. A sufficient number of intersection approaches had data for the above types of loaded phases so that equivalency factors of passenger cars to single unit trucks and of passenger cars to truck combinations were determined. A summary list of the equivalency factors that were determined is presented in Table 2.

Statistical Analysis on Equivalency Factors

The first statistical analysis performed on the equivalency factors was to determine the average of each type of equivalency factor. These averages are presented in Table 2. They are the sample means and not the true mean or population mean. It is necessary to determine the amount of error and a corresponding level of confidence that occurs when using the sample means to represent the population means. This amount of error can be determined from the following equation:

$$\text{error} = \frac{t_{\alpha/2} s}{\sqrt{n}}$$

Where

error = error in using the sample mean as an estimate of the population mean.

$t_{\alpha/2}$ = the value of the 't' distribution leaving an area of $\alpha/2$ to the right with degree of freedom equal to the sample size minus one.

TABLE 2. EQUIVALENT PASSENGER CARS FOR EACH TRUCK TYPE AT A SIGNALIZED INTERSECTION

APPROACH TYPE	TRUCK TYPE		
	COMMERCIAL VEHICLE	SINGLE UNIT OR BUS	TRUCK COMBINATION
3-LANE APPROACH	1.85 (.15)*	1.71 (.21)	2.37 (.19)
2-LANE APPROACH	1.86 (.13)	1.74 (.26)	2.38 (.19)
THROUGH & LEFT LANE	2.22 (.19)	2.30 (.38)	2.29 (.16)
THROUGH LANE	1.92 (.20)	1.61 (.25)	2.24 (.24)
THROUGH & RIGHT LANE	1.74 (.22)	1.57 (.32)	2.31 (.34)

* VALUES IN PARENTHESES INDICATE THE STANDARD ERROR OF THE MEAN

s = standard deviation of the sample.

n = number of equivalency factors in sample =
sample size (27).

The following assumptions are needed in order to use the above equation:

1. The sample size is less than or equal to 30.
2. The population variance is unknown.
3. The population is approximately bell-shaped.

The first two assumptions are given conditions, and the last assumption has to be proven by performing a test.

One of the ways to test for a bell-shaped approximation or a normal distribution is to perform a normality test on the sample data. The normality test chosen for this research was the 'W' test developed by S. S. Shapiro and M. B. Wilk in 1965 (34). This test does not involve using the mean and variance as part of the hypothesis, which is common in other tests for normality. In 1968 Shapiro, Wilk, and Chen demonstrated that the 'W' test was generally superior in detecting normality over sample sizes ranging from 10 to 50.

A summary of the results from the 'W' test is shown in Table 3. No sample was found to have a W value less than the critical value at the alpha level of .05 or 5%. (The alpha level indicates the probability of rejecting a hypothesis which is actually true.) Thus, the hypothesis of normality was not rejected for any given sample.

TABLE 3

SUMMARY OF RESULTS FROM THE 'W' TEST

<u>Equivalency Factor</u>	<u>N*</u>	<u>W Calculated</u>	<u>W Crit.**</u>	<u>Normality</u>
trucks on a 3-lane a approach	7	.920	.803	accept
trucks on a 2-lane approach	9	.883	.829	accept
trucks on a left lane	8	.913	.818	accept
trucks on a center lane	10	.910	.842	accept
trucks on a right lane	14	.978	.874	accept
single units on a 3-lane approach	7	.903	.803	accept
single units on a 2-lane approach	7	.957	.803	accept
single units on a left lane	7	.957	.803	accept
single units on a center lane	6	.847	.788	accept
single units on a right lane	12	.935	.859	accept
truck combs. on a 3-lane approach	6	.876	.788	accept
truck combs. on a 2-lane approach	6	.973	.788	accept
truck combs. on a left lane	4	.897	.748	accept
truck combs. on a center lane	6	.941	.788	accept
truck combs. on a right lane	8	.896	.818	accept

*N is sample size

**W critical is based on an alpha level of .05

The next step was to calculate the error in using the sample means as estimates of the population means. A level of confidence of 95% or an alpha level of 5% was used in the calculations. The largest error calculated for all commercial vehicles was .15, and the smallest error calculated was .10. These errors are considered reasonable; and thus, the sample means for the truck equivalency factors, as presented in Table 2 under commercial vehicle, are recommended to be used within the variable range indicated in Table 4.

The largest error in using the sample means as estimates of population means in single unit truck equivalency factors was calculated to be .38. This error was considered to be excessive. The single unit equivalency factor for an approach with through and left turning movements, as shown in Table 2, therefore, should not be used for application. With respect to sample mean truck combination equivalency factors, the only sample mean with an excessive error was for an approach lane with through and left turning movements where it was .29. The sample means for single unit trucks and truck combinations, as presented in Table 2, are recommended to be used with the above exceptions. These sample means should only be applied to those intersection approaches that have variables within the ranges indicated in Table 4.

TABLE 4
RANGE OF VARIABLES AFFECTING TRUCK EQUIVALENCY FACTORS

<u>Variable Number</u>	<u>Variable</u>	<u>Unit of Measurement</u>	<u>Range</u>
X1	Peak Hour Load Factor	No. of Loaded Phases/Total Phases	.20 to .97
X2	Approach Width	Feet	20 to 38
X3	Green Phase Length	Seconds	21. to 42.
X4	Peak Hour Approach Volume	Vehicles per Hour	581 to 1505
X5	Percent Truck Combinations	Percent of Peak Hour Volume	0.3 to 12.3
X6	Right Turn Curb Radius	Feet	10 to 100
X7	Peak Hour Factor	Vehicles per Peak Hour	.66 to .95
X8	Curbing on Approach	4xVeh. per Peak 15 min.	Yes & No
X9	Percent of Trucks	Yes or No	1.5 to 19.3
X10	Percent Right Turns	Percent of Peak Hour Volume	0.1 to 42.1
X11	Degree of Right Turn	Percent of Peak Hour Volume	45 to 135
X12	Metropolitan Area - Population	Degrees	109,378 to 1,110,000
X13	Parking on Approach	People	Yes & No
X14	Percent Left Turns	Yes or No	0 to 47.8
X15	Percent Single Units	Percent of Peak Hour Volume	1.1 to 8.3
X16	Speed Limit on Approach	Miles per Hour	25 to 35

Regression Analysis on Equivalency Factors

A further application of the truck equivalency factors was to devise a method of obtaining a truck equivalency factor for any given intersection approach. This method would be especially useful when a more accurate factor is desired than the mean factor as determined from other intersections. An example of the use of an equivalency factor of this accuracy would be in timing of a fixed-time traffic signal.

The method chosen to develop a truck equivalency factor, unique to any given intersection approach, was multiple regression analysis. Regression analysis reports which variables have the greatest effect upon the dependent variable. Multiple regression analysis is especially valuable when many predictor or independent variables are known and it is necessary to eliminate those that have very little predicting power. The two purposes for employing multiple regression analysis in this research were:

1. To develop equations that will predict the truck equivalency factor at any given intersection approach based upon the physical, traffic, and environmental conditions unique to that approach.
2. To determine the significance of each variable employed in the multiple regression analysis.

An available computer program was used to perform the regression analysis. The computer package program entitled

"SPSS 15: Regression" was acquired through the Purdue University Statistical Library Program (35). The particular multiple regression process used was stepwise linear regression. The stepwise linear regression program will enter predictor variables one at a time to the regression equation, in order of highest partial correlation with the dependent variable. This program will continue adding predictor variables until the list is exhausted or until there remains no significant variables. The computer determines the significance of a particular variable by evaluation of its F-ratio and tolerance level (7).

Data from sixteen intersection approaches was used in the stepwise regression program. Ten of these were two-lane intersection approaches, and six were three-lane intersection approaches. Table 4 presents a list of all variables that were measured at each of the 16 approaches. It was assumed that each of these variables could have some predicting power on the dependent variable. The variable parking on approach was deleted from the program because of the lack of a suitable range of 'yes' and 'no' conditions. This left 14 variables to explain the variation about the mean of the values of the dependent variable. The variable curbing was measured as either 'yes' or 'no', depending on whether curbing existed or not for a particular approach. A dummy variable was used to transform this qualitative variable into a quantitative variable. A value of 00 was assigned for a 'no' measure, and a value of 01 was assigned for a 'yes' measure.

The stepwise linear regression program was initially run on the original 14 predictor variables. The results yielded an R^2 value of .8449 and a F value of 1.36. R^2 is a common name for the multiple correlation coefficient. This multiple correlation coefficient ranges in value from 0 to 1; and the closer the value is to 1, the higher the correlation is between the predictor variables and the dependent variable. The F value is compared against the critical F value in order to determine the significance of the regression. The significance of the regression increases with increases in the F value.

The R^2 value of .8449 indicates that the predictor variables explain 84.49% of the variation about the mean of the given values for the dependent variable, truck equivalency factor. This value is acceptable. However, the F value of 1.36 is below the critical F value of 8.74 for a 5% significance level. This indicates that the regression equation is not significant.

The best method to increase the significance of the regression equation was to eliminate those predictor variables from the regression equation that display small R^2 values. A partial F test was conducted after each step to determine if the addition of that predictor variable to the regression equation resulted in a significant increase in the R^2 value. The partial F values were computed by dividing the mean square of the predictor variable by the

mean square of the residual (30). These partial F values were then compared to a critical value of F based on an alpha level of .50. If a predictor variable was found to have a nonsignificant F value, it was removed and the regression equation was recomputed. The following variables were removed from the regression equation because of nonsignificant partial F values:

1. Percent of left turns
2. Percent of single unit trucks
3. Metropolitan area population
4. Percent of right turns
5. Speed limit on approach
6. Peak hour factor
7. Curbing on approach
8. Peak hour approach volume
9. Right turn curb radius

The five predictor variables that remained in the regression equation produced a R^2 value of .3024 and a F value of 8.12. This F value of 8.12 is well above the critical value of 3.33.

The next step was to consider any interactions that might improve the predicting power of the regression equation. From using the correlation matrix in the stepwise linear regression program previously printed and the investigator's judgement as to which interactions have a

practical 'sense', the following interactions were computed:

1. Percent of truck combinations x right turn curb radius
2. Speed limit on approach x degree of right turn
3. Speed limit on approach x percent of right turns
4. Peak hour approach volume x green phase length

These four interaction variables were added to the original 14 predictor variables, and the regression equation was recomputed. Partial F tests eliminated 11 of the 18 variables. The resulting R^2 value was .9578; and the F value was 19.87, which was well above the critical value of 3.73. This R^2 increased .1554 from the original value of .8024. The final regression equation is as follows:

$$Y = -.907 + .362X_1 + .0199X_2 + .0837X_3 - .0000675X_3X_4 \\ - .00183X_5 + .000340X_5X_6 + .00251X_4 - 1.159X_7$$

Where

Y = Truck equivalency factor in passengers cars per commercial vehicle

X_1 = load factor in hundredths

X_2 = approach width in feet

X_3 = green phase length in seconds

X_4 = peak hour approach volume in vehicles

X_5 = percent of truck combinations

X_6 = right turn curb radius in feet

X_7 = peak hour factor in hundredths

and the estimate of the standard error = .041.

An additional investigation into the practicality of the regression equation as a predictor of truck equivalency factors involved an examination of the residuals. The residuals are the difference between the actual truck equivalency factors found at the study intersections and the truck equivalency factors predicted by the regression equation. Regression analysis was employed under the assumption that these residuals are normally distributed (30). To check this assumption the W test was performed on the residual values. The calculated W value of .932 was found to be greater than the critical value of .837 for an alpha level of .05. Thus, the hypothesis of normality was not rejected.

Two additional decisions were made at this stage. Both of these decisions involved deleting predictor variables because of the difficulty in measuring them. The first decision was to develop a second regression equation that eliminated the variables percent of single unit trucks and percent of truck combinations and that used the variable percent of trucks (or commercial vehicles). Since field data of intersection volumes usually does not subdivide trucks into single units and truck combinations, the variable percent of trucks would be more practical in determining truck equivalency factors from existing field data. The second decision was to eliminate the variable load factor. Field data of intersection approaches usually does not include load factors of the approaches. Thus, in an

effort to make the regression equation more applicable to existing field data, this variable was also deleted.

Based on these two additional decisions, a second regression equation was developed. The predictor variables load factor, percent of truck combinations, and percent of single unit trucks were deleted and the predictor variable percent of trucks was added to the regression program. The same interactions were inserted, with the exception of percent of truck combinations x right turn curb radius. This interaction was substituted by the interaction percent of trucks x right turn curb radius.

Partial F tests removed 8 of the original 16 variables from the second regression equation. The resulting R^2 value was .9054 and the F value of 6.38 was above the critical value of 4.10. The final regression equation is as follows:

$$Y = -1.245 + .0934X_3 + .0112X_2 + .00189X_4 - .238X_8 \\ + .0000085X_9X_6 - .0000626X_3X_4 + .00845X_6 \\ - .00232X_{10} - .00949X_9$$

Where

Y = truck equivalency factor in passenger cars per commercial vehicle

X_2 = approach width in feet

X_3 = green phase length in seconds

X_4 = peak hour approach volume

and the estimate of the standard error = .066.

X_6 = right turn curb radius in feet

X_8 = curbing on approach where yes = 1 and no = 0.

X_9 = percent of trucks

X_{10} = percent of right turns

The W test was performed on the residual values and yielded a calculated W of .954, which is greater than the critical W of .887 for an alpha level of .05. Thus, the hypothesis of normality was not rejected.

An examination of the arithmetic sign preceding each regression coefficient in the regression equation is informative and reveals the practicality of the regression equation. However, one must be careful not to examine the logic or interpretation of the arithmetic sign preceding each predictor variable without due consideration of the effect that the particular predictor variable has on the other predictor variables. The positive or negative contribution of each predictor variable in the regression equation must not be interpreted as an isolated contribution but as a contribution that effects both the dependent variable and the other predictor variables.

The arithmetic signs preceding each regression coefficient in the above regression equation were interpreted as follows:

1. The positive sign preceding green phase length indicates that a longer green phase length will cause an increase in the truck equivalency factor. Longer green phases are associated with higher

volume intersection approaches. Higher volume approaches result in more vehicles being effected by the slower performance characteristics of a truck.

2. The positive sign preceding approach width indicates that an increase in pavement width causes an increase in the truck equivalency factor. Large pavement widths are usually accompanied by higher speed limits. These higher speed limits increase the effect of trucks on passenger cars because of the proportionally slower time it takes trucks to accelerate to a higher speed than passenger cars.
3. The positive sign preceding peak hour volume indicates that an increase in peak hour approach volume results in an increase in the truck equivalency factor. As previously stated, a higher approach volume produces more vehicles to be effected by the presence of a truck, thus increasing the truck equivalency factor.
4. The negative sign preceding curbing on approach indicates that the presence of curbing causes a decrease in the truck equivalency factor. Curbing along a roadway will cause a decrease in vehicular speeds for the following reasons:

1. The presence of curbing along a roadway usually indicates commercial or residential development along the abutting property. Increased development requires a large number of access driveways which causes speed to be reduced along the roadway.
2. The presence of curbing acts as an obstruction to the driver resulting in slower speeds and often causing lateral placement of the moving vehicles.

Therefore, the reduction in vehicular speed caused by the presence of curbing will tend to offset or decrease the effect of trucks on passenger cars.

5. The positive sign preceding curb radius indicates that an increase in curb radius causes an increase in the truck equivalency factor. This appears to be a false statement with respect to a practical interpretation and the investigators have no obvious explanation. Right turn curb radius also appears in **an interaction variable** in the regression equation, and it is thought that this double appearance causes the lack of a practical interpretation.
6. The negative sign preceding percent of right turns indicates that the truck equivalency factor decreases with an increase in right turns. Vehicles

turning right do not travel as fast as through vehicles; and consequently, a through vehicle will incur a delay from a right-turning vehicle preceding it. This causes a reduction in speed for the through vehicle, and as previously stated, an existing reduction in vehicular speed will offset or decrease the effect of trucks on passenger cars.

7. The negative sign preceding percent of trucks indicates that an increase in trucks reduces the truck equivalency factor. Trucks often appear at intersection approaches in groups, spaced one behind the other. When this occurs the slowest truck will govern the speed of the platoon of vehicles, and the other trucks will follow the slowest truck with a speed that is similar to that of a passenger car following the slower truck. Thus, only the slowest truck has an effect on the platoon of vehicles, and this effect will be proportioned out to the other trucks. This results in an actual decrease in the effect of trucks on passenger cars.

The arithmetic signs preceding the interaction variables are dependent upon the magnitude and effect of each variable in the interaction. It is very difficult to determine how the magnitude and effect of each variable in the

interaction affects the preceding arithmetic sign; and thus, the arithmetic signs preceding interaction variables were not examined.

Comparing Headway Analysis to Capacity Analysis

Equivalency factors were determined from the headway analysis and the capacity analysis at five intersection approaches. Truck equivalency factors were determined from the headway analysis by dividing the average headway time for two successive trucks by the average headway time for two successive passenger cars. Table 5 presents the truck equivalency factors determined from the two analysis procedures. The average equivalency factor from the capacity analysis was 1.88, and the average factor from the headway analysis was 2.01. A paired observations test was performed to determine if the two averages were statistically different (30). Before this test was performed it was necessary to test the two sets of equivalency factors for normality. The W test was again employed; and the calculated W values were .787 for the capacity observations and .880 for the headway observations. Both of these values were larger than the critical W value of .762; and thus, the hypothesis of normality was not rejected.

TABLE 5
A COMPARISON OF TRUCK EQUIVALENCY FACTORS DETERMINED FROM
THE HEADWAY ANALYSIS AND CAPACITY ANALYSIS

<u>Headway Analysis</u>	<u>Capacity Analysis</u>
2.11 (passenger car units)	2.00 (passenger car units)
2.01	1.98
2.19	1.96
2.05	1.86
1.71	1.59

The mathematical equation for the paired observations test is as follows:

$$t = \bar{D} / s_{\bar{D}},$$

where the hypothesis of the means being equal is rejected if t is greater than or equal to $t(1 - \alpha)$ (Degrees of freedom), and, where $\bar{D}$ is the average difference between the two analysis procedures and $s_{\bar{D}}$ is the sample standard deviation of the differences (30). The calculated t value was 1.757. This t value falls outside of the critical range of t greater than or equal to 2.132 for an alpha level of .05 and for 4 degrees of freedom. However, before the hypothesis of both means being equal is accepted, the Type II error or the beta value is determined. The Type II error represents the acceptance of a hypothesis which is false. From Appendix 10 in Statistics in Research (30),

the beta value of .35 is found for the given conditions of the problem. This indicates that if the hypothesis of equality is accepted, there is a 35% chance that is actually false. This beta value is too large to permit acceptance of the hypothesis that the two means are equal. If the alpha level is increased to .10, the calculated value of t of 1.757 will fall inside the new critical region of t greater than or equal to 1.533. In this case, the hypothesis of equality is rejected with a 10% chance of the hypothesis actually being true. This is a smaller risk than accepting the hypothesis with a 35% chance of it actually being false. Therefore, the hypothesis that the two means are equal is rejected, and it is concluded that the mean equivalency factor obtained from the headway analysis is significantly different from the mean equivalency factor obtained from the capacity analysis.

This analysis and the conditions of data collection resulted in the conclusion that using a headway analysis to determine truck or commercial vehicle equivalency factors at intersection approaches is unreliable. Headways of two successive trucks or passenger cars are constantly changing through an intersection. This research used a headway measured at the center of the intersection to represent an average headway throughout the intersection. It is doubtful if headway measured at any one point will provide reliable results.

Pilot Study for Commercial Vehicle Delay

The commercial vehicle delay was determined by subtracting the average travel time through the intersection when commercial vehicles were present from the average travel time through the intersection when commercial vehicles were not present. The average travel time was composed of the running travel time and the stop travel time. The stop travel time in this research is the time from when a vehicle stops because of a red traffic signal to the time the traffic signal turns green. The running travel time includes the running time of a vehicle through an intersection and the time that is incurred from the delay of a vehicle to start up once the traffic signal turns green. In this research, travel time means running travel time only, with stop travel time not being included. The stop travel time was analyzed separately from the travel time.

The number of vehicle runs needed to determine an average travel time was determined statistically from a pilot study. The pilot study was performed at three intersections. The results from the pilot study are listed in Table 6.

TABLE 6

RESULTS FROM COMMERCIAL VEHICLE DELAY PILOT STUDY

<u>Mean Travel Time</u>		<u>Number of Runs</u>		<u>Standard Deviation</u>		<u>Truck</u>
<u>Trucks</u>	<u>No Trucks</u>	<u>Trucks</u>	<u>No Trucks</u>	<u>Trucks</u>	<u>No Trucks</u>	<u>Delay</u>
<u>sec.</u>	<u>sec.</u>					<u>sec.</u>
48.7	39.6	7	7	10.9	8.4	9.1
41.2	30.8	10	10	6.6	6.1	10.4
40.3	32.1	10	10	8.9	9.3	8.2

The method for determining the number of runs or sample size required uses the following equation:

$$n'_0 = (z_a^2 / r^2) / (1 + z_a^2 / r^2 N)$$

Where

n'_0 = required value of the sample size

z_a = 100a percent point of the standard normal distribution

$a = (1 + \lambda) / 2$, where λ is the given probability,

$\lambda = .90$

r = error tolerance/standard deviation

N = population = vehicles per peak period (26)

In order to use the above equation, an estimate of the standard deviation and vehicles per peak period is needed. An acceptable error tolerance with an accompanying probability level of not exceeding the error tolerance is also needed. In this research, the standard deviation and vehicles per peak period estimates were determined by averaging

the data from the pilot study. The acceptable error tolerance was determined as one-half the average truck delay calculated from the pilot study data. This tolerance was chosen because it assured that the total error possible from both average travel times would never exceed the truck delay, and thus would not cause a negative truck delay. From the pilot study, the average truck delay was found to be 9.2 seconds per vehicle. The acceptable tolerance error was then $9.2 / 2$, or 4.6 seconds. If this error is rounded down to 4 seconds, it can be comfortably assured of always having a truck delay greater than zero. The probability level of not exceeding the error tolerance was set at 90%.

By substituting the above values into the equation for determining sample size, a value of 11.4 is calculated for n'_0 . Thus, a minimum of 11 travel time runs is required to be 90% assured of an estimate within 4 seconds of the true mean travel time. This requires 22 travel time runs at each intersection approach or 11 runs for each mean travel time, with trucks and without trucks.

Commercial Vehicle Delay

Twenty-three intersection approaches were analyzed for commercial vehicle delay. Fifteen of these were located in Indianapolis, five in Lafayette, one in Kokomo, one in Hammond, and one in East Chicago.

Table 7 presents the average travel times, average stop times, and commercial vehicle or truck delay. Truck delay was found at twenty-two of the twenty-three intersection approaches. At one approach, the presence of trucks decreased the average travel time by 1.2 seconds. The average truck delay for all the intersection approaches was 9.2 seconds. The reason for the decreased average travel time when trucks were present at the one approach was caused by the absence of truck combinations and large single unit trucks. The only trucks present at the approach were small single unit trucks with performance characteristics similar to those of passenger cars.

Statistical Analysis on Commercial Vehicle Delay

The Wilcoxon Signed Rank Test was performed on the twenty-three average truck delay times to determine if a significant truck delay existed (30). This test is widely acceptable when the data occur in pairs; and the investigator wishes to compare the effects of two treatments, with one treatment associated with one member of the pair and the other treatment associated with the second member of the pair. The calculations for this test are presented in Appendix C. The calculated T value for this test was -1, and the corresponding critical T value was 62 at a level of significance of .01. Since T calculated is less than T critical, the hypothesis of no difference between the

TABLE 7

SUMMARY OF DATA FOR COMMERCIAL VEHICLE DELAY STUDY

<u>Intersection</u>	<u>City</u>	<u>Approach</u>	<u>Trucks Present</u>		<u>Trucks Not Present</u>		<u>Truck Delay</u>
			<u>Travel Time</u>	<u>Stop Time</u>	<u>Travel Time</u>	<u>Stop Time</u>	
Kentucky at Morris	Indianapolis	Northeast	49.5 sec	10.0 sec	42.2 sec	10.3 sec	5.8 sec
Kentucky at West	Indianapolis	Southwest	57.3	23.5	41.7	15.6	15.6
Kentucky at Harding	Indianapolis	Northeast	49.4	19.4	38.4	9.7	11.0
Morris at Tibbs	Indianapolis	East	44.4	11.7	32.6	10.7	11.8
Morris at Harding	Indianapolis	East	72.9	5.1	64.7	5.2	8.2
Morris at Tibbs	Indianapolis	West	40.7	12.3	33.2	6.6	7.5
Morris at Harding	Indianapolis	East	58.7	14.5	46.4	13.8	12.3
Virginia at Stevens	Indianapolis	Northwest	35.8	1.4	28.9	0.5	6.9
Kentucky at White R.	Indianapolis	Northeast	39.2	12.4	26.5	8.0	12.7
Morris at Harding	Indianapolis	West	54.3	4.2	38.6	6.0	15.7
Meridian at South	Indianapolis	North	43.1	7.8	32.8	10.9	10.3
South at West	Indianapolis	East	52.2	40.1	44.2	40.8	8.0
Morris at Belmont	Indianapolis	East	40.9	0	36.3	0	4.6
Morris at Holt	Indianapolis	East	38.5	2.9	28.7	2.1	9.8
Northwestern at 16th	Indianapolis	South	38.2	6.6	39.4	0.2	1.2
U.S. 52 at South	Lafayette	South	50.7	20.6	39.2	27.9	11.5
U.S. 52 at South	Lafayette	North	44.8	13.9	31.1	15.4	13.7
U.S. 52 at Main	Lafayette	South	41.7	9.3	34.0	16.8	7.7
U.S. 52 at Main	Lafayette	North	41.2	14.0	30.8	17.8	10.4
Union at 18th	Lafayette	West	60.4	11.3	53.7	26.1	6.7
U.S. 31 at Markland	Kokomo	South	51.4	27.9	44.6	40.3	6.8
Kennedy at 169th	Hammond	North	76.7	33.7	69.2	41.3	7.5
Indianapolis at 141st							
	East Chicago	North	54.0	6.2	38.9	5.3	15.2

two treatments or average travel times is rejected, and there is 95% confidence that a truck delay exists.

The next step was to determine if the average stop time when trucks were present was significantly different from the average stop time when trucks were not present. The Wilcoxon Sign Rank Test was again employed, and the calculations are presented in Appendix D. The calculated T value of 121.0 was greater than the critical T value of 55 at a level of significance of .05. Since T calculated is greater than T critical, the hypothesis of no difference between the average stop times is not rejected. It is therefore concluded that the presence of trucks at an intersection does not significantly increase or decrease the average stop time for a vehicle. As a result stop time was not used in determining truck delay in this research. Only the running time was used in determining truck delay.

Developing A Model for Predicting Commercial Vehicle Delay

Regression analysis was chosen as the method to develop a model that would predict commercial vehicle delay at any given signalized intersection. Eighteen factors that were thought to have some effect on commercial vehicle delay were measured at each of the 23 intersection approaches studied. These factors or variables are presented in Table 8. Five of these variables are qualitative and the other thirteen are quantitative.

TABLE 8
RANGE OF VARIABLES AFFECTING COMMERCIAL VEHICLE DELAY

<u>Variable Number</u>	<u>Variable</u>	<u>Unit of Measurement</u>	<u>Range</u>
X1	Degree of Right Turn	Degrees	42 to 135
X2	Green Phase Length	Seconds	21. to 54.
X3	Percent Right Turns	Percent of Peak Hour Volume	0 to 37.9
X4	Peak Hour Load Factor	Number of Loaded Phases/ Total Phases	0 to .95
X5	Right Turn Curb Radius	Feet	10 to 100
X6	Parking on Approach	Yes or No	Yes & No
X7	Peak Hour Factor	Vehicles per Peak Hr.	.66 to .95
X8	Left Turn Lane	4xVeh. per Peak 15 min.	Yes & No
X9	Right Turn Lane	Yes or No	Yes & No
X10	Curbing on Approach	Yes or No	Yes & No
X11	Percent Left Turns	Yes or No	Yes & No
X12	Left Turn Green Phase	Percent of Peak Hour Volume	0.6 to 47.8
X13	Peak Hour Approach Volume	Yes or No	Yes & No
X14	Percent Single Units	Vehicles per Hour	544 to 1505
X15	Percent Truck Combinations	Percent of Peak Hour Volume	1.1 to 11.9
X16	Speed Limit on Approach	Percent of Peak Hour Volume	0 to 12.3
X17	Approach Width	Miles per Hour	25 to 55
X18	Metropolitan Area - Population	Feet	21. to 50.
		People	83,000 to
X19	Percent of Trucks	Percent of Peak Hour Volume	1,110,000
			1.5 to 20.4

The first step in developing the regression model was to use the "R-Squares For All Possible Regressions" statistical program (32). This program was available in the CDC 6500 Statistical Program Library at Purdue University. This Fortran IV routine computes and prints R-squares for every possible regression equation which involves the specified independent variables. This program is useful in determining a high R-square, when a large number of variables exist, and a minimum combination of variables is desired in the regression equation. A total of 262,142 R-squares were calculated for the eighteen independent variables and one dependent variable specified. Thirty-five R-squares were found to have values above .85. The highest R^2 was .871. The minimum combination of variables yielding a high R^2 was determined for a combination of 14 variables and a R^2 of .864.

Stepwise linear regression was then performed with these 14 independent variables on the dependent variable. Dummy variables were substituted for the qualitative variables and assigned a 00 value for a 'no' measure and a 01 value for a 'yes' measure. As previously stated, this combination of variables yielded a R^2 value of .864. The F value of 3.67 was found to be significant at the .05 alpha level.

Partial F tests were then performed on each variable, using a critical F value based on an alpha risk of .05. All 14 variables were found to have significant partial F values.

Improving the Model

The two alternatives that were used to improve the predicting power of the regression model were to consider interactions between predictor variables and to consider different transformations on each predictor variable.

Interactions that might add to the predicting power of the model were found by using the correlation matrix in the stepwise linear regression program previously printed and then applying the investigator's judgement as to which interactions have a practical 'sense'. The following interactions were computed and added to the stepwise linear regression program:

1. Percent of single unit trucks + percent of truck combinations
2. Percent of trucks x peak hour approach volume
3. Percent of left turns x peak hour approach volume x left turn lane x left turn green phase
4. Speed limit on approach x metropolitan area population
5. Degree of right turn x percent of trucks x percent of right turns

The resulting R^2 value was .882, which was an increase of .018 from the previous model. Partial F tests removed 5 variables, and the F value of 3.90 was found to be significant at the .05 probability level.

Relevant transformations on the variables were determined by interpreting plots of independent variables versus the dependent variable for a nonlinear relationship. Nonlinear relationships were interpreted for the variables speed limit on approach and degree of right turn. The plots of these two variables are presented in Appendix E. Both variables were interpreted to have a square root relationship with the dependent variable.

The addition of these two transformations to the program produced a new R^2 value of .917 and a F value of 5.15, which is significant at the .025 alpha level. This new R^2 increased .025 from the previous model.

Two more variables were created by combining each transformed variable with an existing interaction. However, the addition of these two variables to the program resulted in a decrease of .063 in the R^2 value. This new model was rejected.

A review of the developed model revealed that the two variables degree of right turn and square root of degree of right turn accounted for a combined R^2 of .242. The factor degree of right turn accounted for 25% of the variation about the mean truck delay. It was the investigator's judgement that this value of 25% was excessive for the actual effect of degree of right turn on truck delay. The decision was made to delete the variables degree of right turn and square root of degree of right turn from the model, and create the interaction degree of right turn

x percent of trucks. It was assumed that the predicting power lost from the deletion of these two variables would be transferred to the new interaction variable and to a lesser extent to the other predictor variables.

The stepwise linear regression program was again submitted, and the resulting model produced a R^2 value of .871 and an F value of 4.67, significant at the .025 alpha level. As expected, the R^2 of .242 from the original degree of right turn variables was transferred to the other predictor variables. The interaction degree of right turn x percent of trucks yielded an R^2 of .107. Other notable increases in R^2 were incurred by approach width, percent left turns x peak hour volume x left turn lane x left turn green phase, left turn green phase, left turn lane, and percent of trucks. The predictor variables found to be significant and their respective R^2 values are listed in Table 9. **The estimate of the standard error was 2.23.**

TABLE 9

SIGNIFICANT PREDICTOR VARIABLES FOR TRUCK DELAY

<u>Variable No.</u>	<u>Variable Description</u>	<u>R-square Change</u>
13	Peak hour volume in vehicles	.132
26	Degree of right turn x percent trucks	.107
19	Percent trucks	.099
08	Left turn lane (yes = 1 and no = 0)	.092
22	Percent left turns x peak hour volume x left turn lane x left turn green phase	.091
12	Left turn green phase in sec.	.075
24	Metropolitan area population x speed limit in miles per hour	.056
03	Percent right turns	.052
05	Curb radius in feet	.049
09	Right turn lane (yes = 1 and no = 0)	.046
17	Approach width in feet	.038
18	Metropolitan area population	.021
10	Curbing on approach (yes=1 and no=0)	.014

The model in its final form using the variable numbers as subscripts (see Table 9) is presented as follows:

$$\begin{aligned}
 Y = & -2.436 + .00969X_{13} + .0000427X_{26} + .036X_{19} -3.867X_8 \\
 & + .0000222X_{22} + 5.238X_{12} + .000000886X_{24} \\
 & - .236X_3 - .222X_5 + 5.920X_9 + .336X_{17} - .0000203X_{18} \\
 & - 9.572X_{10}
 \end{aligned}$$

Where Y is the truck delay in seconds. Variables that did not exhibit significance were parking, peak hour factor, load factor, and green phase length.

Further methods for improving the model, such as an orthogonal analysis, were not explored. It was decided the R^2 of .871 was satisfactory; and increasing it by such methods might result in a hypothetical increase, but would not be practically justified. Any sizable increase in R^2 probably can only be accomplished by the addition of new variables. Examples of variables that were not measured and which might be valuable are opposing peak hour volume, average daily traffic, and a variable capable of measuring the differences in human driving patterns.

An investigation of the residuals from the final model was used to check the practicality of the model in predicting truck delay. The frequency chart of the residuals is presented in Figure 3. The result appeared to approximate a normal curve; however, a W test was performed to validate the normality of the residuals. The calculated W value of .968 was greater than the critical value of

FIGURE 3. FREQUENCY OF RESIDUALS FOR TRUCK DELAY MODEL

.914 for an alpha level of .05. Thus, the hypothesis of normality was not rejected.

An examination of the arithmetic sign preceding each regression coefficient in the model proved to be very informative and revealed the practicality of the model. The signs were interpreted to read as follows:

1. The positive sign preceding peak hour volume indicates that an increase in peak hour volume increases truck delay.
2. The positive sign preceding percent of trucks indicates that the more trucks on an intersection approach, the greater the truck delay.
3. The negative sign preceding left turn lane indicates that a left turn lane reduces truck delay. The presence of a left turn lane indicates that at least one approach lane is used exclusively for through movements, (In this research, left turn lanes existed at only those approaches that had at least three approach lanes.) and the right lane carries the right-turning movements and some through movements. Most of the through passenger cars will travel in the center lane and most of the through trucks will travel in the right lane. Since most of the through passenger cars are not in the same lane as the trucks, a minimum truck delay is incurred.

4. The positive sign preceding left turn green phase indicates that a left turn green phase increases truck delay. This is probably due to the fact that a separate left turn green phase will decrease the through green phase time on that approach. This condition may cause drivers to accelerate faster than normal from a stop at an intersection. Since trucks are unable to equal the faster acceleration of passenger cars, a greater delay results.
5. The negative sign preceding percent of right turns indicates that truck delay decreases with an increase in right turns. Vehicles turning right do not accelerate as fast as through vehicles from a stop position. Consequently, a through vehicle will incur a delay from a right-turning vehicle preceding it. Therefore, an increase in percent of right turns will increase average vehicle delay but will offset or decrease the truck delay.
6. The negative sign preceding curb radius indicates that an increase in curb radius results in a decrease in truck delay. Trucks are able to increase their right-turning speed at a higher rate than passenger cars for larger curb radii. This increase in curb radius results in a decrease in truck delay.

7. The positive sign preceding right turn lane indicates that a right turn lane will increase truck delay. The presence of a right turn lane usually indicates that there is at least one other lane for through movements only. Consequently, the through-only lane or lanes are forced to carry all the through trucks. This condition increases truck delay for the following reasons:
 1. Right turning vehicles will cause no delay to through vehicles and thus, will not offset truck delay.
 2. Through vehicles cannot change lanes to avoid a slower moving truck.
8. The positive sign preceding approach width indicates that an increase in pavement width causes an increase in truck delay. Large pavement widths are usually accompanied by higher speed limits. These higher speed limits will increase truck delay because trucks take a proportionally longer time to accelerate to a higher speed than passenger cars.
9. The negative sign preceding metropolitan area population indicates that a larger metropolitan area reduces truck delay. In this research it was found that fringe areas and outlying business districts in larger metropolitan areas had speed

limits that were usually lower than those in similar locations in smaller metropolitan areas. As previously stated, lower speed limits reduce truck delay. Another reason for a larger metropolitan area reducing truck delay is that drivers in larger cities tend to be more aggressive. These drivers are more cautious of slow moving trucks and will often negotiate quick lane change maneuvers to avoid the slower moving truck.

10. The negative sign preceding curbing on approach indicates that the presence of curbing reduces truck delay. As previously stated in The Regression Analysis on Equivalency Factors, curbing among a roadway causes a reduction in vehicular speeds. This reduction in average vehicular speed increases average vehicular delay but offsets or decreases truck delay.

Arithmetic signs preceding interaction variables were not examined because at least one variable in every interaction had appeared separately as a significant predictor variable.

Testing The Model

An additional check of the practicality of the commercial vehicle delay model was accomplished by testing it against an independent intersection. This test intersection

was independent of those used in the data collection. The method of collecting travel times was also independent of that used in the data collection. Rather than using the Floating Car method, four observers were stationed in a fire training tower 40 feet above the intersection. Travel times were collected by visually noting landmarks that were adjacent to the intersection reference points. (See Data Collection for Commercial Vehicle Delay.) Vehicles were timed as they passed these landmarks.

A sample of 38% of the through vehicles was collected on the east approach of Teal Road at 18th Street in Lafayette. One-hundred and twenty-one travel times were collected when no commercial vehicles were ahead of the timed vehicle in the platoon of vehicles. This sample yielded an average travel time of 37.85 seconds, a sample standard deviation of 5.806, and a 95% confidence that the average travel time was within 1.03 seconds of the true average. Twenty-seven travel times were collected when commercial vehicles were ahead of the timed vehicle in the platoon of vehicles. This sample yielded an average travel time of 40.63 seconds, a sample standard deviation of 7.19, and a 95% confidence that the average travel time was within 2.71 seconds of the true average. The resulting commercial vehicle delay was $(40.63 - 37.85)$ or 2.78 seconds per vehicle.

The following intersection variables were measured at the test intersection:

1. Peak hour volume = 610 vehicles (394 through vehicles)
2. Degree of right turn = 90°
3. Percent of trucks = 5.6%
4. Left turn lane = yes
5. Percent of left turns = 23.3%
6. Left turn green phase = yes
7. Metropolitan area population = 109,378
8. Speed limit on approach = 35 miles per hour
9. Percent of right turns = 12.1%
10. Curb radius = 100 feet
11. Right turn lane = yes
12. Approach width = 42.5 feet
13. Curbing on approach = no (curbing actually existed but not at the edge of the traveled lane)

The values of these variables were inserted into the model, and a resulting value of 2.82 seconds was obtained. This value is only .04 seconds higher than that obtained from large scale sampling. This small error substantiates the model's practicality.

Right Turn Study

Nineteen curb radii were studied for this research project. Each curb radius was measured as a simple curve

radius. Radii varied from 10 feet to 90 feet. Table 10 summarizes the data collected for the Right Turn Study.

The first step in this study was to define the relationship between curb radii and vehicular speeds. Passenger car, single unit and bus, and truck combination right turn speeds were each plotted as a function of curb radius. No plot clearly defined a linear relationship. Transformations were then performed on the predictor variable curb radius, and the stepwise linear regression program (18) was employed to determine the best correlation between curb radius and vehicular speeds. The results are presented as regression equations as follows:

1. Passenger car right turn speed as a function of curb radius:

$$Y = 10.3299 + .8258(\text{curb radius})^{0.5}$$

$$R^2 = .643, F = 30.57, F_{.05} = 4.46, \text{stand. err.} = 1.11$$

2. Single unit and bus right turn speeds as a function of curb radius:

$$Y = 5.6825 + 2.3151(\text{curb radius})^{0.333}$$

$$- .0002907(\text{curb radius})^2$$

$$R^2 = .561, F = 10.21, F_{.05} = 3.64, \text{stand. err.} = .94$$

3. Truck combination right turn speed as a function of curb radius:

$$Y = -2.6314 + 4.377(\text{curb radius})^{0.333}$$

$$- .004601(\text{curb radius})^2$$

$$R^2 = .750, F = 22.45, F_{.05} = 3.68, \text{stand. err.} = 1.23$$

TABLE 10
SUMMARY OF DATA FOR RIGHT TURN STUDY

Intersection	Location	Curb Radius	Cross St. Width	Approach Width	Curbing Present	Pass. Car	Average Right Turn Speed	
							(mph) Single Unit	Truck Comb.
E. Chicago at Calumet	Hammond	16.0 ft	25.0 ft	12.5 ft	No	12.2	9.9	8.2
College at Washington	Indpls.	24.0	35.0	11.0	Yes	15.5	10.8	10.1
Emerson at English	Indpls.	10.0	20.5	10.0	Yes	13.5	11.1	6.6
Morris at Harding	Indpls.	27.5	21.5	18.0	Yes	14.8	12.3	9.5
Morris at Holt	Indpls.	52.5	36.0	12.0	No	17.0	13.8	12.0
Morris at Raymond	Indpls.	90.0	24.0	12.0	No	19.3	14.3	13.8
Morris at Tibbs	Indpls.	40.0	23.0	11.0	No	15.1	12.8	10.0
Penn. at South	Indpls.	16.0	19.5	15.5	Yes	12.0	13.0	---
Raymond at Morris	Indpls.	75.0	12.0	12.0	No	15.3	12.7	11.5
Raymond at Tibbs	Indpls.	34.0	24.0	11.2	No	16.5	14.6	12.5
Shadeland at 21st	Indpls.	50.0	33.0	18.0	Yes	18.2	14.3	14.6
South at Penn.	Indpls.	13.0	30.0	10.0	Yes	13.4	11.4	7.3
Tibbs at Kelly	Indpls.	52.5	12.0	10.5	No	13.0	13.0	12.1
Tibbs at Morris	Indpls.	40.0	24.0	16.0	Yes	14.3	13.3	9.5
21st at Shadeland	Indpls.	47.0	33.0	15.0	Yes	16.0	13.9	13.4
West at Washington	Indpls.	17.0	33.0	12.0	Yes	14.0	10.7	9.3
South at U.S. 52	Lafayette	40.0	38.0	19.0	Yes	15.0	13.2	11.8
U.S. 52 at South	Lafayette	40.0	27.0	12.0	Yes	15.1	12.9	10.9
Lincoln at Wicker	Scherville	60.0	39.0	14.5	Yes	16.8	13.2	13.9

The regression line plots resulting from each of the regression equations are presented in Figures 4, 5, and 6.

Figure 7 combines all three regression line plots. The shaded area in Figure 7 represents the 30 to 50 foot curb radius that is recommended for trucks at intersecting streets. (See Literature Review.) This 30 to 50 foot range was subdivided into 5 foot intervals, and the regression equations were employed to calculate the resulting vehicular speeds. The results are presented in Table 11.

TABLE 11

RIGHT TURN VEHICULAR SPEEDS FOR 30 TO 50 FEET CURB RADII

Curb Radius (feet)	Passenger Car Speed (mph)	Single Unit Tk. Speed (mph)	Truck Comb. Speed (mph)	Pas. Car Speed - Single Unit (mph)	Pas. Car Speed - Truck Comb. (mph)
30	14.85	12.60	10.54	2.25	4.34
35	15.21	12.89	11.11	2.32	4.10
40	15.55	13.12	11.58	2.43	3.97
45	15.87	13.32	11.99	2.55	3.88
50	16.17	13.47	12.32	2.70	3.85

The difference between passenger car and single unit truck average right turn speed was smallest at a 30 foot curb radius and largest at a 50 foot curb radius. The difference between passenger car and truck combination average right turn speed was smallest at a 50 foot curb radius and largest at a 30 foot curb radius. From a passenger

FIGURE 4. RIGHT TURN SPEED OF PASSENGER CARS AS A FUNCTION OF CURB RADIUS

FIGURE 5. RIGHT TURN SPEED OF SINGLE UNIT TRUCKS AS A FUNCTION OF CURB RADIUS

FIGURE 6. RIGHT TURN SPEED OF TRUCK COMBINATIONS AS A FUNCTION OF CURB RADIUS

FIGURE 7. RIGHT TURN SPEEDS AS A FUNCTION OF CURB RADIUS

car delay viewpoint and within A.A.S.H.O. recommended limits, minimum delay caused by a right-turning single unit truck is incurred at a curb radius of 30 feet, and minimum delay caused by a right-turning truck combination is incurred at a curb radius of 50 feet. A further inspection of Figure 7 revealed that single unit truck and truck combination speeds increased very little beyond a 60 foot radius. From a 60 foot to 90 foot curb radius, the increase in speed for a single unit truck was less than .1 mile per hour and for a truck combination was .4 miles per hour. These small increases in speed do not justify a 30 foot increase in curb radius, and result in a 60 foot curb radius as the maximum desirable. Therefore, it is recommended that a 30 foot curb radius be employed at intersections on major streets that use a single unit truck as the design vehicle. At intersections on major streets that use a truck combination as the design vehicle, a 60 foot curb radius is recommended. These recommendations apply to intersections located in fringe areas and outlying business districts of metropolitan areas.

Three additional variables were added to the regression equations previously determined, in an effort to increase the predicting power of the equations. The three variables were approach turning width, cross street turning width, and curbing on approach. Approach turning width was measured as the pavement width of the right approach lane.

For this research, approach turning widths ranged from 10 to 19 feet. The cross street turning width was measured as the pavement width from right edge to centerline of cross street. Cross street turning widths varied from 12 to 39 feet. Curbing was recorded as a 'yes' or 'no'. Twelve approaches had curbing present and seven did not. Curbing was entered into the stepwise linear regression program as a dummy variable with a value of 00 assigned for a 'no' recording and a value of 01 assigned for a 'yes' recording.

The regression analysis was then performed using the following predictor variables.

1. Curb radius in feet
2. Transformations of curb radius in feet
3. Approach turning width in feet
4. Cross street turning width in feet
5. Curbing on Approach (yes = 1 and no = 0)
6. Interactions of 3. with 1., 2., and 4.
7. Interactions of 4. with 1. and 2.

The regression equations that yielded the highest R-squares are listed as follows:

1. Passenger car right turn speed = Y_1

$$Y_1 = -3.5854 + .00309X_1^2X_2 - 6.2116X_1^5 + 16.8311X_1^{.333} - .6540X_4$$

$$R^2 = .806, F + 14.59, F_{.05} = 3.13, \text{stand. error} = .90$$

2. Average single unit right turn speed = Y_2

$$Y_2 = 4.1992 + 2.7449X_1^{.333} + .00291X_1X_2 + .3495X_3 \\ - .01071X_2X_3 - .1078X_1 - 7.03(10^{-9})X_1^3X_2$$

$$R^2 = .689, F = 4.44, F_{.05} = 3.00, \text{stand. error} = .91$$

3. Average truck combination right turn speed = Y_3

$$Y_3 = - 5.0673 + 5.3821X_1^{.333} + .02707X_1^{.333}X_2 \\ - .09291X_1 - .9098X_4$$

$$R^2 = .862, F = 20.36, F_{.05} = 3.19, \text{stand. error} = .98$$

Where:

X_1 = curb radius in feet

X_2 = cross street turning width in feet

X_3 = approach turning width in feet

X_4 = curbing on approach (yes = 1 and no = 0)

A W test was performed on the residuals for each regression equation. The hypothesis of normality of the residuals was not rejected for any of the regression equations.

The R^2 and F values from the regression equation for average single unit right turn speed were not extremely high. The regression equations for passenger car and truck combination average right turn speeds yielded high R^2 and F values; and thus, no hesitation should be made in using them to determine average right-turning speeds. Figures 8 and 9 are plots of these regression equations.

The stepwise linear regression program (35) was also employed to determine a regression equation for predicting average right turn speeds for commercial vehicles. The data

FIGURE 8. RIGHT TURN SPEED, IN MILES PER HOUR, FOR PASSENGER CARS

FIGURE 9. RIGHT TURN SPEED, IN MILES PER HOUR, FOR TRUCK COMBINATIONS

for right turn speeds of single unit trucks and truck combinations was averaged to obtain speed data for commercial vehicles. The same predictor variables previously used were again employed. The resulting regression equation is as follows:

Average commercial vehicle right turn speed = Y_4

$$Y_4 = .3364 + 3.6651X_1^{.333} + .00100X_2X_3 + .000000X_1^3X_2 - .0008271X_1^2$$

$$R^2 = .844, F = 17.65, F_{.05} = 3.19, \text{stand. error} = .79$$

A W Test was performed on the residuals, and the hypothesis of normality of the residuals was not rejected.

The R^2 and **F values** from the regression equation are significantly high. Figure 10 is a plot of the regression equation.

FIGURE 10. RIGHT TURN SPEED, IN MILES PER HOUR, FOR COMMERCIAL VEHICLES

SUMMARY OF RESULTS

The significant results of this research project are listed below.

Capacity Analysis

1. Truck equivalency factors were determined for the general classification of commercial vehicles and for the subclassifications of single unit trucks and buses, and truck combinations. For each of these three classifications, equivalency factors were determined for the following approach conditions.

- 1) Three lane approach
- 2) Two lane approach
- 3) Through and left turn lane
- 4) Through lane
- 5) Through and right turn lane

Fifteen truck equivalency factors were determined and, they are listed in Table 2.

2. Stepwise multiple regression analysis was employed to develop regression equations that would predict truck equivalency factors for any given intersection approach. The first regression equation developed used eight intersection variables to explain the variation in the original

16 commercial vehicle equivalency factors. This regression equation and its R^2 and F values are given as follows:

$$\begin{aligned} \text{Truck Equivalency Factor} = & -.907 + .362(\text{load factor}) \\ & + .0199(\text{approach width}) + .0837(\text{green phase length}) \\ & - .0000675(\text{green phase length} \times \text{peak hour approach} \\ & \quad \text{length}) \\ & - .00183(\text{percent of truck combinations}) + .000340 \\ & \quad (\text{percent of truck combinations} \times \text{right turn} \\ & \quad \text{curb radius}) \\ & + .00251(\text{peak hour approach volume}) - 1.159(\text{peak} \\ & \quad \text{hour factor}) \end{aligned}$$

$$R^2 = .9578$$

$$F = 19.87, F_{\text{critical}} = 3.73, \text{significant}$$

The second regression equation was modified by deleting the variables load factor, percent of truck combinations, and percent of single unit trucks. These variables are difficult to measure in the field. The variable percent of trucks was then added to the remaining list of variables. The second regression equation used nine intersection variables and is given as follows:

$$\begin{aligned} \text{Truck Equivalency Factor} = & - 1.245 + .0934(\text{green phase} \\ & \text{length}) + .0112(\text{approach width}) + .00189(\text{peak hour} \\ & \text{approach volume}) - .238(\text{curbing}) + .0000085(\text{per-} \\ & \text{cent of trucks} \times \text{right turn curb radius}) - \\ & .0000626(\text{green phase length} \times \text{peak hour approach} \\ & \text{volume}) + .00845(\text{right turn curb radius}) - .00232 \end{aligned}$$

(percent of right turns) - .00949(percent of trucks)

$$R^2 = .9054$$

$$F = 6.38, F_{\text{critical}} = 4.10, \text{significant}$$

3. A study was made to determine if a headway analysis could be used at an intersection for determining truck equivalency factors. Truck equivalency factors were determined at five intersections using both the capacity analysis and the headway analysis methods. The analysis and conditions of data collection resulted in the conclusion that using a headway analysis to determine truck or commercial vehicle equivalency factors at intersection approaches is unreliable.

Delay Analysis

1. Twenty-three intersection approaches were studied for delay caused by commercial vehicles. It was found that a passenger car's running travel time through an intersection is increased from an average of 39.8 seconds to 49.4 seconds, when one or more commercial vehicles are traveling ahead of it in the same platoon of vehicles. This condition applies to outlying business districts and fringe areas of metropolitan areas.

The average stop time at the twenty-three intersection approaches was 13.5 seconds when commercial vehicles were present in the queue of vehicles and 14.4 seconds when

commercial vehicles were not present in the queue. There was no significant difference found between these two average stop times.

2. A regression model was developed to predict average commercial vehicle delay at any given signalized intersection. Eighteen factors affecting commercial vehicle delay were considered in the regression analysis along with interaction and higher order terms. The final model used thirteen factors or variables to explain the variation in the average commercial vehicle delay at the 23 approaches studied. The final model and its R^2 and F values are given as follows:

$$\begin{aligned}
 Y \text{ (seconds per vehicle)} = & -2.436 + .00969(\text{peak hour volume}) + \\
 & .0000427(\text{degree of right turn} \times \text{percent trucks}) + .236(\text{percent of trucks}) - 3.867 \\
 & (\text{left turn lane}) + .0000222(\text{percent left turns} \times \text{peak hour volume} \times \text{left turn lane} \\
 & \times \text{left turn green phase}) + 5.238(\text{left turn green phase}) + .000000886(\text{metropolitan} \\
 & \text{area population} \times \text{speed limit}) - .236 \\
 & (\text{percent right turns}) - .222(\text{curb radius}) \\
 & + 5.920(\text{right turn lane}) + .336(\text{approach lane}) - .0000203(\text{metropolitan area popu-} \\
 & \text{lation}) - 9.572(\text{curbing on approach})
 \end{aligned}$$

$$R^2 = .871$$

$$F = 4.67, F_{\text{critical}} = 3.84, \text{ significant}$$

The significance of the model was evaluated by the typical normality test on the residuals. The practicality of the model was determined by an examination of the sign condition preceding the coefficients of the independent variables. The results of both substantiated the significance and practicality of the model.

A further check on the practicality of the model was performed by testing it against results from an independent intersection. Travel times were obtained from 38% of the through vehicles at the test intersection. An average commercial vehicle delay of 2.78 seconds per vehicle was calculated from the data. The value obtained from the model was 2.82 seconds. An error of .04 seconds between the model and actuality substantiated the practicality of the model.

Right Turn Analysis

1. The stepwise multiple regression procedure was employed to relate the right turn speeds of passenger cars, single unit trucks, and truck combinations to right turn curb radii. It was found that curb radii terms explained 56% of the variation of right turn speeds of passenger cars, 64% of the right turn speeds of single unit trucks, and 75% of the right turn speeds of truck combinations. The predicted values of right turn speeds were plotted as a function of curb radii for the three classes of vehicles. These plots are presented in Figures 4, 5, and 6.

2. In an attempt to improve the predicting power of the above regression equations, the independent variables approach turning width, cross street turning width, and curbing on approach were added to the stepwise linear regression program. A fourth regression equation with the dependent variable commercial vehicle right turn speed was also determined. The resulting regression equations yielded the following R^2 values: .8065 for predicting passenger car right turn speeds, .6894 for predicting single unit truck right turn speed, .8623 for predicting truck combination right turn speed, and .844 for predicting commercial vehicle right turn speed. Figures 8, 9, and 10 were developed to incorporate the above regression equations in a graph form. The regression equation for single unit truck right turn speed was not incorporated in a graph form because of the rather low predicting power of its regression equation.

CONCLUSIONS AND RECOMMENDATIONS

General Conclusions

The following general conclusions concerning the effects of commercial vehicles on intersection capacity and delay were determined from this research project.

1. In a capacity sense, one commercial vehicle is equivalent to 1.85 passenger cars at a signalized intersection. Also, one single unit truck is equivalent to 1.72 passenger cars, and one truck combination is equivalent to 2.37 passenger cars at a signalized intersection.

2. The presence of commercial vehicles in a platoon of vehicles approaching a signalized intersection does not significantly increase or decrease the average vehicle stop time, as defined in this research, at the signalized intersection.

3. The factors that have a significant effect on increasing commercial vehicle delay are peak hour volume, percent of commercial vehicles, the presence of a left turn green phase, the presence of a right turn only lane, and the approach width. The factors that have a significant effect on reducing commercial vehicle delay are the

presence of a left turn only lane, the percent of right turns, the right turn curb radius, the metropolitan area population, and the presence of curbing on the approach. Factors that did not cause a significant decrease or increase in commercial vehicle delay are parking on the approach, peak hour factor, load factor, and green phase length.

4. An analysis of the Right Turn Study reveals the maximum right turn speed for a truck combination at a signalized intersection is approximately 14 miles per hour and approximately 15 miles per hour for a single unit truck.

5. The presence of curbing at a signalized intersection approach was found to decrease the right turn speed of passenger cars by .7 miles per hour, to decrease the right turn speed of truck combinations by .9 miles per hour, and to have no effect on the right turn speed of single unit trucks.

Recommendations

The following recommendations resulting from this research project are presented.

1. The equivalency factors of passenger cars to a commercial vehicle, to a single unit truck, and to a truck combination as presented in Table 2 are recommended for application to signalized intersections that have variables falling within the ranges indicated in Table 4.

2. The commercial vehicle delay model presented in the Analysis Of Data is recommended for application to signalized intersections that have variables falling within the ranges indicated in Table 8. This model will predict the average commercial vehicle delay in seconds experienced by a vehicle traveling through a signalized intersection.

3. The right turn speeds of passenger cars, truck combinations, and commercial vehicles as presented in Figures 8, 9, and 10 are recommended for application in corner radius design or in delay and capacity analysis.

4. The American Association of State Highway Officials recommends a 30 to 50 foot curb radius range for trucks at intersecting streets (1). This research found that a 30 foot radius caused the least delay for a passenger car following a single unit truck and a 50 foot radius caused the least delay for a passenger car following a truck combination. It was also determined that corner radii greater than 60 feet did not appreciably increase right turn speeds of single unit trucks and truck combinations. Therefore, it is recommended that a 30 feet corner radius be employed at intersections on major streets that use a single unit truck as the design vehicle. On major streets at intersections that use a truck combination as the design vehicle, a 60 feet corner radius is recommended where economically feasible.

RECOMMENDATIONS FOR FURTHER RESEARCH

During the development of this research project the need for further research on the following subjects was noted:

1. The commercial vehicle equivalency factors and commercial vehicle delay model that were developed in this research should be applied to metropolitan areas outside of Indiana to test their applicability outside Indiana.

2. The author was unable to measure all the variables that could have an effect on commercial vehicle delay at signalized intersections. Further research can be directed towards measuring these variables and thus modifying the model developed in this research.

3. The use of regression analysis in predicting delay at intersections has not been explored extensively. This research project has developed a model, based on a regression equation, that predicts commercial vehicle delay at signalized intersections. It is suggested that other researchers attempt to apply regression analysis in predicting vehicle delay at intersections.

4. An economic study of intersection design warrants based on excessive commercial vehicle delay is recommended. This economic study would determine the amount of commercial vehicle delay that is needed to justify intersection designs that would minimize this delay. Examples are installing larger corner radii or an additional lane for slow moving trucks.

5. The regression equation for predicting right turn speed of single unit trucks yielded an R^2 of only .689. This was considered low in comparison of R-squares from passenger car, truck combination, and commercial vehicle regression equations. It is recommended that a further study of right turn speeds of single unit trucks be initiated with the purpose of developing a regression equation with a R^2 above .80.

BIBLIOGRAPHY

BIBLIOGRAPHY

1. American Association of State Highway Officials, A Policy on Design of Urban Highways and Arterial Streets, Washington, D.C., 1973.
2. American Association of State Highway Officials, A Policy on Geometric Design of Rural Highways, Washington, D.C., 1965.
3. Automobile Manufacturers Association, Inc., 1971 Automobile Facts and Figures, 320 New Center Building, Detroit, Michigan.
4. Automobile Manufacturers Association, Inc., 1971 Motor Truck Facts, 320 New Center Building, Detroit, Michigan.
5. Baerwald, John E., Traffic Engineering Handbook, Institute of Traffic Engineers, Washington, D.C., 1965, p. 30.
6. Berry, Donald S., and Van Til, C.J., "A Comparison of Three Methods for Measuring Delay at Intersections", Traffic Engineering, December, 1954, pp. 93-99.
7. Bent, Nie, and Hull, Statistical Package for the Social Sciences, McGraw Hill, 1970.
8. Bone, A.J., "Effect of Traffic Delays on Gasoline Consumption", Highway Research Board, Proceedings, Volume 19, 1939, pp. 99-125.
9. Bureau of the Census, 1970 Census of the Population, Volume 1, Part A, Section 1, U.S. Department of Commerce, May, 1972
10. Campbell, L.R., and Sagi, G.S., "Vehicle Delay at Signalized Intersections", Traffic Engineering, February, 1969, pp. 32-40.

11. Chicago Area Transportation Study, "Automobile Operating Costs at Various Speeds", Volume III, April 1962, p. 126.
12. City of Los Angeles Traffic Department, "Operation of Delay Meter", Staff Report 26.00, October, 1967.
13. Claffey, Paul J., "Running Cost of Motor Vehicles as Affected by Highway Design - Interim Report", National Cooperative Highway Research Program, Report 13, Highway Research Board, Washington, D.C., 1965.
14. Deen, Thomas B., "Acceleration Lane Lengths for Heavy Vehicles", Traffic Engineering, February, 1957, p. 215.
15. Geiger, D.R., Sofokidis, H., and Tilles, D.L., "Evaluation of Intersection-Delay Measurement Techniques", Highway Research Record 453, 1973, pp. 28-39.
16. Hanley, Dan G., The Value of Travel Time for Passenger Cars: A Preliminary Study, Stanford Research Institute, Menlo Park, California, January, 1963, p. 5.
17. Highway Research Board, Highway Capacity Manual, Highway Research Board Special Report 87, Washington, D.C., 1965.
18. Humphreys, Jack B., "Effect of Trucks on the Urban Freeway", Highway Research Record 308, 1970, pp. 62-75.
19. Humphreys, Jack B., and Tidwell, John E., "Relation of Signalized Level of Service to Failure Rate and Average Individual Delay", Highway Research Record 321, 1970, pp. 16-28.
20. Institute of Traffic Engineers, "Motor Vehicle Registration", Traffic Engineering, August, 1973, p. 90.
21. Institute of Traffic Engineers, "Route Selection Subcommittee Report", Traffic Engineering, April, 1973, pp. 23-25.
22. Koeford, K.V.M., "The Capacity of Urban Streets and Intersections", Traffic Engineering, August, 1955, pp. 450-458.

23. Kope, C.T., "Requirements for Truck Performance Evaluation", Society of Automotive Engineers, 485 Lexington Avenue, New York, 1957.
24. Lane, R., "Introduction to Traffic Engineering-4. Road Capacity", Traffic Engineering and Control, January, 1968, p. 442.
25. Leong, H.J.W., "Some Aspects of Urban Intersection Capacity", Australian Road Research Board, Proceedings 1964, Volume 2, Part 1, pp. 305-334.
26. Levinson, Herbert S., and Votaw, David F., Elementary Sampling for Traffic Engineers, Eno Foundation for Highway Traffic Control, Saugatuck, Connecticut, 1962, pp. 80-81.
27. Myers, Ramond H., and Walpole, Ronald E., Probability and Statistics for Engineers and Scientists, Macmillan Company, New York, 1972.
28. Norman, Olav Koch, "Variations in Flow at Intersections as Related to Size of City, Type of Facility, and Capacity Utilization", Highway Research Bulletin 352, 1962, pp. 55-99.
29. Norman, O. K., and Walker, W. P., "Highway Capacity-Practical Applications of Research", Public Roads, October and December, 1949, pp. 229-245.
30. Ostle, Bernard, Statistics in Research, Second Edition, The Iowa State University Press, Ames, 1963.
31. Rodgers, Lionel M., and Sands, Leo G., Automobile Traffic Signal Control Systems, Chilton Book Company, 1969, pp. 114-117.
32. "R-Squares for all Possible Regressions", Statistical Laboratory, Library Program, Purdue University, June, 1972.
33. Scraggs, D.A., "The Passenger Car Unit Equivalent of a 'Heavy' Vehicle in Single Flow at Traffic Signals", Laboratory Note No. LN-573, Road Research Laboratory, Harmondsworth, England, April, 1964.
34. Shapiro, S.S., and Wilk, M.B., "ANOVA Test for Normality (Complete Samples)", Biometrika, 52, 3 and 4, 1965, pp. 591-611.

35. "SPSS 15: Regression", Statistical Laboratory, Library Program, Purdue University, April, 1973.
36. Van Til, C.J., "A New Camera-Intervalometer for Taking Spaced Serial Photos", University of California, Berkley, ITTE Research Report 17, 1954.
37. Wardrop, J.G., and Webster, F.V., "Capacity of Urban Intersections", Traffic Engineering and Control, November, 1962, pp. 396-401.
38. Wildermuth, B.R., "Average Vehicle Headways at Signalized Intersections", Traffic Engineering, November, 1962, pp. 17-20.
39. Winfrey, Robley, Economic Analysis for Highways, International Textbook Company, 1969.

APPENDICES

APPENDIX A

COST OF VEHICLE TRAVEL TIME

The cost of vehicle travel time is the cost of operating, maintaining, and depreciation of the vehicle and the value of time for the driver and passengers. The cost of operation, maintenance, and depreciation is about 13 cents a mile, according to a recent study by Winfrey (39). The average vehicle speed in an urban area is 27 miles per hour (2). Thus, 13 cents per mile is equal to 6 cents per vehicle minute in an urban area.

The American Association of State Highway Officials recommended a value of time of \$1.55 per vehicle hour in 1959 (16). Since prices and salaries are about doubled from those of 1959, it is assumed that the value of time has also doubled to \$3.10 per vehicle hour or 5 cents per vehicle minute. The total cost of travel time is then 6 cents plus 5 cents or 11 cents per vehicle minute.

Appendix B

DATA COLLECTION SHEETS

GENERAL ANALYSIS OF INTERSECTION

NAME OF INTERSECTION _____ & _____

LOCATION OF INTERSECTION _____

LOCATION WITHIN METRO. AREA _____

SIZE OF METRO. AREA _____

APPROACH BEING ANALYZED _____ APPROACH, ON _____

APPROACH LANE WIDTHS:

LEFT _____

CENTER _____

RIGHT _____

RIGHT TURN CURB RADIUS _____, TYPE OF CURVE _____

SPEED LIMIT ON APPROACH _____

TYPE OF SIGNAL _____

CYCLE TIME ON APPROACH:

GREEN _____ YELLOW _____ RED _____ SPECIAL GREEN _____ TOTAL _____

TIME OF DAY _____

WEATHER _____

ADT OF ROADWAY _____ & PEAK-HOUR TRAFFIC _____

PEAK-HOUR FACTOR _____

PARKING CONDITIONS _____

CHANNELIZATION ON APPROACH _____

CROSS STREET WIDTH: RIGHT-TURNING VEHICLES _____ & LEFT-TURNING VEHICLES _____

CURBING ON APPROACH _____

CAPACITY ANALYSIS
(continued)

[illegible][illegible][illegible]

SUB _____
TOT _____

LOAD FACTOR

% COM. VEH'S.

4 TURNS

LEFT

CENTER

RIGHT

TOTAL

Abstract

—————

Abstract

RIGHT-TURN ANALYSIS

DATE	TIME
------	------

VEH. MAKING RT. TURN	AMT. OF EN- CROACHMENT	TIMES
-------------------------	---------------------------	-------

PAS. CAR

SIN. UNIT

TK. COMB.

BUS

AVG. PAS. CAR TIME _____
AVG. SIN. UNIT TIME _____
AVG. TK. COMB. TIME _____

HEADWAY ANALYSIS

DATE	TIME
------	------

TYPE OF SUC- CESSIVE VEH'S	MOVEMENT @ INTER.	TIMES
-------------------------------	----------------------	-------

PAS. CAR

STRAIGHT

LEFT TN.

RT. TN.

SIN. UNIT

STRAIGHT

LEFT TN.

RT. TN.

TK. COMB.

STRAIGHT

LEFT TN.

RT. TN.

BUS

STRAIGHT

LEFT TN.

RT. TN.

AVG. PAS. CAR HEADWAY _____
AVG. TRUCK HEADWAY _____

APPENDIX C

WILCOXIN SIGNED RANK TEST FOR AVERAGE TRUCK DELAY TIMES

<u>Intersection</u>	<u>Sample Number</u>	<u>Differences*</u>	<u>Rank</u>	<u>Signed Rank</u>	
				<u>Positive</u>	<u>Negative</u>
Kentucky at Morris	1	5.8	3	3	
Kentucky at West	2	15.6	22	22	
Kentucky at Harding	3	11.0	15	15	
Morris at Tibbs	4	11.8	17	17	
Morris at Harding	5	8.2	11	11	
Morris at Tibbs	6	7.5	7.5	7.5	
Morris at Harding	7	12.3	18	18	
Virginia at Stevens	8	6.9	6	6	
Kentucky at White R.	9	12.7	19	19	
Morris at Harding	10	15.7	23	23	
Meridian at South	11	10.3	13	13	
South at West	12	8.0	10	10	
Morris at Belmont	13	4.6	2	2	
Morris at Holt	14	9.8	12	12	
Northwestern at 16th	15	-1.2	1		1
U.S. 52 at South	16	11.5	16	16	
U.S. 52 at South	17	13.7	20	20	
U.S. 52 at Main	18	7.7	9	9	
U.S. 52 at Main	19	10.4	14	14	
Union at 18th	20	6.7	4	4	
U.S. 31 at Markland	21	6.3	5	5	
Kennedy at 169th	22	7.5	7.5	7.5	
Indianapolis at 141st	23	15.2	1	1	

*Differences = truck delay

Totals: 275

1

 $T_{\text{minimum}} = 1$ $T_n = 23, \alpha = .05 = 73$

Reject hypothesis of equality. $T = 1$ is significant at a 95% confidence level.

APPENDIX D

WILCOXON SIGNED RANK TEST FOR AVERAGE STOP TIMES

<u>Intersection</u>	<u>Sample Number</u>	<u>Differences*</u>	<u>Rank</u>	<u>Signed Rank</u>	
				<u>Positive</u>	<u>Negative</u>
Kentucky at Morris	1	-.3	3		3
Kentucky at West	2	+7.9	20	+20	
Kentucky at Harding	3	+9.7	21	+21	
Morris at Tibbs	4	+1.0	9	+9	
Morris at Harding	5	-.1	2		2
Morris at Tibbs	6	+5.7	15	+15	
Morris at Harding	7	+.7	5	+5	
Virginia at Stevens	8	+.9	7.5	+7.5	
Kentucky at White R.	9	+4.4	14	+14	
Morris at Harding	10	-1.8	11		11
Meridian at South	11	-3.1	12		12
South at West	12	-.6	4		4
Morris at Belmont	13	0	1		
Morris at Holt	14	+.8	6	+6	
Northwestern at 16th	15	+6.4	16	+16	
U.S. 52 at South	16	-7.3	18		18
U.S. 52 at South	17	-1.5	10		10
U.S. 52 at Main	18	-7.0	17		17
U.S. 52 at Main	19	-3.8	13		13
Union at 18th	20	-14.8	23		23
U.S. 31 at Markland	21	-12.5	22		22
Kennedy at 169th	22	-7.6	19		19
Indianapolis at 141st	23	+.2	7.5	+7.5	

*Differences = stop time with trucks Totals: 121.0 154

- stop time without tks. $T_{\text{minimum}} = 121.0$

$T_{n=23, \alpha=.05} = 73$

Do not reject hypothesis of equality. $T = 121.0$ is not significant at 95% confidence level.

Appendix E

PLOTS OF NONLINEAR RELATIONSHIPS
BETWEEN INTERSECTION VARIABLES AND TRUCK DELAY

FIGURE EI. TRUCK DELAY AS A FUNCTION
OF SPEED LIMIT ON APPROACH

FIGURE E2. TRUCK DELAY AS A
FUNCTION OF DEGREE OF
RIGHT TURN

COVER DESIGN BY ALDO GIORGINI