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16. Abstract

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The Poisson arrival process and queueing theory were utilized to derive predictive models of delay for the four study cases. These models predict delay as a function of ramp volume, frontage road volume, and gap acceptance parameters.

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DELAY TO FRONTAGE ROAD VEHICLES AT INTERSECTIONS WITH RAMPS

Research Report 402-2

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Warrants for One-Way Frontage Roads
Research Study Number 2-8-86-402

Sponsored by
Texas State Department of Highways and Public Transportation
In cooperation with the
U.S. Department of Transportation
Federal Highway Administration

Texas Transportation Institute
The Texas A&M University System
College Station, Texas

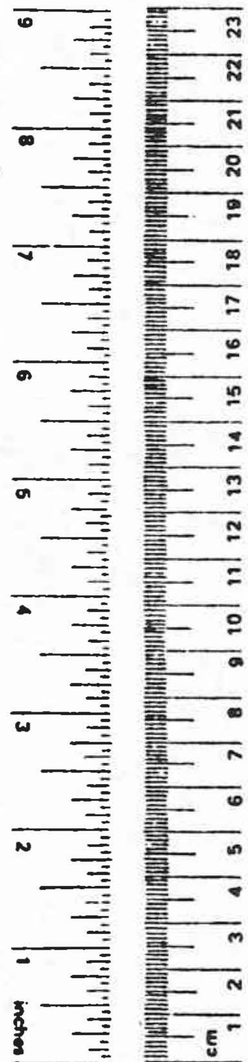
June 1988

METRIC CONVERSION FACTORS

Approximate Conversions to Metric Measures

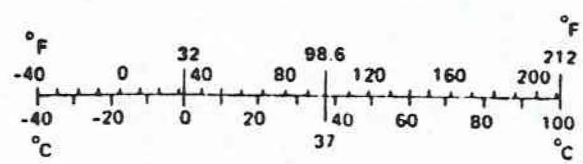
Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.



Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



ABSTRACT

This report contains details of and results from investigations of the delay incurred by two-way frontage road traffic at intersections with freeway ramps. All of the studies were conducted where frontage road traffic yielded to the ramp traffic. Studies were conducted on both lanes of a two-way frontage road at an intersection with an exit ramp, and on a lane of a two-way frontage road opposing the traffic turning onto the entry ramp. One study was conducted at an exit ramp intersection with a two lane, one-way frontage road for contrast with the two-way situations. All of the studies were situated in medium-sized towns in Texas.

The Poisson arrival process and queueing theory were utilized to derive predictive models of delay for the four study cases. These models predict delay as a function of ramp volume, frontage road volume, and gap acceptance parameters.

KEY WORDS: Delay, Entry Ramps, Exit Ramps, Frontage Roads, Queueing.

SUMMARY

This report contains details of and results from investigations of the delay incurred by frontage road traffic at intersections with freeway ramps. Four studies were conducted at sites in medium-sized Texas cities where two moving lanes of frontage road traffic yielded to ramp traffic. The intent of the study was to derive relationships between delay and the volumes on the ramp and frontage road.

The following situations were studied:

Case 1 - one-way frontage road intersection with exit ramp converging movement (used for comparison with two-way frontage road delay);

Case 2 - two-way frontage road intersection with exit ramp, converging movement;

Case 3 - two-way frontage road intersection with exit ramp, contraflow movement; and

Case 4 - two-way frontage road intersection with entry ramp, contraflow movement.

Data from the studies were processed in a manner so that individual vehicles could be tracked as they traveled through the area of the ramp--frontage road intersection. The sequence of vehicle passages was recorded in real time so it could be determined by examination of the data whether or not a frontage road vehicle yielded to a ramp vehicle. The amount of delay to the frontage road vehicles was found for several 15-minute intervals. The size of headways between ramp vehicles which the frontage road motorists found large enough to accept were also evaluated.

Assuming that ramp traffic arrivals could be described by the Poisson process, and knowing the headway acceptance tendencies of each site, that part of the total time period with adequate headways for frontage road vehicles to proceed was found. This value, divided by the headway at which frontage road vehicles would follow each other through the intersection, yielded potential capacity for frontage road traffic at the intersection. This potential capacity is the same as service rate in queueing theory. By modeling the frontage road stream as a queueing system, the queueing delay per frontage road vehicle was found. Recognizing that non-queueing sources of delay, such as time lost while resuming speed after having yielded, are also present, field-measured total delay was regressed against queueing delay to derive

models by which total delay could be predicted. Thus delay to frontage road vehicles was expressed, through a sequence of calculations, as a function of ramp volume, frontage road volume, and gap acceptance parameters.

In addition to predicting delays, the fraction of frontage road traffic which was delayed was expressed as a function of the frontage road volume divided by the service rate. Referring to the previous Study 288, which, based on accident experience, proposed warrants to convert two-way frontage roads to one-way when volumes reached certain levels, it was found that these warranting volumes would be accompanied by 25% to 50% of the frontage road traffic having a conflict with the ramp traffic.

IMPLEMENTATION

This report contains models which can be used to predict the anticipated delay to frontage road vehicles required to yield at ramp--frontage road intersections. These models can be used to predict delays for various frontage road operational strategies, specifically two-way versus one-way operation. When the delays at ramp--frontage road intersections are evaluated along with other delays (such as at crossing street intersections) and vehicle running times, the overall net travel time advantages and disadvantages can be calculated. This information would be useful when evaluating the impacts of converting a frontage road from two-way to one-way operation, and in better understanding the various trade-offs involved in such conversions.

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(Dist. 15), and Red Lindsay (FHWA).

The field studies which were conducted would not have been possible without the assistance of Department employees. Our San Marcos study was arranged through Mr. R.A. Brown, District Engineer for District 14. Mr. Delbert Chance of the San Marcos office provided field assistance. Studies in Bryan and in College Station were arranged through Mr. Carol Ziegler, District Engineer for District 17. Mr. Delmar Smith of the Bryan office provided field assistance.

DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the opinions, findings, and conclusions presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration, or the State Department of Highways and Public Transportation. This report does not constitute a standard, specification, or regulation.

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I. INTRODUCTION

This report contains details of and results from investigations of the delay incurred by frontage road traffic at intersections with freeway ramps. Four separate studies were conducted in medium-sized Texas cities during the summer and fall of 1986. All of the studies were conducted at locations with two moving lanes of frontage road traffic yielding to ramp traffic. The intent of the study was to derive relationships based on frontage road and ramp volume which could be used to compare and contrast the delays of vehicles on two-way frontage roads with delay experienced by vehicles on one-way frontage roads. This information would be useful when evaluating the impacts of converting a frontage road from two-way to one-way operation, and in better understanding the various trade-offs involved in such conversions.

BACKGROUND

Many miles of freeways in Texas have frontage roads. The predominate freeway design practice in Texas has been to connect the freeway entry and exit ramps to the parallel frontage roads, and not to the intersecting cross street. Instead of going directly from the freeway to the crossing street via the ramps, the vehicle proceeds off the ramp onto the frontage road, then along the frontage road to the crossing street. A 1979 Texas law requires that all frontage road traffic yield to both entry and exit ramp traffic. The frontage road vehicles potentially encounter some amount of delay each time one yields to an entry or exit ramp vehicle.

Almost all Texas frontage roads are one-way in large urban areas. Outside of the developed urban areas, the frontage roads usually have two-way traffic. Over a period of time, many types of land development will often occur along the frontage roads, taking advantage of the access and mobility provided by the freeway frontage road system. Such development includes commercial, industrial and residential. This occurs in both medium-sized towns and in the developing fringe of larger urban centers. Traffic volumes along an undeveloped frontage road are usually low, but subsequent land development creates increased traffic volumes.

Two-way frontage roads attract development, leading to suburban and urban traffic situations with increasing safety and congestion problems. The traffic situations created by higher volumes on two-way frontage roads include congestion at frontage road intersections with crossing streets, and a potential for accidents where the freeway ramps have the right-of-way when intersecting with the two-way frontage road.

Safety and operational considerations have caused the State Department of Highways & Public Transportation (SDHPT) to consider converting segments of two-way frontage road to one-way operation. Such conversions have been met on occasion with resistance from various segments of the public. When a frontage road is converted to one-way, the relative locations of certain trip origins and destinations will require some drivers to proceed along the one-way frontage road on the opposite side of the freeway from his destination down to the next crossover, then turn left across the freeway and turn left again along the other one-way frontage road. This increased indirection caused by a one-way frontage road system is one source of objections to conversion.

The indirection which accompanies one-way frontage roads translates into increased travel times. But there are other sources of delay on freeway frontage roads, and some of these delays may be greater with two-way systems than with one-way. When frontage roads are converted to one-way operation, the increased travel time due to indirection may be somewhat offset by decreases in other sources of delay. Vehicles may be delayed

1. at ramp terminals,
2. at intersections with streets which cross over or under the freeway main lanes,
3. at T-intersection with local side streets, and
4. at intersecting driveways.

As far as the ramp terminal--frontage road intersections are concerned, there are more sources of delay with a two-way frontage road system than with a one-way frontage road system. All of the traffic on a one-way frontage road moves in the same direction as the traffic on the adjacent freeway main lanes. Flows on a one-way frontage road diverge at an entry ramp, so no yielding manuevers are required; yielding is required where the exit ramp converges with the frontage road. A part of the traffic on two-way frontage road moves in the same direction as the traffic on the adjacent parallel freeway main lanes, but another portion moves in the opposite or "contraflow" direction. Traffic flows on two-way frontage roads which are moving in the same direction as the main lanes must yield at junctions with exit ramps, just as in the case of one-way frontage roads. But additional yielding situations arise with two-way frontage road operation; i.e., the contraflow traffic must yield at the intersection with both the exit ramp and the entry ramp.

A given one-way, two-lane frontage road will have two lanes moving in the same direction at intersections with crossing streets, and no opposing movements across the intersection. The left turning vehicles on one-way frontage roads will not face oncoming through traffic to which they must yield. If the same frontage road were two-way, there would only be one lane for a given approach at the intersection, and there would be opposing left, through, and right movements from the opposite side of the intersection. There is a greater potential for delay as left turning vehicles wait and the following traffic queues.

Since the frontage road would normally have the right-of-way at T-intersections of side streets and drives, the delay results when a contraflow direction vehicle wishes to turn left into the side street or drive, and following vehicles queue until the left-turning driver finds a suitable opening in oncoming traffic in which to make the left turn. This manuever and the resulting delay would not take place on a one-way frontage road.

PREVIOUS WARRANTS

As traffic volumes increase on a two-way frontage road, safety and operational concerns may increase. The previous Texas Transportation Institute (TTI) Study 288 by Woods and others (1, 2, 3, 4) funded by the SDHPT examined accidents, volumes, and type of surroundings (rural, urban, or intermediate). The study recommended the following warrants for conversion from two-way to one-way operation:

1. Volume Warrant
   Rural: 7,500 VPD (total of both frontage roads)
   Intermediate: 6,000 VPD (total of both frontage roads)
   Urban: 5,000 VPD (total of both frontage roads)
2. Accident Warrant
   20 accidents/mile per year, average of three years
   30 accidents/mile, for any one year

As frontage road volume increases, the potential for conflict at the ramp--frontage road intersection increases. This potential seems to translate into a higher number of accidents.

PURPOSE OF THE PROJECT

In an attempt to better define the problems associated with frontage road conversion from two-way to one-way operations and to ultimately propose effective solutions, SDHPT requested TTI to conduct Study 402, "Warrants for One-Way Frontage Roads." The study is a two-year effort and has the following five objectives:

1. Identify specific problems encountered by SDHPT in converting from two-way to one-way frontage road operations.
2. Identify the circumstances and the groups making requests for converting existing frontage road flow from on design condition to the other case.
3. Develop guidelines for examining typical frontage road operational situations from the traffic and business community viewpoints.
4. Develop strategies for ameliorating the positions of local interest groups that may conflict with proposed frontage road warrants.
5. Determine the traffic conditions required for converting existing two-way frontage roads to one-way operations to improve the level of service along the facility and to improve safety through accident and conflict reductions.

One portion of this project consists of identifying and analyzing the opinions of various interest groups. This was done by conducting 121 separate interviews in fifteen Texas cities. City staff, city council members, real estate appraisers, real estate and development interests, and owners and managers of businesses abutting frontage roads were surveyed. Research Report 402-1 contains the results of these surveys (5).

As previously stated, this report contains details of and results from investigations at the delay incurred by two-way frontage road traffic at intersections with freeway ramps. When included with other safety and operational considerations, information about the total delay for given ramp and frontage road volumes under two-way verses one-way operation can assist the engineer in evaluating the effects of frontage road conversion to one-way operation.

DELAY MODELS

A significant amount of research addresses the delay situations encountered at "normal" right angle (or near right angle), unsignalized intersections. While this body of information contains principles applicable to the situation of a freeway frontage road intersecting with a ramp terminal, the differences in geometric alignment and in right-of-way control lead to driver behavior unique to frontage road--ramp intersections.

The interaction of frontage road traffic with the ramp traffic stream may be viewed as a queueing system. As a queueing system, the ramp volume dictates the operation of the intersection. After the ramp traffic has passed through the intersection, a certain amount of time remains to serve the frontage road demand. When this "remaining capacity" value is approached by the actual frontage road demand volume, both the delay per vehicle and the fraction of frontage road vehicles delayed can be expected to increase.

The Intersection Viewed As a Queueing System

In order to proceed through the intersection, the frontage road driver must find an adequate opening in the ramp traffic stream. This phenomenen is analogous to a queueing system operation, in that the time spent waiting for an adequate ramp stream headway is time waiting to be "served." For some frontage road vehicles the time of service will be zero (0), in that they will be served instantaneously. It is expected that the heavier the ramp volume, the longer the average wait to be served, thus the larger the delay.

The queueing system service rate is the maximum number of frontage road vehicles per unit of time that can be expected to proceed through the intersection. The service rate varies with ramp volume. The higher the ramp volume, the lower the service rate, since the presence of a ramp vehicle precludes the servicing of a frontage road vehicle. For a given time interval, p is the frontage road flow rate (a) divided by the service rate (u).

$$p = a/u$$

The expected average queueing system delay in seconds per vehicle averaged over all frontage road vehicles is W (6).

$$W = [p/(1 - p)] / a, \text{ or } W = 1/(u - a).$$

Service Rate and Capacity

The parameter of service rate, u, must be known in order to utilize queueing theory. The service rate can be expressed as the ability to accomodate frontage road vehicles for a given time interval, or a capacity for frontage road vehicles.

The concepts of gap, lag, and block (7) help explain certain phenomena as a stream of traffic waits for an adequate opening in the traffic stream having the right-of-way. If we denote the roadway having the right-of-way as the main street (in this case the ramp), then a gap is the interval from the arrival of one main street vehicle to the arrival of the successive main street vehicle, as measured at the intersection. A lag is the interval from the arrival of a side street (frontage road) car at an intersection to the arrival of the next main street car. Raff (7) presented the concept of blocks (and antiblock), a block being that time during which no side vehicle can cross the main street. This block time includes a margin of safety in advance of an oncoming vehicle, as well as the time between successive main street vehicles if they are too closely spaced for the side street vehicle to cross. Oliver (8) called gaps only those headways greater than that needed for a side street vehicle to cross, while those less were said to be nongaps.

In one of the earliest theoretical traffic papers (9), Adams investigated pedestrian delay at unsignalized intersections. Adams determined (10) that the proportion of pedestrians delayed is

$$1 - e^{-qT},$$

where q is the main street flow rate (veh/sec) and T is the critical gap (sec).

A frontage road vehicle can be delayed by either having to wait for an adequate opening in the ramp traffic stream, or by queueing behind another yielding vehicle. If frontage road traffic is approaching the ramp--frontage road intersection in a random arrival manner, then the Poisson distribution can be used to predict the probability of arrival, and the exponential equation to predict time between arrivals. In order to utilize these equations, the parameters of minimum frontage road vehicular headway (F, in seconds), and average headway between ramp vehicles accepted by frontage road motorists (H), must be determined from the data.

Any ramp headway is either long enough so that it is accepted by the frontage road vehicle, or too short and rejected. A series of one or more of the inadequate headways will eventually be followed by an adequate headway. The probability of the occurrence of an adequate headway "h" is the probability of no ramp vehicle arrivals within a length of time of H for a given ramp flow rate qr.

$$P(h > H) = e^{(-H * qr)}$$

The total amount of time within any period, such as one hour, that will have ramp vehicle headways which are adequate for frontage road vehicles to cross the ramp stream is the amount of time in the period multiplied by the P (h > H). If this available time is divided by the headway at which frontage road vehicles will follow each other through the intersection, the remaining capacity available to frontage road flow for a given ramp flow can be found,

$$C = T * e^{(-H * qr)} / F$$

When dealing with two frontage road lanes in the same direction, modifications must be made. The presence of two lanes essentially doubles the value of the capacity C as calculated.

$$C = 2 * T * e^{(-H * qr)} / F$$

One would suspect that as the frontage road arrival rate approached the capacity allowed by the ramp flow rate, a larger proportion of the frontage road traffic would have to yield. A larger fraction of traffic being required to yield would be indicative of increased conflict potential between the ramp and frontage road traffic streams. More yielding also increases the opportunity for rear-end collisions with following frontage road traffic.

Total Delay

The capacity per hour is the same as service rate per hour. So long as the frontage road demand volume and service rate are in the same units, the ratio p can be found. Substituting into the earlier equation for W, the queueing system delay per vehicle is calculated.

There are other factors in addition to queueing system delay which contribute to the total delay incurred by frontage road traffic at ramp intersections. The time lost while returning to normal speed after having yielded contributes to total delay. Sluggish operation of a vehicle which has been delayed will cause extra delay to itself and vehicles behind it. The sum of these non-queueing delays and queueing system delays will equal total delay.

Vehicle Position Projection

As a means to estimate actual driving behavior, the projections of vehicle positions can be made. Drew first proposed using gap acceptance as a means to control freeway entry ramp merging activity (11). This strategy consisted of measuring and projecting the gaps in the freeway outside lane traffic stream at a point upstream of the entry ramp junction. A traffic control signal permitted ramp vehicles to enter the freeway when an acceptable gap was detected. This strategy involved the concept of "gap stability," or how much the size of the gap would change from the time it was measured upstream until the gap arrived at the entry ramp junction. Researchers found (11) the larger gap sizes showing more variation downstream of the measurement point than did the smaller gap sizes. As the downstream distance increased, changes in gap size became more pronounced; this change was found to be more prominent for the larger gap sizes than for the smaller ones. Over a fixed distance, the variations in gap size seemed to be normally distributed with a variance not related to the size of the gap. The size of the gaps in the merging area were shown to be highly correlated with the gap size some distance upstream, making it possible to predict downstream gap size from an upstream measurement with a degree of certainty.

II. STUDY PLAN

SITES

Four separate studies at three sites were conducted to measure delay to frontage road vehicles at intersections with freeway ramps. Each study examined one of the four following cases:

Study 1 - one-way frontage road intersection with exit ramp converging movement (used for comparison with two-way frontage road delay);

Study 2 - two-way frontage road intersection with exit ramp, converging movement;

Study 3 - two-way frontage road intersection with exit ramp, contraflow movement; and

Study 4 - two-way frontage road intersection with entry ramp, contraflow movement.

Each of the studies was conducted with a two lane frontage road, and one lane ramp. The studies were made during daylight hours with dry pavement.

A limited amount of information as to the actual volumes occurring on ramp and intersecting frontage roads was already available. From the available information, one study site for each case was chosen. Ideally, each study site would have exhibited a wide range of ramp and frontage road volumes, so the effects of various volumes could be studied.

Study 1 was conducted in San Marcos (SM), at the northbound I-35 exit ramp to SH 21. This frontage road has three lanes, but the outside lane was coned-off for the study. The exit ramp is a slip ramp. The roadway gradients are level.

Study 2 and Study 3 were conducted in College Station at the SH 6 southbound exit ramp to SH 30. The exit ramp is a buttonhook ramp. Study 2 is referred to as College Station Southbound (CS) since the frontage road traffic is flowing in a southward direction. Study 3 is referred to as College Station Northbound (CN). The ramp gradient and the contraflow direction gradient on the frontage road are slightly downhill. The converging flow movement encounters a moderate downhill grade.

Study 4 was conducted in Bryan (B) at the SH 6 entry ramp from FM 1176. The entry ramp is a buttonhook ramp. The ramp and frontage portion of the ramp approach are level; the frontage road contraflow approach is downhill.

The studies are summarized in the following Table 1.

TABLE 1
LIST OF STUDY SITES

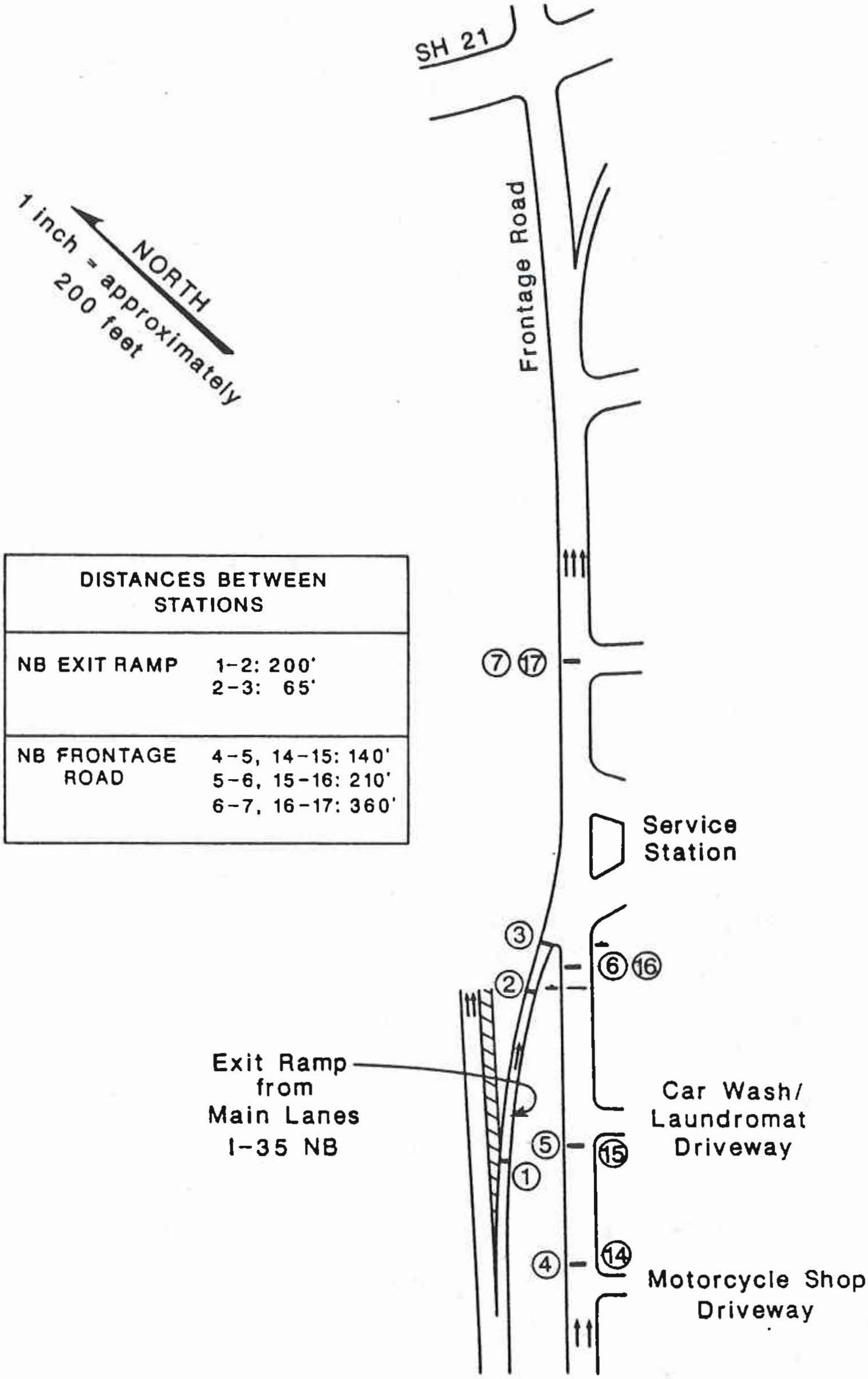
	Study Type	Ramp Location	Study Date
1.	One-way frontage road intersection with converging exit ramp (for comparison with two-way frontage roads)	San Marcos, I 35 nb exit to SH 21	July 25, 1986
2.	Two-way frontage road intersection with converging exit ramp	College Station, SH 6 sb exit to SH 30	Aug. 26, 1986
3.	Two-way frontage road intersection with contraflow (opposing) exit ramp	College Station, SH 6 sb exit to SH 30	Aug. 27, 1986
4.	Two-way frontage road intersection with contraflow (opposing) entry ramp	Bryan, FM 1179 sb entry to SH 6	Oct. 1, 1986

Schematic plan view drawings of the four study site layouts are shown in Figures 1 thru 4. A pair of photographs for each site are shown in Figures 5 thru 8. The upper photo of the pair shows the site with little traffic, while the lower photo shows frontage road traffic yielding to ramp traffic. The photographs were not made while the studies were being conducted.

DATA COLLECTION

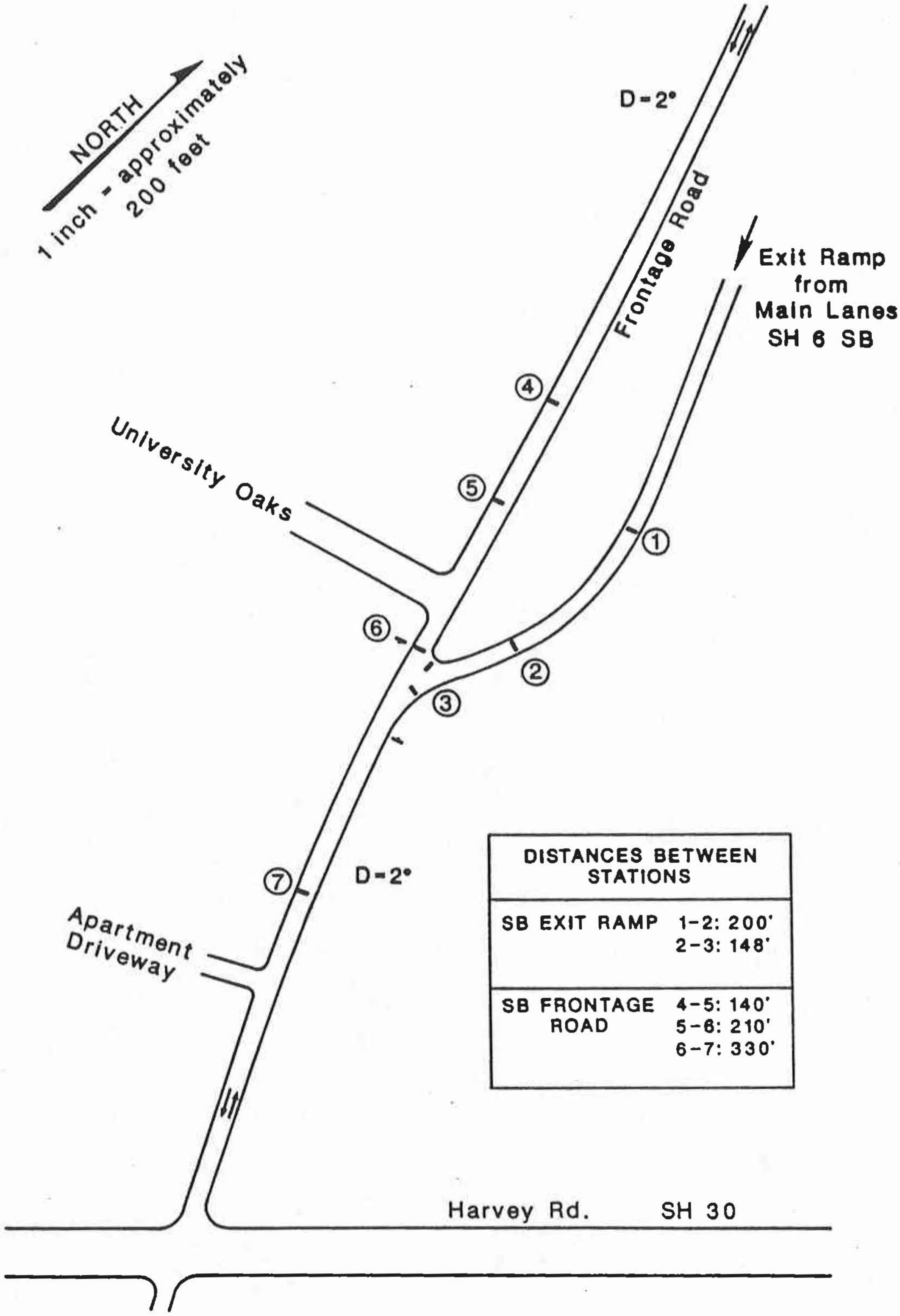
These studies found the amount of delay to the frontage road traffic by comparing the travel times of the vehicles which yielded to ramp traffic with the travel times of those vehicles which did not yield. A vehicle was categorized as yielding if it actually yielded to a ramp vehicle, or it was delayed or queued behind another yielding vehicle.

Similar strategies to obtain data were used at the four sites. The plan was slightly modified at each site to accommodate the peculiarities of the specific ramp--frontage road intersection type.

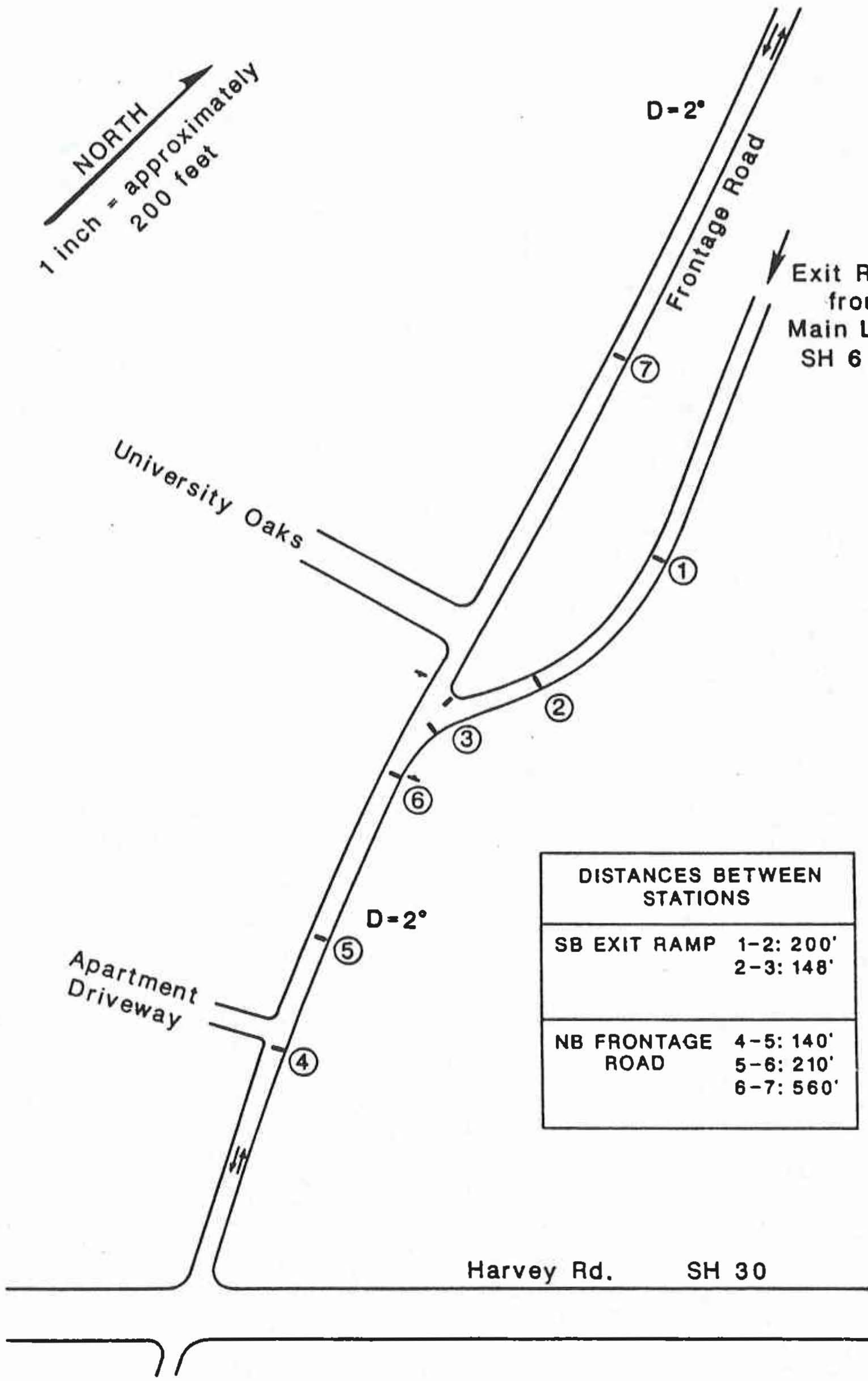


DISTANCES BETWEEN STATIONS	
NB EXIT RAMP	1-2: 200' 2-3: 65'
NB FRONTAGE ROAD	4-5, 14-15: 140' 5-6, 15-16: 210' 6-7, 16-17: 360'

STUDY 1 - SAN MARCOS STUDY LAYOUT
FIGURE 1

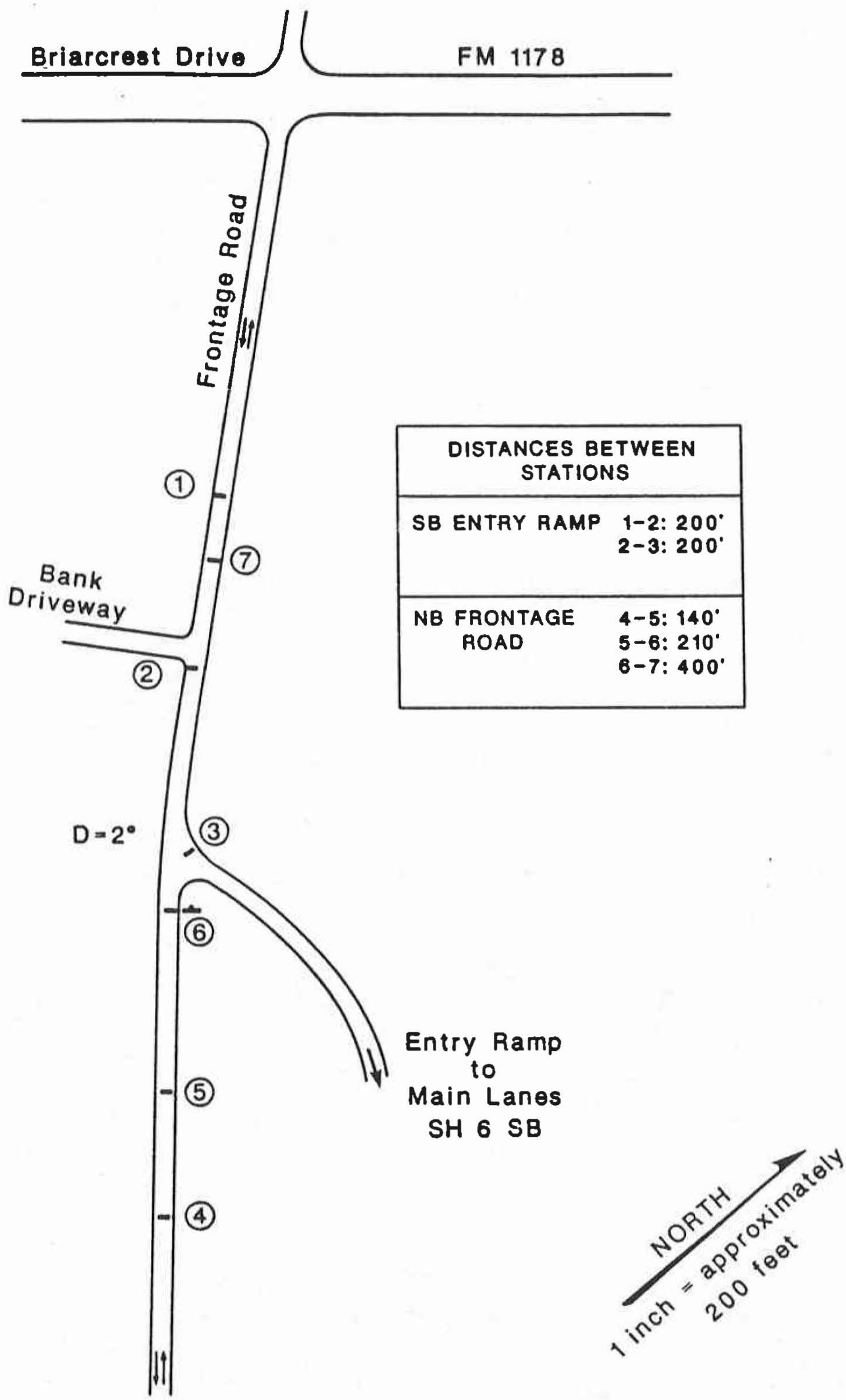


STUDY 2 - COLLEGE STATION SB STUDY LAYOUT
FIGURE 2

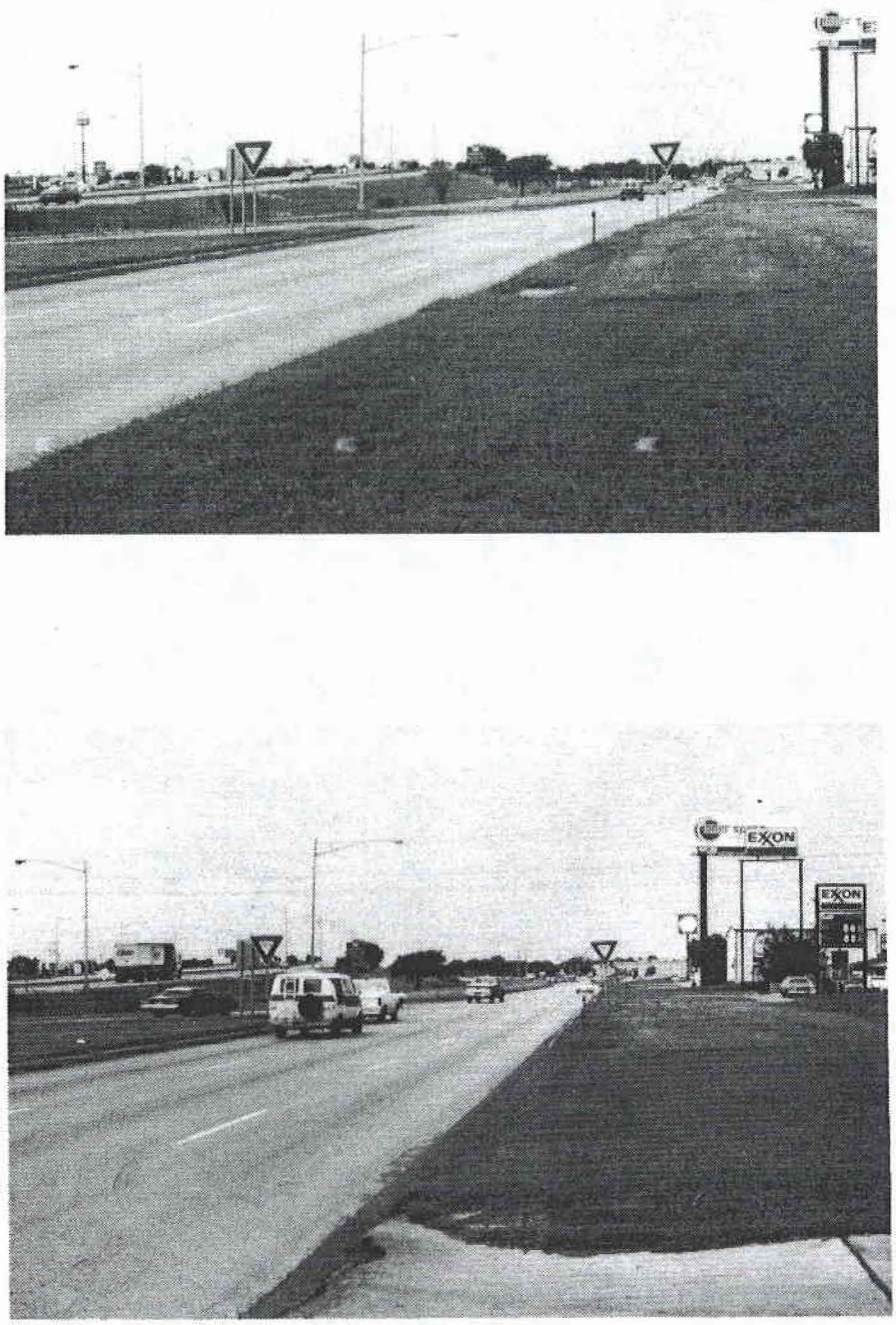


DISTANCES BETWEEN STATIONS	
SB EXIT RAMP	1-2: 200' 2-3: 148'
NB FRONTAGE ROAD	4-5: 140' 5-6: 210' 6-7: 560'

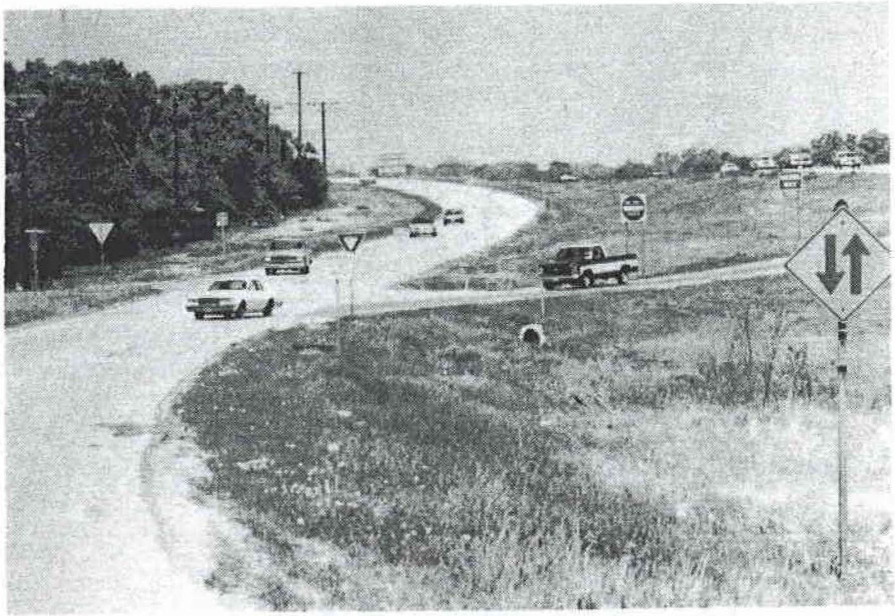
STUDY 3 - COLLEGE STATION NB STUDY LAYOUT
FIGURE 3



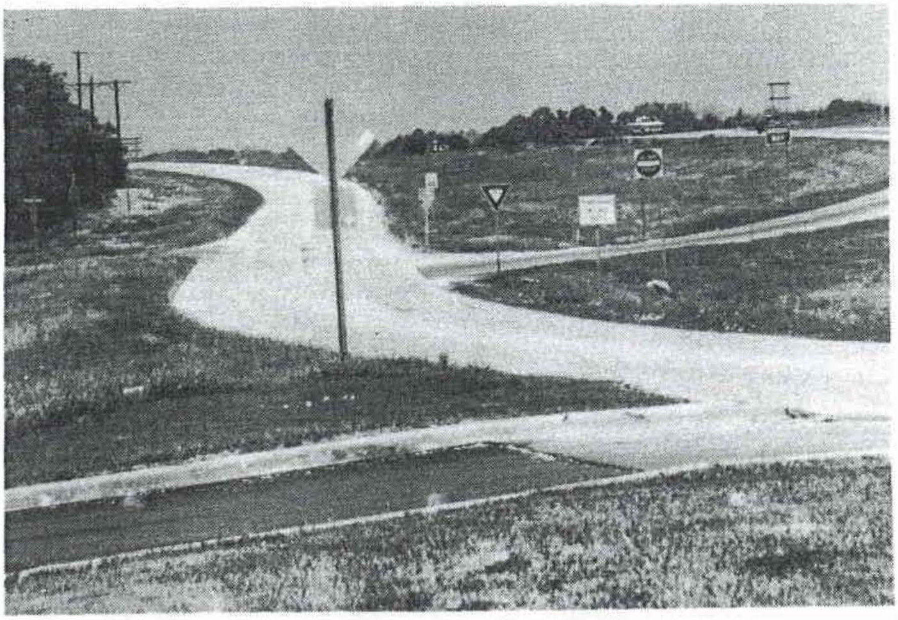
STUDY 4 - BRYAN STUDY LAYOUT
FIGURE 4



STUDY 1 - SAN MARCOS SITE PHOTOGRAPHS
FIGURE 5



STUDY 2 - COLLEGE STATION SB SITE PHOTOGRAPHS
FIGURE 6



STUDY 3 - COLLEGE STATION NB SITE PHOTOGRAPHS
FIGURE 7

STUDY 4 - BRYAN SITE PHOTOGRAPHS
FIGURE 8

Field Study Procedure

Preliminary field investigations with radar guns were made to determine the approximate points at which most ramp and frontage road vehicles modified their speeds when approaching the ramp--frontage road intersections. These locations (called stations) were then defined by measuring their distances away from the intersection. The initial station on both the ramp and the frontage road were chosen to reflect normal vehicular speed, before any anticipatory slowdown began. The second stations were positioned to be at the end of the anticipatory slowdown, and the beginning of a greater rate of deceleration, if a vehicle experienced such a deceleration. The area between the first two stations is one of slight deceleration.

The third station on the ramp was chosen to provide an approximate time of passing through the intersection. These stations were set back from the actual intersection, because wide variations of vehicle travel paths were observed to occur in the immediate intersection area.

The third frontage road station was placed in the area where those vehicles which did stop came to rest. The final frontage road station was placed where the yielding vehicles seemed to have recovered from their slowdown, and resumed normal speed. The patterns of behavior varied among individual vehicles, so these stations were set in a way to approximate median behavior.

The speed and real time of passage of both ramp and frontage road vehicles were recorded at a series of Tapeswitch brand sensing devices installed on the roadway surface. A pair of these strips was placed perpendicular to the traffic flow, and separated by a known distance (five feet). The pressure from the front tires of passing vehicles actuated each of the Tapeswitches in the pair, providing the needed time and velocity.

The field study team parked a minivan at a roadside location suitable for viewing the entire study area. With a portable generator supplying electrical power, team members connected the wiring from the sensing devices and inductive loops to minicomputers placed inside of the van. The van acted as a field observation post.

Special computer software, which had already been developed by researchers at Texas Transportation Institute, permitted the recording of data into computer memory for later analysis. In addition to the actuation from the field hardware, two observers each operated an eight-button pushbutton bank to code in the real time of pre-specified traffic events. The observers tracked vehicles from station to station, and recorded other events such as turning movements, by means of these pushbutton impulses.

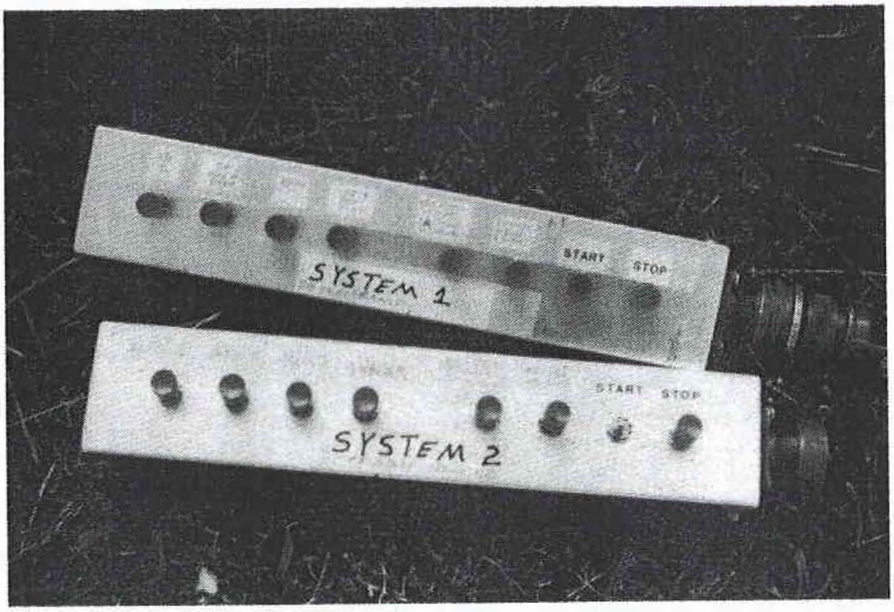
The resulting pattern consisted of three sequential stations along the ramp. The first tapeswitch pair was placed 400 feet in advance of the point at which the exit ramp geometry was fully merged with the frontage road geometry and called station R1. The second ramp pair was called R2 and located 200 feet after R1. The third ramp pair was installed as close as possible to the intersection, and called R3. An additional single Tapeswitch was placed at the exit ramp which intersected with a two-way frontage road, to record the time of passage for those vehicles which made a sharp right turn, of necessity at low speed.

A series of four Tapeswitches was installed along the frontage road. The pair initially encountered by frontage road traffic, called F4 or F14, was installed 350 feet in advance of where the frontage road traffic stopped when it did yield. The second pair was 210 feet in advance of this point, and called F5 or F15. The third pair, F6 or F16, was placed at the stopping area. In addition to the sensing strips, an inductive loop to sense the presence of a vehicle was installed immediately past the strips. The final pair, F7 or F17, was installed downstream of the ramp--frontage road intersection, where the yielding frontage road vehicles seemed to have resumed speed.

Stations F14, F15, F16, and F17 were present only at the San Marcos study site. These stations were located on the outer frontage road lane, while the usual nomenclature of F4 through F7 was used to denote the stations on the inside lane. Except when specifically referring to the San Marcos frontage road outside lane stations, references to frontage road stations will be by F4 through F7, and the application to the adjacent F14 through F17 station will be implied.

At each study site, field hardware was installed on the roadway and wired to the van site. The installation of the field hardware and testing of the circuits required most of a day. The data gathering lasted for one day for each of the four traffic patterns. Hardware removal after completion of data gathering required about half of a day at each location. Photographs in Figure 9 show a preliminary radar check (upper) and an installation of the sensing strips (lower). The photographs in Figure 10 show the van as a field observation site (upper), and the pushbutttons (lower).

ACTIVITIES PRIOR TO DATA COLLECTION
FIGURE 9



DATA COLLECTION FIELD OPERATIONS
FIGURE 10

Data Processing

The impulses from the field hardware to the portable computers required two stages of processing to yield readable output. The output was in the form of a listing of vehicle number (i.e., in sequence, beginning with "1"), real time of passage, and velocity. In addition, the frontage road station with a loop had a record of the dwell time on the loop. A separate output existed for each station at a site.

These separate station outputs were then matched with outputs of the adjacent stations so as to create a tracking of the vehicle as it proceeded from the initial to the final station. A Lotus 1-2-3 spreadsheet was employed to do this. The inductive loop data aided the tracking process. By showing how much time a given frontage road vehicle spent at the yield point, the inductive loop data helped explain some of the lengthy elapsed times between F6 and F7.

After the data at each ramp and each frontage road lane were arranged so that each vehicle could be tracked, the data sets were transferred to a mainframe computer at Texas A&M University. There, the programming and statistical capabilities of the Statistical Analysis System (SAS) were utilized to evaluate the travel patterns of the approximately 8000 vehicles observed in the four studies.

RELATED OBSERVATIONS DURING FIELD STUDY

During the time that the study sites were being reconnoitered and during the studies themselves, certain traffic activities were observed. Three types of activities were photographed and are discussed: right turns off of exit ramps onto two-way frontage roads, exit ramp traffic cutting across the opposing (contraflow) lane on a two-way frontage road, and frontage road yielding patterns at an entry ramp.

Right Turn From Exit Ramp

With the presence of two-way frontage roads, opportunities for right turns off of exit ramps and onto entry ramps exist. Both of the activities were observed during the studies; a vehicle turning right off of an exit ramp into the contraflow lane of a two-way frontage road was photographed. Given the skewed angle of intersection between many exit ramps and two-way frontage roads, the right-turning ramp traffic has to slow to a speed lower than that of most ramp vehicles. This speed differential was observed to cause brief congestion on the ramp. It would also seem to increased the potential for rear-end collisions on the ramp. The upper photo of Figure 11 shows such a right turning movement from an exit ramp.

The lower photograph shows rain runoff ponded in the rut created by right turning exit ramp traffic. At this location the vehicles turning right have tracked off of the asphaltic surface onto the softer crushed stone gore area, creating the ruts.

Exit Ramp Traffic Cutting Across Lane

In what appears to be an effort to maintain a higher exit ramp speed by increasing the radius of the vehicle path, some exit ramp vehicles were observed cutting across the oncoming (contraflow) lane of a two-way frontage road. Figure 12 is a photograph of an exit ramp vehicle at the junction of a buttonhook exit ramp with a two-way frontage road. It is cutting across the painted yield bar pavement marking of the oncoming lane.

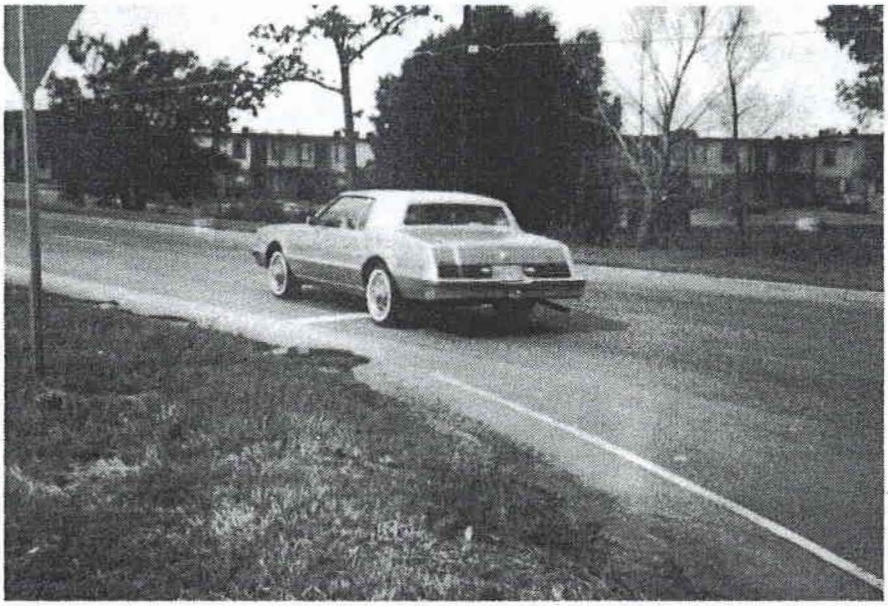
Sometimes the contraflow frontage road traffic was observed stopping well in advance of the yield bar so as not to encroach upon this area used by the cutting-across exit ramp vehicles. At a few other times it seemed that the presence of a stopped frontage road vehicle at the yield bar caused an exit ramp vehicle to slow in the intersection in order to "swing around" the frontage road vehicle.

Frontage Road Yielding Patterns

A few instances of what were judged to be "failure-to-yield" violations were observed at ramp--frontage road intersections. With a two-way frontage road, such a violation could lead to a head-on collision. With a one-way frontage road, a sideswipe or rear-end accident would be likely in the event of a violation.

At the exit ramp intersection with the two-way frontage road, an instance was observed where the converging (merging) frontage road vehicle did not yield to the ramp vehicle. Not to be denied, the ramp vehicle proceeded to travel the wrong way in the contraflow lane along side the unyielding frontage road car. They proceeded together until the ramp vehicle was confronted with an oncoming vehicle in the contraflow lane. Then the ramp vehicle slowed and pulled in behind the frontage road vehicle.

EXIT RAMP RIGHT TURN
FIGURE 11

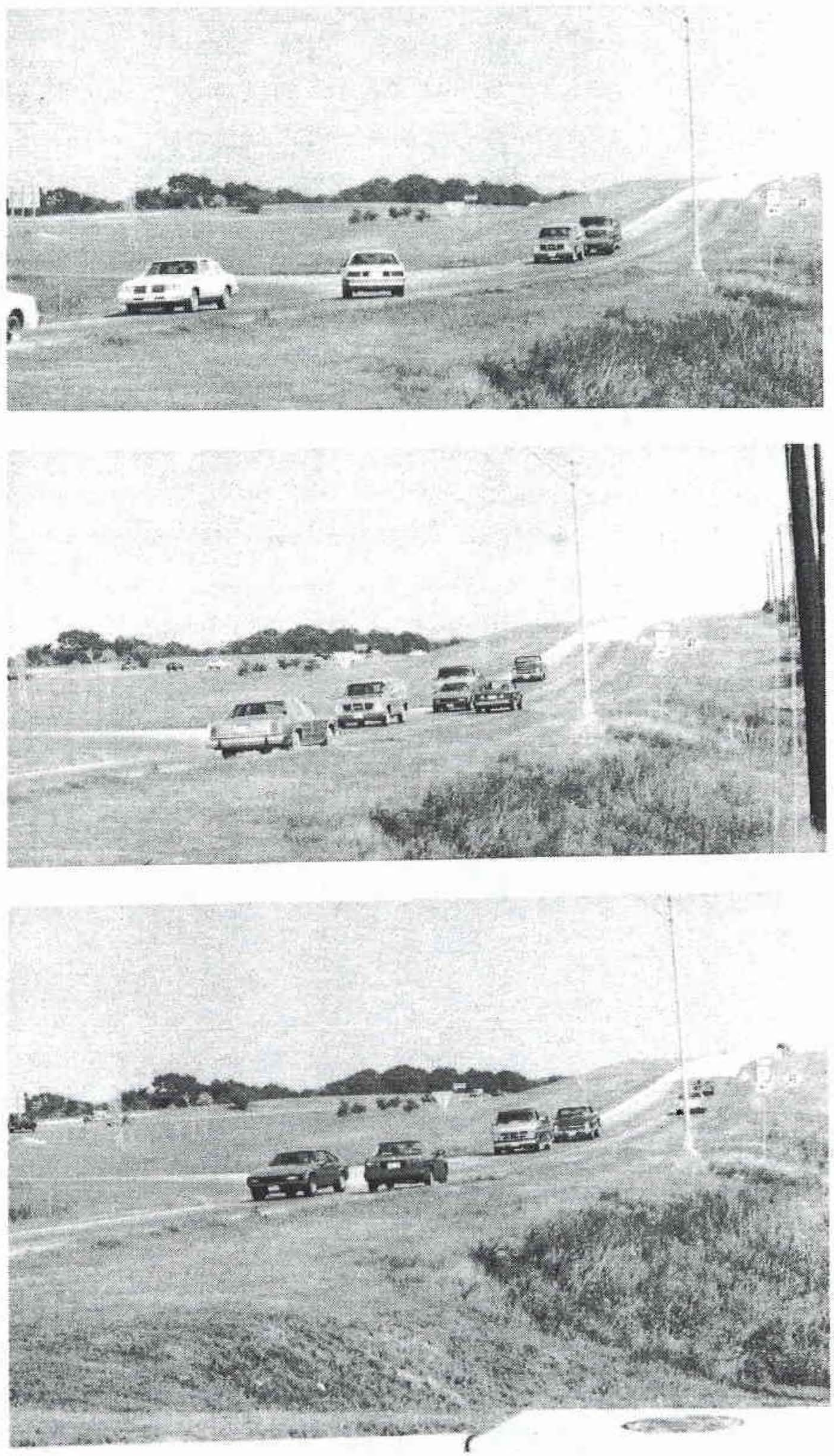


EXIT RAMP VEHICLE CUTTING ACROSS ONCOMING LANE
FIGURE 12

The three photographs of Figure 13 show a sequence of events at an entry ramp--two-way frontage road intersection. All three photographs were shot within a one minute time span. The first photo shows a ramp vehicle swinging wide as it begins to enter the ramp. This maneuver was due to the light colored frontage road vehicle accepting an inadequate gap in the ramp traffic stream, leaving little clearance for the entering ramp vehicle.

The second photo shows oncoming frontage road vehicles yielding; the lead car doesn't know whether the vehicle proceeding from left to right in the photo will enter the ramp or stay on the frontage road. The oncoming frontage road driver could have proceeded if he had known the intention of the driver having the right-of-way; but not knowing what path would be chosen, he was unnecessarily delayed.

The last photo shows a more blatant version of events similar to those in the first photo. The oncoming sports car did not yield and blocked the path of the car trying to enter the ramp. In the original photo the flash of a left turn signal is visible at the rear of the ramp entry vehicle, so the sports car driver had notification of the intent of the driver having the right-of-way.



TWO-WAY FRONTAGE ROAD YIELDING PATTERNS
FIGURE 13

III. DATA ANALYSIS

In order to find the necessary numerical inputs for model formulation, a number of separate analyses were made with the data. These analyses were used to define various characteristics of motorists and vehicles at ramp--frontage road intersections.

DRIVER PROJECTION OF LAGS AND GAPS

As a frontage road vehicle approaches the intersection with the freeway ramp, the driver may encounter a queue composed of preceding vehicles which have formed behind a vehicle yielding to ramp traffic. If so, it will have to join the queue. If not, the driver must decide in advance whether to yield to the ramp vehicle or to proceed unimpeded. The frontage road driver mentally projects his position and speed to the intersection, and also mentally projects the positions and speeds of ramp vehicles to the intersection. The frontage road driver may feel that he can proceed through the intersection based on these mental projections of vehicle positions, without having either a collision or a "close-call." This frontage road driver is said to have accepted a lag; he considered the time from his projected arrival at the intersection to the arrival time of the next arriving ramp vehicle to be adequate. However, if the frontage road driver yields by slowing or stopping, he has rejected the lag. Such a yielding frontage road driver is then presented with one or more gaps between successive ramp vehicles. The frontage road driver who rejected the lag must wait until he finds an acceptable gap, i.e., a period of time between the projected arrival of successive ramp vehicles at the intersection which he deems adequate. The first gap presented may be accepted, or it may be rejected and a later gap finally accepted.

The projection of vehicle position is a key concept to this study. At ramp--frontage road intersections where the frontage road traffic yields to ramp vehicles, the initial acceptance or rejection of a ramp vehicle is based on the projections of future positions to the intersection. (If there are preceding frontage road vehicles queued up and stopped at the intersection, then this concept does not apply, since the queue ahead of the frontage road vehicle must clear before the frontage road vehicle can decide to proceed or to yield). All frontage road vehicles which are stopped near the stop line, whether due to rejecting the lag presented to them, or due to having a preceding queue clear out, must project the position of oncoming ramp vehicles to the intersection to determine whether to proceed or to wait.

Projections of both ramp vehicles and frontage road vehicles were employed in these data analyses. These projected arrivals at the intersection may or may not coincide with the actual arrival times.

CONCEPTS FOR DATA ANALYSIS

A number of parameters were calculated from the values recorded in the field. These parameters were utilized in subsequent data analyses.

Ramp Traffic Projected Arrival Time

For this study, the arrival time of each ramp vehicle at the ramp--frontage road intersection was projected based on time and speed at two upstream recording stations. Thus it is assumed that the upstream behavior of the ramp vehicles is a valid predictor of ramp vehicle behavior at the intersection. There is a two-fold need to project ramp vehicles position. One, the frontage road motorist must make a similar projection, thus actual behavior is simulated. Two, some ramp vehicles come to a near stop at the intersection; it is possible that many of the frontage road drivers would have proceeded instead of yielding if they knew in advance how long the gap would actually be, but they yielded because they were acting on the gap anticipated from upstream ramp behavior.

The projected speed of the ramp vehicle at the third ramp station is:

for $V1 > V2$: $VP3 = V2 - (V1 - V2) * L23 / L12$, or

for $V1 \leq V2$: $VP3 = V2$,

where

V1 = velocity at ramp station 1, (see Figures 1-4)

V2 = velocity at ramp station 2,

VP3 = projected velocity at ramp station 3,

L12 = distance between ramp stations 1 and 2, and

L23 = distance between ramp stations 2 and 3.

A minimum "floor" of 15 mph was set for VP3. The projected time of ramp vehicle arrival at the intersection for the three exit ramps is

$$TPIF = T2 + L23 / [(V2 + VP3)/2] + L3I / VP3$$

where TPIF = projected real time of arrival at the intersection,

T2 = real time at ramp station 2, and

L3I = distance between ramp station 3 and intersection.

The one entry ramp situation (Bryan) required a slightly different configuration of Tapeswitches to reflect the different nature of the entry ramp geometry as compared with the exit ramp. The projected time of ramp vehicle arrival at the intersection for the entry ramp is

$$TPIF = T2 + L2I / ([\ V2 + (\ V2 - L2I * (V2 - VP3) / L23\ )] / 2\ ).$$

Frontage Road Traffic Projected Arrival Time

The arrival time of frontage road vehicles at the ramp--frontage road intersection was projected from the speed of the vehicle at F4 and F5. This projected time is:

$$TPIF = T5 + (L56 + L6I) / [(V4 + V5) / 2].$$

The projected time of arrival at the intersection is based on an average speed between F4 and F5.

Measures of Travel Time

Two measures of travel time were used to evaluate frontage road vehicle behavior. The first was the total travel time required for a vehicle between the beginning and the ending of the study area. This time between F4 and F7 was called TIMET47.

The second measure was the predicted total travel time between F4 and F7, found by dividing the known distance between F4 and F7 by the average of V4 and V7. This time was called TIMEP47.

Determination of Vehicle Delay

Some portion of the frontage road traffic will not encounter any ramp vehicles near the ramp--frontage road intersection, and can proceed unimpeded. These vehicles incur no delay.

Other frontage road vehicles will either have to yield the right-of-way to ramp vehicles at the intersection or will queue behind such a yielding vehicle. The amount of time that these vehicles are delayed is their actual travel time in excess of a hypothetical travel time had there been no yielding or queueing.

actual travel time - hypothetical travel time = travel time delay

This hypothetical time will never be known, since the event of proceeding through the intersection without delay did not occur for any delayed vehicle. However, it can be approximated.

Similar methods of delay calculation were used with both the TIMET47 and the TIMEP47 parameters. The TIMEP47 parameter was converted to TDIFF47 for delay calculation purposes. TDIFF47 is the difference between actual elapsed travel time between F4 and F7 and the travel time predicted by averaging the velocities at F4 and F7.

$$TDIFF47 = (T7 - T4) - L47 / [(V4 + V7) / 2].$$

The TIMET47 and TDIFF47 values for those frontage road vehicles which were not delayed were summed and averaged. These average values became adjustment factors to determine the hypothetical "non-delay time" for the delayed vehicles. This estimator may be low or high for any given delayed vehicle, but should be reliable when considering averaged delays. Delay was not calculated for each individual vehicle, but rather was calculated for all of the vehicles that fell within a specified time period, such as for a five minute period, in the following manner:

$$\text{average(avg.) time per each yield vehicle} = \frac{\text{sum of yield TIMET47}}{\text{number of yielding vehicles}}$$

$$\text{avg. delay per each yield vehicle} = \text{avg. time yield} - \text{avg. time nonyield}$$

$$\text{avg. delay for all} = \frac{\text{avg. delay per yield} * \text{number of yield}}{\text{number of yield + nonyield}}$$

A value of delay based on TDIFF47 was calculated in a similar manner.

PRELIMINARY ANALYSIS OF DELAY

The data were originally aggregated into five-minute periods in order to perform analyses. It was suspected that random effects of the five-minute periods were creating a highly variable response, that is, a highly variable delay for similar traffic conditions. It was felt that fifteen-minute periods would overcome this instability, but that there were an insufficient number of periods when the data was subdivided into longer time groupings. As a way to overcome this problem, "traveling" fifteen-minute periods were used. These traveling fifteen minute periods were formed in the same way as the five-minute periods, with one exception. Instead of beginning the j+1 fifteen-minute period at the end of the preceding or the period, the j+1 period was set to begin five minutes after the start of the j-th period. Thus the time periods partially overlap, and any given time period shares some data with adjacent periods. This means that the time period data are not entirely independent of each other. The summaries of fifteen-minute data for the four studies are in Appendix B.

QUEUEING THEORY MODELING

The data were analyzed to determine the average accepted headway between ramp vehicles (H), and the minimum normal car following headway between frontage road vehicles (F). The acceptable headways were found by grouping headways in one-half second intervals and then finding the fraction of those headways accepted. As the headway sizes progressed from smaller to larger, the fraction accepted grew larger. A "trend line," i.e., regression, was mathematically constructed in the region showing a change from "more unaccepted headways" to "more accepted headways." This regression was used to estimate the headway size that was accepted 50% of the time. The minimum normal frontage road following headways were found by evaluating the smaller frontage road vehicle headway sizes, and are given in Appendix A.

Capacity and Delay Calculations

Since the ramp flow rate for each 15 minute period (qr) was known from field data, the portion of time which had sufficient headways to allow passage of a frontage road vehicle could be calculated. This portion divided by the car following headway is the potential capacity (C).

$$C = T * e^{(-H * qr)} / F$$

Converting C to units of vehicles per second (u), and the frontage road flow rate for the given 15-minute period to vehicles per second (a), the following calculations were performed to determine queueing system delay (W).

$$p = a / u$$
$$W = [p / (1 - p)] / a, \text{ or } W = 1 / (u - a)$$

Since there are other components of total delay, such as that due to sluggish behavior of traffic and due to time lost while resuming speed after having yielded, the total delay per vehicle is greater than the queueing delay alone. The actual observed total delays were regressed against queueing system delay to derive a predictive model of the expected delay per vehicle for each of the four study site situations.

Delay Validation Procedure

In order to validate the delay models, an estimated observed queueing delay was compared with calculated W values. This estimated observed queueing delay was determined by subtracting non-queueing delay from total observed delay. The calculation of non-queueing system delay involved an iterative estimation procedure. The total delay per frontage road vehicle for each 15 minute period (DLA) was known from the field data. The proportion of this traffic which had to yield (or slow down due to the yielding of a proceeding vehicle) was also known (FD). An initial estimate of the nonqueueing delay per vehicle was made (DNQest), following by an estimate of queueing delay (West).

West = DLA - DNQest * FD

A series of calculations, previously described, could then be made.

Pest = a / (a + 1 / West)
Cest = Qf / Pest

The actual measured delays will fluctuate for similar volumes, due to the effects of chance and to error in recording and processing the data. But if the underlying theory is valid, some degree of agreement between the calculated and field estimated p and C is expected. The value of DNQest was adjusted to bring calculated and estimated values of p and C toward agreement.

Fraction Delayed Calculation

In addition to delay, the fraction of the frontage road traffic which was required to yield (or to slow down due to the yielding of preceding vehicles) was evaluated. Relationships between the number of frontage road vehicles observed to yield during the field study and p (arrival rate/service rate) were derived. The fraction of vehicles delayed can be used as a qualitative measure to compare a "level of inconvenience" between two-way and one-way frontage road operations.

IV. RESULTS

Observed ranges of volumes, delays to frontage road vehicles, and fractions of vehicles delayed are shown in Table 2. Typical data are listed for low and moderate off-peak conditions, and for peak volumes. Formulation of the delay models was based on these data.

From Table 2 one can see, for instance, that typical peak volumes for Study 3 occurred around 5:15. During this 15-minute interval, each frontage road vehicle was delayed an average of 21.7 seconds, and 82% of the frontage road vehicles were delayed.

The prediction of delay per vehicle is essentially a three-step process. From known ramp volumes (and known frontage road car-following headway and gap acceptance headway parameters), the capacity to accommodate frontage road flow can be estimated. Then the frontage road capacity is translated to a queueing system service rate, and finally the overall delay is derived.

FRONTAGE ROAD CAPACITY

Two ways were used to determine frontage road capacity which is possible with a given ramp volume. The previously presented exponential headway capacity formula was one method. The second way involved using a regression to approximate the more involved exponential headway formula. The second method provides a simple, quick alternative to using the exponential headway formula.

An approximation of the number of frontage road vehicles per hour which can be accommodated for a given ramp flow is shown in Figure 14. Each of the four lines of this figure reflects results from one of the four study cases. The lines were fitted to the exponential headway capacity equation curves within the ranges of study data. Since the curves were rather flat except when near an axis, the listed regression equations provided a simple capacity estimate with R2 values near 0.99. Each of the lines approximates the possible volume per frontage road lane on a two-lane frontage road. To find the predicted capacity for both lanes of a two-lane, one-way frontage road, multiply the y-axis coordinate by two.

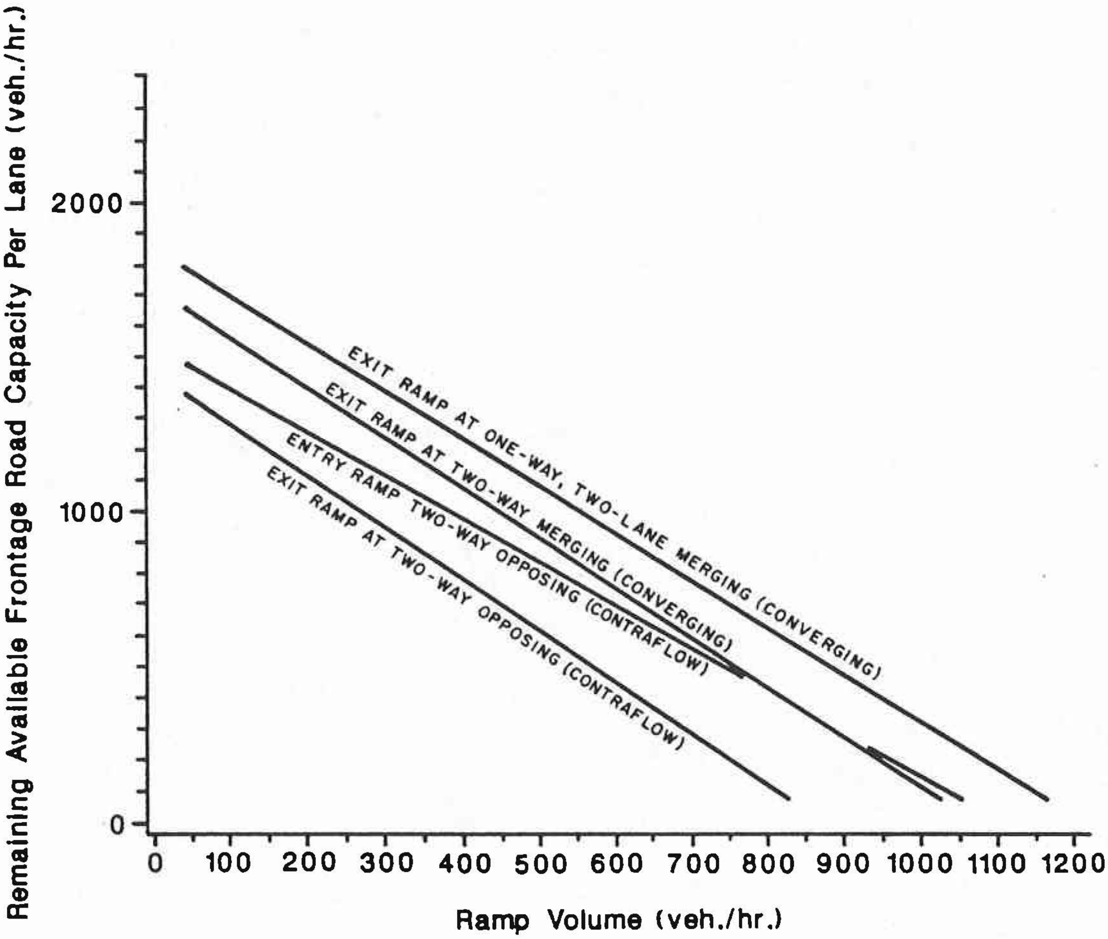
The capacity equations for three of the four cases result in lines that are roughly parallel. The exception is the line for Study 4, entry ramp at two-way opposing (contraflow). It is possible that additional data would alter the line to be generally parallel with the other three. Note that for

TABLE 2
TYPICAL OBSERVED FIELD DATA

all data for 15-minute intervals,
with volume converted to one-hour equivalents

Study >	1:SM	2:CS	3:CN	4:B
Off-peak, low volume				
ramp (veh./hr.)	192	244	236	308
frontage road (veh./hr.)	320	88	96	112
total delay (sec./veh.)	1.6	2.9	6.8	3.8
fraction delayed	0.21	0.32	0.38	0.32
time observed	8:15	8:35	10:20	3:15
Off-peak, moderate volume				
ramp (veh./hr.)	252	360	408	540
frontage road (veh./hr.)	408	216	200	168
total delay (sec./veh.)	1.9	4.1	10.1	7.9
fraction delayed	0.27	0.46	0.62	0.55
time observed	4:00	4:10	12:05	4:55
Peak volume				
ramp (veh./hr.)	304	484	564	748
frontage road (veh./hr.)	532	232	272	196
total delay (sec./veh.)	2.5	6.8	21.7	10.9
fraction delayed	0.30	0.60	0.82	0.68
time observed	4:50	12:15	5:15	5:15

Note: All times are between 8 a.m. and 6 p.m.



Study 1 (exit ramp, one-way converging)	C = 1858 - 1.5259 Qr
Study 2 (exit ramp, two-way converging)	C = 1724 - 1.6120 Qr
Study 3 (exit ramp, two-way opposing)	C = 1444 - 1.6564 Qr
Study 4 (entry ramp, two-way opposing)	C = 1535 - 1.3852 Qr

where C is approximate hourly per lane capacity, and Qr is ramp volume.

RAMP VOLUME--FRONTAGE ROAD CAPACITY RELATIONSHIPS
FIGURE 14

Study 4, the entry ramp volume includes all vehicles which approached the ramp from the converging direction, whether they actually entered or continued along the frontage road.

The capacity equations indicate that as ramp volume approaches zero (0), the ramp traffic will have minimal influence on frontage road operation. The capacity of the frontage road at the ramp--frontage road intersection will approach the capacity of the frontage road itself. Anticipation of yielding may keep maximum capacity at the intersection with the ramp somewhat below capacity on adjacent sections of the frontage road. On the other hand, high ramp volumes will leave few adequate openings for frontage road vehicles to proceed, and frontage road capacity will tend towards 0. The predictive equations and lines probably should not be utilized with low or high ramp volumes; to reflect this, the lines on Figure 14 were omitted for ramp or frontage road volumes approaching 0.

IDENTIFICATION OF OUTLIERS

Plots of field measured delay against calculated queueing system delay are presented in Appendix C. Even though a positive correlation appears to exist, a good deal of variation or scatter is present. A review of the plots and tabular data of Appendix B indicates that some of the data points are "outliers," i.e., behaving in a way dissimilar to the majority of the data.

For Study 1, data groups 6, 7, 8, 9, 10, and 18 have frontage road capacities calculated from field data that are improbably high. They ranged from 4836 to 6276 vehicles per hour for both lanes. This leads one to suspect that the delays per vehicle were unusually low for the given volume combinations during these time periods. Conversely, the delay for groups 13 and 14 appear to be extraordinarily high for the given volumes; examination of delays for other similar volumes show this to be the case.

In the Study 2 data, the frontage road capacities calculated from field data for groups 7, 8, 12, 13, 29, and 37 were greater than 1775 vehicles per hour. The lowest ramp volume for these groups was 276 vehicles per hour. Thus over 2050 vehicles per hour-lane were estimated to be leaving the ramp--frontage road intersection, an unlikely situation.

The Study 4 data had two groups showing field-data-calculated volumes greater than 1800 vehicles per hour, groups 12 and 13. Groups 4, 5, and 6 showed a delay per vehicle much in excess of delay for other similar volumes, and groups 28 and 28 had delays much lower.

The Study 3 data was very well behaved. No outliers were apparent.

FRONTAGE ROAD DELAY

Using the previously described capacity and queueing equations, the frontage road capacity can be converted to queueing system service rate, and then the queueing system delay can be calculated. In turn, the total delay can be expressed as a function of queueing system delay.

Model Validation

The massive amounts of time required to process the field data precluded the possibility of studying one site to build a model and then studying a second analogous site to prove or validate the model. However, a simple statistical test can be used to determine if the field data is behaving generally as theorized.

If the field data were behaving perfectly according to theory, a plot of field delay vs. theoretical delay would produce a straight line which intercepted the origin (B0 = 0), and had a slope of unity (B1 = 1). Used in a regression equation, these terms appear as

$$Y = B0 + B1 * X$$

Since the queueing system delay does not include nonqueueing delay, an estimated value of nonqueueing delay must be subtracted from field measured delay before making comparisons.

The estimation of nonqueueing delay was previously described. The nonqueueing delay was subtracted from field measured delay, and the resulting estimated field queueing delay regressed against theoretical queueing delay. Each of the four modified data groups (i.e., with outliers deleted) was tested for B0 = 0 and B1 = 1. The outcomes of these tests are given in Table 3.

TABLE 3
TESTS OF SLOPE AND INTERCEPT FOR DELAY MODELS

	Alpha value of F-test for	
	Intercept = 0	Slope = 1
Study 1 SM:	0.399	0.362
Study 2 CS:	0.989	0.988
Study 3 CN:	0.007	0.000
Study 4 B :	0.582	0.468

At a 95% confidence level, the hypothesis of B0 = 0 and the hypothesis of B1 = 1 cannot be rejected for Studies 1, 2, and 4. The two hypotheses are rejected for Study 3. For three out of the four data sets, the hypothesis that the actual data is behaving as the model would suggest cannot be rejected at this 5% significance level.

Examinations of the data, especially Study 3 (having higher delay volume situations than any of the other studies), lead one to suspect that the Poisson/queueing system approach approximates the observed behavior at the ramp--frontage road intersection, but does not fully explain it.

Delay Model Calibration

Plots of field measured delay against calculated queueing system delay are shown in Appendix C. These plots show a positive correlational trend, but a wide degree of variability or scatter. By regressing field measured delay D against calculated queueing system delay, W, the values of B0 and B1 can be calibrated to derive an equation to predict delay for given ramp and frontage road volume combinations. These equations, which predict the average delay D in seconds per vehicle, are presented in Table 4.

TABLE 4
INITIAL CALIBRATED MODELS FOR DELAY

	Equation	R2
Study 1 (exit ramp, one-way converging)	D = -1.7047 + 2.3723 W	0.15
Study 2 (exit ramp, two-way converging)	D = -2.8532 + 1.6760 W	0.46
Study 3 (exit ramp, two-way opposing)	D = -1.6451 + 1.7785 W	0.83
Study 4 (entry ramp, two-way opposing)	D = 3.0396 + 0.7329 W	0.31

Although plots of the data show definite trends, and the test for intercept and slope were supportive, low coefficients of determination appear with these models. Investigation of the plots and the data show that some of the points are outliers. The identification of outliers was discussed in the preceding pages. If the questionable data in Studies 1, 2, and 4 are deleted, the predictive models of Table 5 result.

TABLE 5
MODIFIED CALIBRATED MODELS FOR DELAY

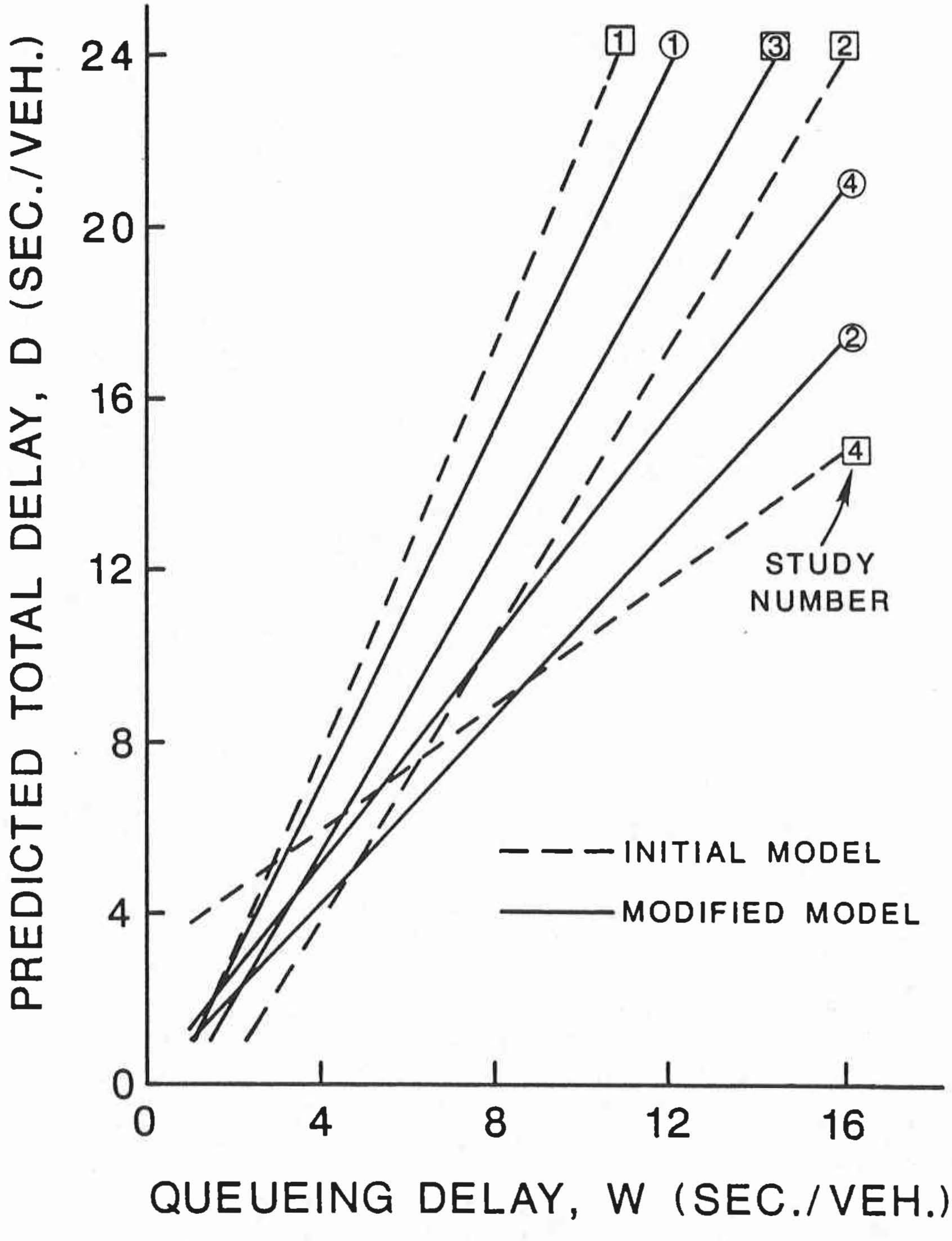
	Equation	R2
Study 1 (exit ramp, one-way converging)	D = -1.3335 + 2.0943 W	0.24
Study 2 (exit ramp, two-way converging)	D = -0.0719 + 1.0922 W	0.32
Study 3 (exit ramp, two-way opposing)	D = -1.6451 + 1.7785 W	0.83
Study 4 (entry ramp, two-way opposing)	D = 0.0538 + 1.3027 W	0.73

After critically evaluating the data and omitting groups with questionable values, the coefficient of determination improved for Study 4, somewhat improved for Study 1, and declined for Study 2.

Evaluating the Models

The models can be evaluated by plotting the predicted total delay. Both the original and modified calibrated models are plotted in Figure 15. Results from all of the studies show similar patterns, but Study 1 results show a greater total delay for higher values of a given queueing delay. Reviewing the data from Study 1, one notices that the range of volumes encountered was rather narrow. This led to resulting delays in a narrow range. Basing the model on a narrow range of data possibly caused the model to be erroneous for higher values of queueing delay.

For a queueing system delay of less than 2.1 (or a service rate minus an arrival rate of greater than 0.48), the Study 3 model seemed to break down, in that it predicted a total delay less than the queueing delay. Another way to state this is that for a W of less than 2.1, the whole became less than the sum of the parts, an improbable situation. Similar breakdowns occurred to Study 1 at 1.2, and Study 2 at 1.1. A review of the data showed that these lower delay values were outside the range of the gathered data.



Study 1 (exit ramp, one-way converging)
Study 2 (exit ramp, two-way converging)
Study 3 (exit ramp, two-way opposing)
Study 4 (entry ramp, two-way opposing)

DELAY MODELS PLOTTED
FIGURE 15

Explaining the Variability

A computer simulation of the queueing process showed that delay can vary widely for a given ramp and frontage road volume combination. For each of the four studies, twenty separate simulations were made of five separate data periods, for a total of 100 simulations per study. A different random number was used to generate traffic for each simulation. Within the twenty simulations for a single period, the total volumes were the same, but the times of arrival for individual vehicles varied.

These simulations indicated that a wide range of variability from the average can be expected with any set of sample observations from a given queueing system. The standard deviations of the twenty simulations varied from 10% to 30% of the average delay. The maximum delays among the twenty simulations for a given period were often more than twice the minimum delay. However, the average delay for the twenty simulations approximated the delay calculated by queueing theory. This offered an explanation of the wide range of delays found during the field studies.

Recommended Models

The modified models for Studies 2 and 4 appear to be preferable to the initial models. They come close to intersecting the origin, and have similar slopes. The original model for Study 3 can be used, as it seems to be acceptable within limits. The models probably should not be used when the queueing system delay is less than 2.5 seconds per vehicle.

The model for Study 1 may be invalid for higher values of queueing delay. As an alternate procedure to estimate delay for Case 1 (a two-lane, one-way frontage road at an exit ramp), the similarity of Case 1 with Case 2 could be utilized. The expected queueing system wait for two-lanes in one direction can be calculated, then the equation for Case 2 used to predict total delay.

FRACTION DELAYED

Those frontage road vehicles which yield to ramp traffic, along with frontage road vehicles which slowed when a preceding vehicle yielded to a freeway ramp vehicle, comprise the fraction of frontage road vehicles delayed. The fraction of frontage road vehicles delayed at a ramp junction is hypothesized to be related to safe ramp operations, traffic conflicts, and the resulting potential for accidents. The calculation of the fraction delayed can qualitatively show a "level of inconvenience" for two-way compared to one-way frontage road operation.

Model Development

The observed fraction delayed was regressed against the value of p. The equations of Table 6 resulted, where FD is the fraction of frontage road traffic delayed.

TABLE 6
INITIAL MODELS FOR FRACTION DELAYED

	Equation	R^2
Study 1 (exit ramp, one-way converging)	FD = 0.0741 + 1.0539 p	0.25
Study 2 (exit ramp, two-way converging)	FD = 0.0195 + 2.0141 p	0.55
Study 3 (exit ramp, two-way opposing)	FD = 0.2430 + 1.1750 p	0.77
Study 4 (entry ramp, two-way opposing)	FD = 0.3384 + 0.9580 p	0.39

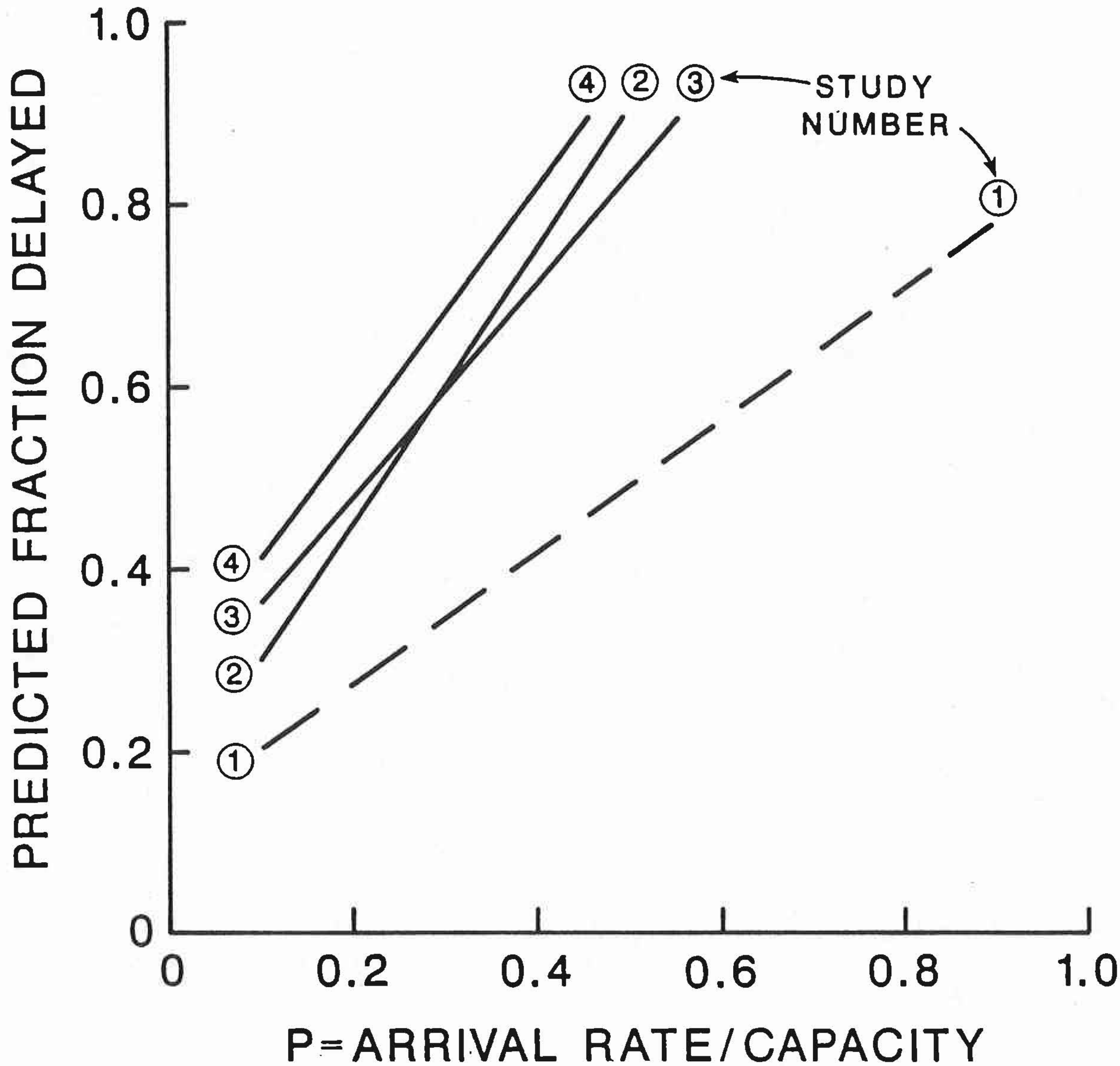
The analysis that was done in the delay model formulation leading to the conclusion that the delays for certain time periods were atypical suggests that data from these time periods should be deleted. The regression equations for these modifed data sets are shown in Figure 16.

The coefficient of determination (R^2) for Study 1 and 2 modified models declined, while R^2 increased for Study 4. However, the "modified" model slope and intercept values among Studies 2, 3, and 4 show more similarity.

As was discussed in the Delay Model section, the Study 1 data was "bunched," which tends to make models based on the data be less reliable. The Study 1 modified model predicts less than 100% of the vehicles being delayed when the demand exceeds the capacity. It is probably better to use the Study 2 model for Study 1 situations.

COMPARISON WITH PREVIOUS ACCIDENT WARRANTS

Study 288 by Woods and others recommended that when the combined volumes on a pair of two-way frontage roads in an "intermediate" area (between rural and urban) reached 6000 vehicles per day --total on both frontage roads, both directions-- conversion to one-way should take place. Using a K-factor of 0.084 to convert average daily traffic to peak hour flow, a 504 vehicle per hour (vph) volume could be expected on the two frontage roads, or 252 vph on



	Equation	R2
Study 1 (exit ramp, one-way converging)	FD = 0.1306 + 0.7234 p	0.17
Study 2 (exit ramp, two-way converging)	FD = 0.1427 + 1.5358 p	0.48
Study 3 (exit ramp, two-way opposing)	FD = 0.2430 + 1.1750 p	0.77
Study 4 (entry ramp, two-way opposing)	FD = 0.2736 + 1.3662 p	0.49

MODIFIED MODELS FOR FRACTION DELAYED
FIGURE 16

each of the two two-way frontage roads during the peak hour. This study also recommended conversion in rural areas when volume on the frontage road pair reaches 7500 per day which, by using the 0.084 factor, would produce a peak hour volume of 315 vehicles on one frontage road (total of both directions).

Although any particular site will have unique volume patterns, estimates of typical ramp volumes associated with the above frontage road volumes result in predictions of about 25% to 50% of the frontage road traffic being delayed during peak hour conditions due to conflicts with the ramp traffic. These calculations assume that the volumes on each of the two frontage roads are equal; in fact, they will probably be somewhat unbalanced. When, during peak hour conditions on a two-way frontage road, roughly a quarter to a half of the frontage road vehicles have their progress affected by the ramp flow, the accident conditions may exist which could be helped by conversion to one-way operation.

USING THE MODELS--AN EXAMPLE

The recommended models to predict capacity, delay, and fraction of the frontage road traffic delayed are shown in Figure 17.

To illustrate the use of the equations, assume a situation consisting of an exit ramp intersecting a two-way frontage road. The hourly ramp volume will be 239 vehicles. The frontage road converging or merging lane will have 143 vehicles per hour, and the contraflow or opposing-the-ramp lane volume is 152 vehicles per hour.

With the given ramp volume, the potential capacity of the merging frontage road lane (Case 2) is

$$Cm = 1724 - 1.6120 * 239 = 1338 \text{ veh/hr.}$$

Knowing the frontage road merging capacity, the queueing system delay per vehicle is

$$Wm = 1 / [(1338 / 3600) - (143 / 3600)] = 3.01 \text{ sec.}$$

The predicted total delay per vehicle is predicted from the queueing delay by

$$Dm = -0.0719 + 1.0922 * 3.01 = 3.22 \text{ sec.}$$

The predicted fraction delayed is estimated as

$$FDm = 0.1427 + 1.5358 * 143 / 1338 = 0.31.$$

The potential capacity, queueing delay, total delay, and fraction delayed

Case 1 (exit ramp, two-lane one-way converging)

$$C = 2 * (1858 - 1.5259 * Qr)$$
$$W = 1 / (u - a)$$
$$D = -0.0719 + 1.0922 * W$$
$$FD = 0.1427 + 1.5358 * p$$

Case 2 (exit ramp, two-way converging)

$$C = 1724 - 1.6120 * Qr$$
$$W = 1 / (u - a)$$
$$D = -0.0719 + 1.0922 * W$$
$$FD = 0.1427 + 1.5358 * p$$

Case 3 (exit ramp, two-way opposing)

$$C = 1444 - 1.6564 * Qr$$
$$W = 1 / (u - a)$$
$$D = -1.6451 + 1.7785 * W$$
$$FD = 0.2430 + 1.1750 * p$$

Case 4 (entry ramp, two-way opposing)

$$C = 1535 - 1.3852 * Qr$$
$$W = 1 / (u - a)$$
$$D = 0.0538 + 1.3027 * W$$
$$FD = 0.2736 + 1.3662 * p$$

where Qr = hourly ramp volume
C = hourly frontage road capacity per direction
a = frontage road flow rate, vehicles per second
u = C / 3600
D = average delay per vehicle, in seconds
p = a / u
FD = fraction of frontage road traffic delayed

Note: All two-way frontage roads have one lane per direction.
The equation of capacity listed above gives an estimate.
See preceeding text for the more exact capacity formula.

SUMMARY OF PREDICTIVE EQUATIONS
FIGURE 17

for the contraflow frontage road lane (Case 3) can be estimated in a similar manner.

$$Cc = 1444 - 1.6564 * 239 = 1048 \text{ veh/hr}$$

$$Wc = 1 / [(1048 / 3600) - (152 / 3600)] = 4.02 \text{ sec}$$

$$Dc = -1.6451 + 1.7785 * 4.02 = 5.50 \text{ sec}$$

$$FDc = 0.2430 + 1.1750 * 152 / 1048 = 0.41$$

The total delay per hour for both lanes of the two-way frontage road would be the traffic volume multiplied by the delay per vehicle.

$$Dt2 = 143 * 3.22 + 152 * 5.50 = 1296 \text{ sec.}$$

If the traffic volumes on both frontage roads of a pair were equal, conversion to one-way operation (Case 1) might result in a volume of 315 per hour, or 158 on each of the two lanes. The per lane capacity, queueing delay, total delay, and fraction delayed would then be

$$Co = 2 * (1858 - 1.5259 * 239) = 2986$$

$$Wo = 1 / [(2986 / 3600) - (315 / 3600) = 1.35 \text{ sec}$$

$$Do = -0.0719 + 1.0922 * 1.35 = 1.40 \text{ sec}$$

$$FDo = 0.1427 + 1.5358 * 315 / 2986 = 0.30$$

The total delay per hour for the one-way frontage road would be

$$Dt1 = 315 * 1.40 = 441 \text{ sec}$$

The estimated decrease in hourly delay after conversion to a one-way frontage road would be

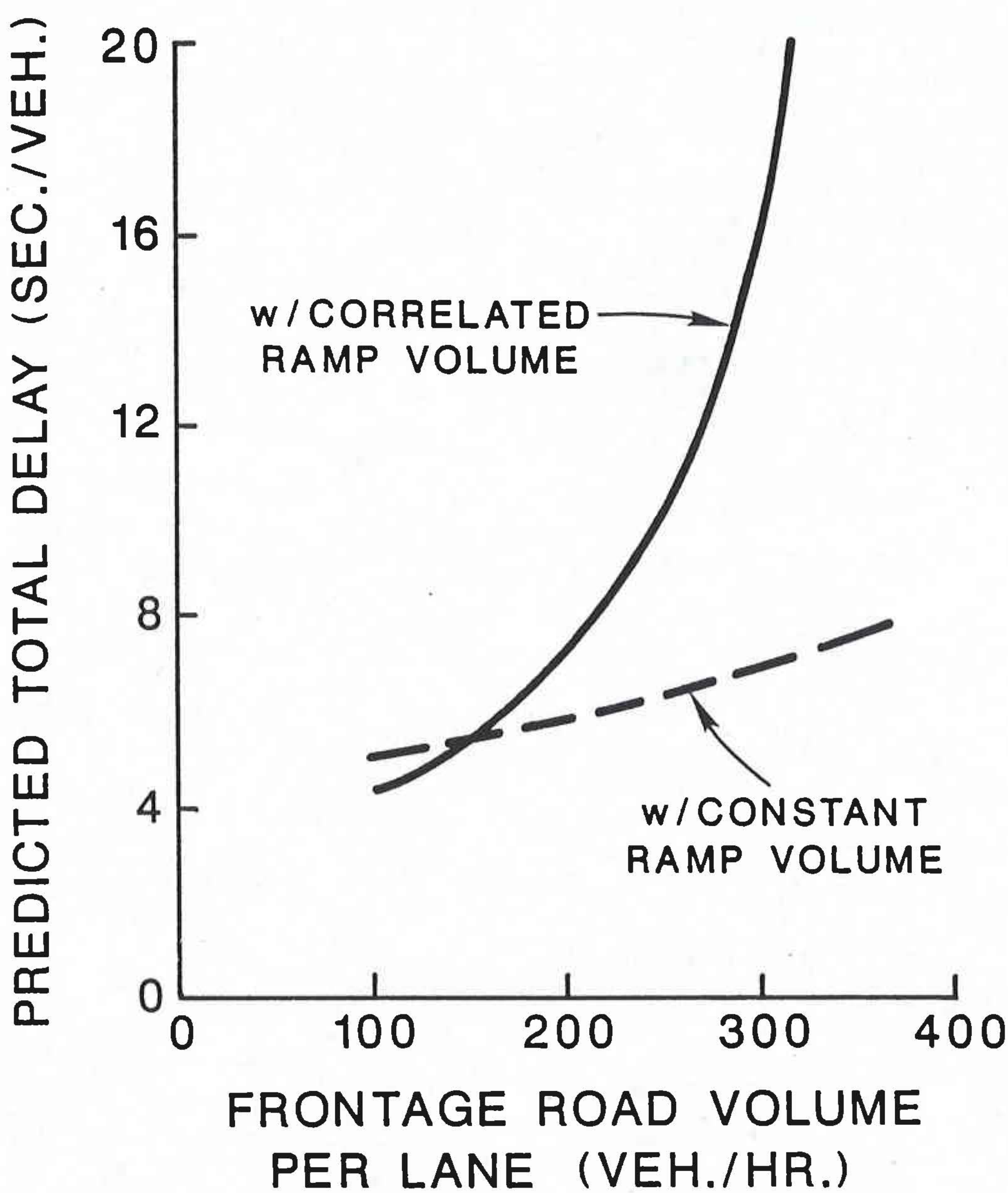
$$(1296 - 441) / 1296 = 0.66,$$

or about a two-thirds decrease.

The ramp volumes and the frontage road volumes showed a time--correlation in all four studies. The ebb and flow of volumes on the ramp and frontage road are somewhat coordinated.

The lower left portion of Figure 18 shows the predicted frontage road delays for a situation of varying frontage road volume with constant ramp volume. The lower right portion shows the delays for the varying frontage road volume levels and a correlated increase in ramp volume. Note that the rate of increase of delay is much greater for the correlated ramp volumes than for the constant ramp volumes. When evaluating ramp--frontage road intersection delay, one should remember that a positive time--correlation between ramp and frontage road volumes can have a significant impact on the total delay.

Figure 19 shows the delays for the four study cases as the frontage road volume increases along with a correlated ramp volume. A relationship between ramp volume and frontage road volume was assumed, and as the frontage road volume increased, the ramp volumes increased in a proportional manner. Again, as correlated volumes increase, the rate of increase in delay per vehicle becomes greater and greater.



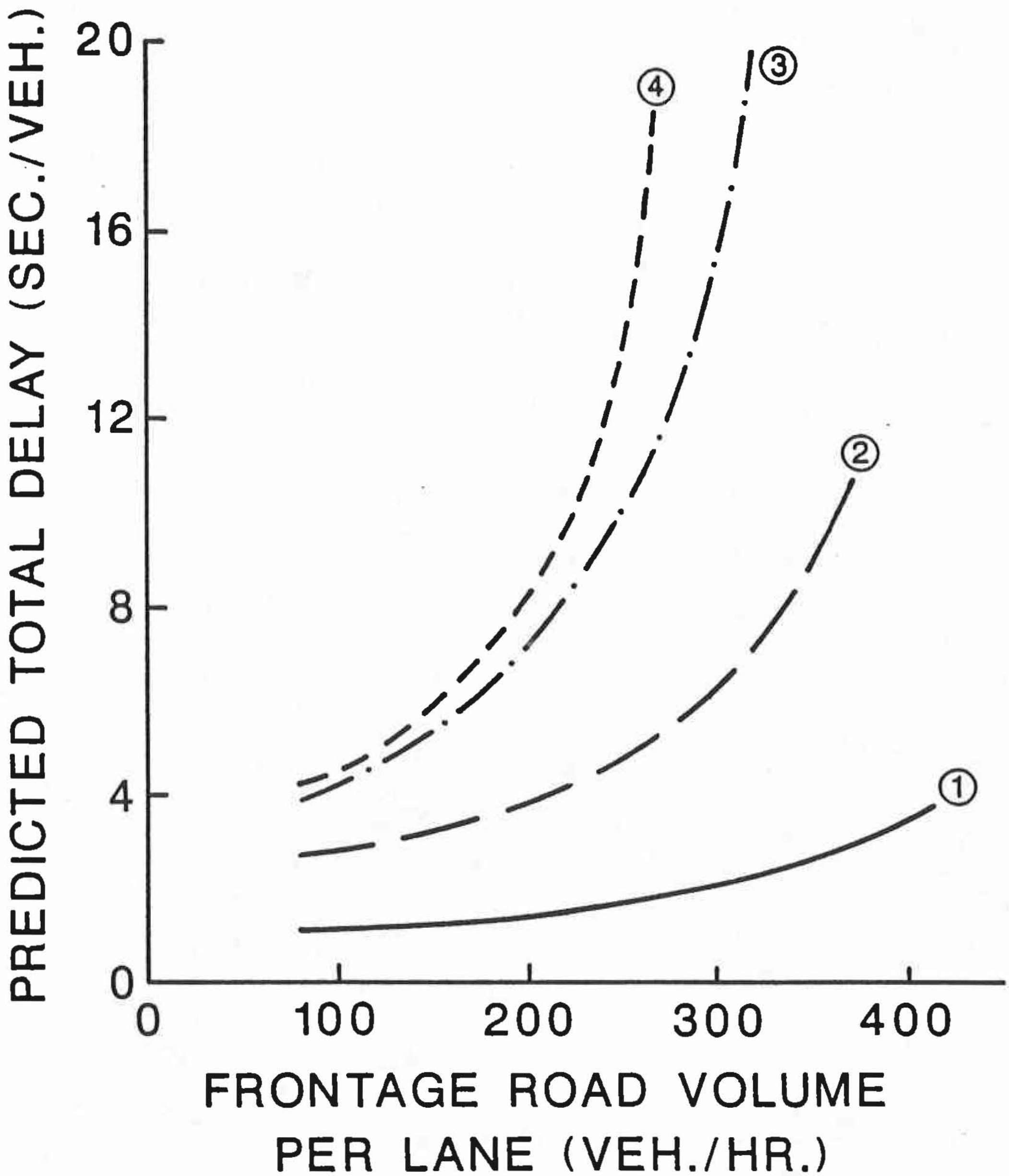
Constant Ramp Volume

Frontage road contraflow lane volume (veh/hr)	Ramp volume (veh/hr)	Total delay, D (sec)
122	239	5.3
152	239	5.5
159	239	5.6
239	239	6.3
318	239	7.1

Correlated Ramp Volumes

Frontage road contraflow lane volume (veh/hr)	Ramp volume (veh/hr)	Total delay, D (sec)
122	191	4.7
152	239	5.5
159	250	5.7
239	375	9.3
318	500	19.9

EFFECTS OF VARYING AN ASSUMED RAMP VOLUME ON DELAY
FIGURE 18



Case 1 (exit ramp, one-way converging)
Case 2 (exit ramp, two-way converging)
Case 3 (exit ramp, two-way opposing)
Case 4 (entry ramp, two-way opposing)

PREDICTED DELAYS FOR AN ASSUMED VOLUME DISTRIBUTION
FIGURE 19

V. FINDINGS, CONCLUSIONS AND RECOMMENDATIONS

The objective of this study was to devise a method to estimate the delay to frontage road vehicles required to yield the right-of-way to ramp traffic. Delays were evaluated as functions of frontage road and ramp volumes.

FINDINGS

After evaluating the data and the results, predictive equations were developed. Due to what was believed to be a narrow range of collected data, some of the equations for Case 1 are suspect. Therefore, to predict Case 1 delays and fraction of vehicles delayed, the use of Case 2 equations is recommended.

Four sets of equations were formulated as shown in Figure 20. First, the remaining available frontage road per lane capacity, C, is estimated as a function of ramp volume. Second, the queueing delay equation is used to calculate W. Third, total delay per vehicle is calculated. Equations to estimate the fraction delayed are the last set.

The equations for Case 3 were the most reliable, as judged by the coefficient of determination. While the other relationships were not as strong, their similarity with the Case 3 models indicates that they are fairly reliable. If, for a given case, the average accepted gap at a particular site varied greatly from that in this study, then the delay prediction could be affected.

The models predict that average delay per frontage road vehicle increases as queueing delay increases. In turn, queueing delay will increase as ramp volume increases, since increased ramp volume reduces the remaining available capacity for frontage road traffic. These models behave as expected.

It should be noted that a correlation between ramp volume and frontage road volume was observed in the field studies. In the off-peak periods, both ramp volumes and frontage road volumes are low. During the peak hours, however, both volumes are high. Thus as the ramp volume increases, the frontage road demand also increases even though the capacity to serve the demand decreases. In actual traffic conditions, delay per vehicle can be expected to increase exponentially as frontage road volume increases, so long as ramp volume also increases.

Case 1 (exit ramp, two-lane one-way converging)

$$C = 2 * (1858 - 1.5259 * Qr)$$
$$W = 1 / (u - a)$$
$$D = -0.0719 + 1.0922 * W$$
$$FD = 0.1427 + 1.5358 * p$$

Case 2 (exit ramp, two-way converging)

$$C = 1724 - 1.6120 * Qr$$
$$W = 1 / (u - a)$$
$$D = -0.0719 + 1.0922 * W$$
$$FD = 0.1427 + 1.5358 * p$$

Case 3 (exit ramp, two-way opposing)

$$C = 1444 - 1.6564 * Qr$$
$$W = 1 / (u - a)$$
$$D = -1.6451 + 1.7785 * W$$
$$FD = 0.2430 + 1.1750 * p$$

Case 4 (entry ramp, two-way opposing)

$$C = 1535 - 1.3852 * Qr$$
$$W = 1 / (u - a)$$
$$D = 0.0538 + 1.3027 * W$$
$$FD = 0.2736 + 1.3662 * p$$

where Qr = hourly ramp volume (for Case 4, includes all vehicles which approached the entry ramp from the converging direction, whether they actually entered or continued along the frontage road).
C = hourly frontage road capacity per direction
a = frontage road flow rate, vehicles per second
u = C / 3600
D = average delay per vehicle, in seconds
p = a / u
FD = fraction of frontage road traffic delayed

Note: All two-way frontage roads have one lane per direction.

SUMMARY OF PREDICTIVE EQUATIONS
FIGURE 20

CONCLUSIONS

The delay incurred by frontage road traffic yielding to ramp traffic at a ramp--frontage road intersection increases as either the ramp or frontage road volumes increase. By using some aspects of queueing theory, models to predict this delay can be derived. The delays at any given location will vary due to differences in tendencies to accepted gaps in the ramp traffic stream and due to statistical variation. The general agreement shown among the models of the four types of cases studied lends credence to the overall ability of the models to estimate delay to the frontage road motorists.

During periods of low traffic, the average delay to the frontage road traffic stream yielding to ramp vehicles should be insignificant. But as the volumes rise, a sharp rise in the rate of increase of delay per vehicle could be expected, since ramp and frontage road volumes may be correlated. In the situation of a two-way frontage road, the previously discussed safety warrants would call for conversion to one-way operation before the delays became excessive.

RECOMMENDATIONS

The models presented in this report can be used to estimate delay to frontage road traffic yielding to the ramp stream. This delay can be combined with the other sources of delay within the frontage road network (such as at intersections) to evaluate the overall delay. When combined with measurements of travel speed and distance, the net effects on overall travel time among various frontage road operational strategies can be evaluated.

The study was limited by the range of volume data collected. If additional studies of this nature are performed, sites exhibiting a wider range of volumes should be identified and studied.

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APPENDIX A

DATA CHARACTERISTICS

APPENDIX A

Gross Descriptors of Data Sets

Gross descriptors of data from the four studies are presented in Table A1. The reader should remember that comparisons between frontage road data from San Marcos and from the other studies must be made with care, due to the presence of two frontage lanes per direction at the San Marcos site compared with one per direction at the other sites. A pair of two-way frontage roads will have one lane per direction on both sides of the freeway, while a one-way frontage road pair will have two lanes in one direction on one side of the freeway, and two lanes in the opposite direction on the other side of the freeway. Therefore, it may be appropriate in some instances to compare two-way and one-way frontage road data on a volume-per-lane basis, not a volume basis.

The frontage road data was divided into "thru" and "turn," the "turns" being those frontage road vehicles that did not travel through the system from Station F4 to F7. A turn vehicle may have turned onto the frontage road after F4 and before F6, or may have turned off of the frontage road after F6. Those parameters of total elapsed travel time between F4 and F7 do not exist for "turn" frontage road vehicles.

Mean Travel Times of Frontage Road Vehicles

Table A2 shows the descriptors of mean travel time for frontage road vehicles. It is interesting to note that the fractions of frontage road turning vehicles at College Station were almost identical for the two dates on which the studies were made. The frontage road travel time data reflects a further subsetting into categories of "yield," "nonyield," and "unknown." The unknown category is composed of those few thru vehicles for which their yielding or nonyielding could not be ascertained from reviewing the sequential data.

This same analysis showed that the TIMET47 of some nonyield vehicles is greater than the TIMET47 of some yield vehicles. During the data collection process, it was observed that some of the nonyielding vehicles proceed through the ramp--frontage road intersection slowly and cautiously, while some of the yield vehicles appear to be trying to recoup lost time with rapid reacceleration to normal speed. This comparison showing the faster yielding vehicles requiring less time than the slower nonyielding vehicles is presented in Table A3.

TABLE A1
GROSS DESCRIPTORS OF DATA

	1	2	3	4
	SAN MARCOS	COL STA SB	COL STA NB	BRYAN
	1-way exit	2-way exit	2-way exit	2-way entry
(1) TOTAL NUMBER OF VEHICLES	1911	2314	1910	1587
(2) NUMBER OF FRONTAGE ROAD VEHICLES NOT GOING THRU INTERSECTION	35	113	0	9
(3) NUMBER OF VEHICLES GOING THRU INTERSECTION, (2) - (1)	1876	2201	1910	1578
(4) NUMBER OF RAMP VEHICLES	649	1392	1286	1193
(5) NUMBER OF FRONTAGE ROAD VEHICLES	1227	809	624	385
(6) NUMBER OF FRONTAGE ROAD VEHICLES PER LANE--DIRECTION	614	809	624	385
(7) NUMBER OF FRONTAGE ROAD TURN	91	135	103	44
7a. Fraction of turn/total, (6)/(5)	0.074	0.167	0.165	0.114
(8) NUMBER OF FRONTAGE ROAD THRU	1136	674	521	341
(9) NUMBER OF THRU FRONTAGE YIELD	253	293	284	180
9a. fraction of yield, (9)/(8)	0.22	0.44	0.54	0.53

TABLE A2
DESCRIPTORS OF FRONTAGE ROAD TRAVEL TIME

STUDY:		1:SM	2:CS	3:CN	4:B
(8) NUMBER OF FRONTAGE ROAD THRU		1136	674	521	341
(9) NUMBER OF THRU YIELD VEHICLES		253	293	284	180
TDIFF47	average	7.22	8.28	17.96	12.65
	std. dev.	4.60	5.91	11.82	8.21
	90th percentile	12.28	16.76	35.95	23.92
	median	6.24	6.94	15.18	10.65
	10th percentile	2.91	1.96	5.64	3.77
TIMET47	average	22.22	23.21	36.04	26.64
	std. dev.	5.00	6.45	12.42	8.47
	90th percentile	28.36	33.00	54.80	38.87
	median	21.40	22.30	32.95	24.45
	10th percentile	17.34	16.00	22.65	16.97
(10) NUMBER OF THRU NONYIELD VEHICLES		878	380	236	161
TDIFF47	average	0.98	0.16	0.44	0.70
	std. dev.	1.51	1.37	1.42	1.38
	90th percentile	2.95	2.04	2.61	2.51
	median	0.64	-0.28	-0.01	0.25
	10th percentile	-0.31	-1.15	-0.71	-0.35
TIMET47	average	15.31	13.58	17.12	12.95
	std. dev.	2.71	2.46	2.67	2.30
	90th percentile	19.20	17.09	21.03	15.80
	median	15.00	13.20	16.80	12.50
	10th percentile	12.10	10.50	14.30	10.32
(11) NUMBER OF THRU UNKNOWNS		5	1	1	0

TDIFF47 is actual travel time minus projected travel time.
TIMET47 is actual travel time.

TABLE A3
COMPARISON OF YIELDING AND NONYIELDING EXTREME TIMET47 VALUES

	1 SAN MARCOS	2 COL STA SB	3 COL STA NB	4 BRYAN
Yielding vehicles, 5th percentile	16.3	14.6	20.8	15.1
Nonyielding vehicles, 95th percentile	20.2	17.8	22.0	18.1

(Values are in seconds.)

The size of ramp vehicle headway which was accepted 50% of the time was determined for each of the four study sites. The normal minimum frontage road car following headways were also determined. These were needed to determine the portion of the time that headways of sufficient size for frontage road acceptance were in existance for a given ramp volume.

TABLE A4
HEADWAY ACCEPTANCE PARAMETERS

	1 SAN MARCOS	2 COL STA SB	3 COL STA NB	4 BRYAN
Ramp vehicle average accepted headway size in seconds	3.6	5.1	7.2	6.0
Normal minimum frontage road car following headway in seconds	1.9	1.9	2.1	1.9

APPENDIX B

SUMMARIES OF FIFTEEN MINUTE DATA

TABLE B-1
STUDY 1: SAN MARCOS

15-minute interval data

Data group number	Ending time of 15-min. interval	Ramp Q	Ramp arrival rate	Frontage road Q	Frontage road arrival rate	p = rho	Queueing delay, W	Capacity by queueing theory	Observed delay	Est. capacity using field data	Observed fraction delayed
3	8:17:20	48	0.0533	80	0.0889	0.102	1.3	782	1.6	711	0.213
4	8:22:20	48	0.0533	82	0.0911	0.105	1.3	782	1.5	730	0.200
5	8:27:20	48	0.0533	81	0.0900	0.104	1.3	782	1.2	874	0.125
6	8:32:20	39	0.0433	84	0.0933	0.104	1.2	811	0.7	1611	0.107
7	8:37:20	42	0.0467	83	0.0922	0.104	1.3	801	0.7	1494	0.108
8	8:42:20	37	0.0411	87	0.0967	0.106	1.2	817	0.8	1440	0.149
9	8:47:20	36	0.0400	83	0.0922	0.101	1.2	820	0.9	1250	0.145
10	8:52:20	35	0.0389	72	0.0800	0.087	1.2	824	0.8	1386	0.111
13	10:27:19	51	0.0567	78	0.0867	0.101	1.3	773	3.1	383	0.308
14	10:32:19	50	0.0556	85	0.0944	0.110	1.3	776	3.1	392	0.294
18	12:45:05	56	0.0622	107	0.1189	0.141	1.4	757	0.9	1267	0.170
19	12:50:05	57	0.0633	123	0.1367	0.163	1.4	754	1.3	907	0.178
20	12:55:05	59	0.0656	129	0.1433	0.172	1.5	748	2.0	628	0.244
24	15:56:59	68	0.0756	108	0.1200	0.150	1.5	722	2.2	563	0.290
25	16:01:59	63	0.0700	102	0.1133	0.139	1.4	736	1.9	630	0.267
26	16:07:20	61	0.0678	109	0.1211	0.147	1.4	742	2.0	592	0.298
27	16:12:20	60	0.0667	112	0.1244	0.150	1.4	745	1.3	906	0.213
28	16:17:20	68	0.0756	131	0.1456	0.182	1.5	722	1.6	742	0.236
29	16:22:20	62	0.0689	143	0.1589	0.193	1.5	739	1.4	855	0.189
33	16:51:00	76	0.0844	133	0.1478	0.190	1.6	699	2.5	529	0.305
34	16:56:00	73	0.0811	119	0.1322	0.168	1.5	707	2.1	589	0.287
35	17:01:00	67	0.0744	114	0.1267	0.157	1.5	725	1.8	658	0.265
36	17:06:52	66	0.0733	112	0.1244	0.154	1.5	728	1.3	910	0.257
37	17:11:52	68	0.0756	113	0.1256	0.157	1.5	722	1.5	809	0.282

TABLE B-2
STUDY 2: COLLEGE STATION SB

15-minute interval data

Data group number	Ending time of 15-min. interval	Ramp Q	Ramp arrival rate	Frontage road Q	Frontage road arrival rate	ρ = rho	Queueing delay, W	Capacity by queueing theory	Observed delay	Est. capacity using field data	Observed fraction delayed
3	8:35:47	61	0.0678	22	0.0244	0.066	2.9	335	2.9	349	0.318
7	10:58:20	69	0.0767	52	0.0578	0.162	3.4	320	1.9	568	0.250
8	11:03:20	72	0.0800	48	0.0533	0.152	3.4	315	0.9	1220	0.167
9	11:08:20	74	0.0822	47	0.0522	0.151	3.4	311	3.6	308	0.255
10	11:13:20	78	0.0867	45	0.0500	0.148	3.5	304	4.4	259	0.267
11	11:18:20	64	0.0711	45	0.0500	0.137	3.2	330	3.8	292	0.311
12	11:23:20	71	0.0789	44	0.0489	0.139	3.3	317	1.9	545	0.227
13	11:28:20	69	0.0767	49	0.0544	0.153	3.3	320	1.8	596	0.245
17	11:56:41	100	0.1111	63	0.0700	0.234	4.4	269	3.8	313	0.377
18	12:01:41	105	0.1167	65	0.0722	0.249	4.6	261	3.4	354	0.444
19	12:06:41	109	0.1211	56	0.0622	0.219	4.5	255	3.9	308	0.482
20	12:11:41	109	0.1211	55	0.0611	0.215	4.5	255	5.4	233	0.600
21	12:16:41	121	0.1344	58	0.0644	0.243	5.0	239	6.8	198	0.603
22	12:21:41	115	0.1278	53	0.0589	0.215	4.6	247	6.5	200	0.547
23	12:26:41	117	0.1300	55	0.0611	0.225	4.8	244	5.3	234	0.509
24	12:31:41	107	0.1189	55	0.0611	0.213	4.4	258	3.3	356	0.473
28	13:00:30	106	0.1178	67	0.0744	0.258	4.7	260	5.3	246	0.463
29	13:05:30	104	0.1156	65	0.0722	0.247	4.6	263	2.6	444	0.400
30	13:13:22	113	0.1256	64	0.0711	0.256	4.8	250	5.5	240	0.578
31	13:18:22	112	0.1244	57	0.0633	0.227	4.6	251	6.2	211	0.579
32	13:23:22	94	0.1044	59	0.0656	0.212	4.1	278	5.1	248	0.508
33	13:28:22	92	0.1022	52	0.0578	0.185	3.9	281	4.5	265	0.462
37	15:45:00	86	0.0956	42	0.0467	0.144	3.6	291	2.1	509	0.262
38	15:50:00	97	0.1078	59	0.0656	0.216	4.2	273	3.4	344	0.390
39	15:55:00	107	0.1189	63	0.0700	0.244	4.6	258	4.0	306	0.492
40	16:00:00	94	0.1044	66	0.0733	0.237	4.2	278	5.0	260	0.545
41	16:05:00	94	0.1044	58	0.0644	0.209	4.1	278	5.1	245	0.534
42	16:10:00	90	0.1000	54	0.0600	0.190	3.9	284	4.1	289	0.463
43	16:15:00	91	0.1011	63	0.0700	0.223	4.1	283	4.0	307	0.460
44	16:20:00	98	0.1089	65	0.0722	0.239	4.4	272	4.2	297	0.462
48	16:50:53	107	0.1189	58	0.0644	0.225	4.5	258	4.4	274	0.483

TABLE B-3
STUDY 3: COLLEGE STATION NB

15-minute interval data

Data group number	Ending time of 15-min. interval	Ramp Q	Ramp arrival rate	Frontage road Q	Frontage road arrival rate	P = rho	Queueing delay, W	Capacity by queueing theory	Observed delay	Est. capacity using field data	Observed fraction delayed
3	8:31:21	66	0.0733	26	0.0311	0.111	4.0	253	5.4	252	0.357
4	8:36:21	62	0.0689	25	0.0278	0.096	3.8	261	5.1	266	0.333
5	8:41:21	57	0.0633	28	0.0311	0.103	3.7	272	5.8	234	0.370
6	8:46:21	52	0.0578	26	0.0289	0.092	3.5	283	3.5	400	0.280
10	10:16:50	67	0.0744	25	0.0278	0.100	4.0	251	9.0	154	0.500
11	10:21:50	59	0.0656	24	0.0267	0.090	3.7	267	6.8	194	0.375
12	10:26:50	67	0.0744	28	0.0311	0.112	4.0	251	5.9	211	0.250
13	10:31:50	69	0.0767	26	0.0289	0.105	4.1	247	4.8	271	0.269
14	10:36:50	67	0.0744	25	0.0278	0.100	4.0	251	3.4	457	0.320
15	10:41:50	65	0.0722	26	0.0289	0.102	3.9	255	6.8	213	0.500
16	10:46:50	63	0.0700	32	0.0356	0.124	4.0	259	5.7	263	0.438
17	10:51:50	66	0.0733	33	0.0367	0.131	4.1	253	7.1	204	0.455
18	10:56:50	71	0.0789	34	0.0378	0.140	4.3	243	3.7	415	0.324
19	11:01:50	76	0.0844	30	0.0333	0.129	4.4	233	4.9	313	0.433
20	11:06:50	88	0.0978	29	0.0322	0.137	4.9	212	6.2	222	0.379
21	11:11:50	93	0.1033	27	0.0300	0.133	5.1	204	8.6	165	0.519
22	11:16:50	89	0.0989	29	0.0322	0.138	5.0	210	7.8	173	0.379
23	11:21:50	68	0.0756	32	0.0356	0.129	4.2	249	4.6	312	0.344
24	11:27:36	71	0.0789	33	0.0367	0.136	4.3	243	3.5	422	0.303
25	11:32:36	83	0.0922	45	0.0500	0.204	5.1	221	5.1	323	0.467
26	11:37:36	101	0.1122	48	0.0533	0.251	6.3	191	8.0	197	0.500
27	11:42:36	91	0.1011	51	0.0567	0.246	5.8	207	7.6	210	0.490
28	11:48:28	87	0.0967	44	0.0489	0.206	5.3	214	8.0	185	0.409
29	11:53:28	85	0.0944	49	0.0544	0.226	5.4	217	9.7	165	0.490
30	11:58:28	95	0.1056	48	0.0533	0.239	5.9	200	11.1	148	0.521
31	12:03:28	96	0.1067	46	0.0511	0.231	5.9	199	12.8	133	0.630
32	12:08:28	102	0.1133	50	0.0556	0.264	6.5	190	10.1	167	0.620
33	12:13:28	125	0.1389	51	0.0567	0.323	8.4	158	9.6	180	0.647
37	12:45:49	98	0.1089	62	0.0689	0.317	6.7	196	11.1	170	0.689
38	12:50:49	93	0.1033	61	0.0678	0.300	6.3	204	8.1	217	0.583
39	12:55:49	95	0.1056	53	0.0589	0.264	6.1	200	10.4	169	0.660
43	17:05:15	117	0.1300	68	0.0756	0.405	9.0	168	15.3	140	0.677
44	17:11:23	128	0.1422	73	0.0811	0.474	11.1	154	18.6	131	0.746
45	17:16:23	141	0.1567	68	0.0756	0.490	12.7	139	21.7	117	0.815

TABLE B-4
STUDY 4: BRYAN

15-minute interval data

Data group number	Ending time of 15-min. interval	Ramp Q	Ramp arrival rate	Frontage road Q	Frontage road arrival rate	P = rho	Queueing delay, W	Capacity by queueing theory	Observed delay	Est. capacity using field data	Observed fraction delayed
3	15:19:14	77	0.0856	28	0.0311	0.099	3.5	283	3.8	341	0.321
4	15:24:14	74	0.0822	38	0.0422	0.131	3.6	289	8.2	173	0.500
5	15:29:14	72	0.0800	39	0.0433	0.133	3.5	293	8.6	166	0.538
6	15:34:14	79	0.0878	44	0.0489	0.157	3.8	280	9.1	164	0.523
7	15:39:14	89	0.0989	37	0.0411	0.141	4.0	262	4.8	275	0.324
8	15:44:14	94	0.1044	35	0.0389	0.138	4.1	253	5.1	250	0.314
9	15:49:14	102	0.1133	29	0.0322	0.121	4.3	240	5.6	224	0.345
10	15:54:14	103	0.1144	27	0.0300	0.113	4.3	238	5.3	254	0.444
11	15:59:14	104	0.1156	28	0.0311	0.118	4.3	237	5.4	265	0.536
12	16:04:14	91	0.1011	36	0.0400	0.139	4.0	258	3.4	451	0.417
13	16:09:14	97	0.1078	34	0.0378	0.137	4.2	248	3.0	514	0.382
14	16:14:14	94	0.1044	37	0.0411	0.146	4.2	253	3.4	436	0.378
15	16:19:14	106	0.1178	43	0.0478	0.184	4.7	234	5.7	264	0.535
16	16:24:14	111	0.1233	42	0.0467	0.186	4.9	226	6.8	223	0.595
17	16:29:14	120	0.1333	35	0.0389	0.164	5.1	213	8.5	172	0.657
18	16:34:14	124	0.1378	26	0.0289	0.125	5.0	207	7.3	190	0.615
19	16:39:14	127	0.1411	30	0.0333	0.148	5.2	203	6.8	204	0.533
20	16:44:14	130	0.1444	34	0.0378	0.171	5.5	199	6.9	197	0.471
21	16:49:14	135	0.1500	32	0.0356	0.166	5.6	193	8.2	168	0.531
22	16:54:14	137	0.1522	41	0.0456	0.216	6.0	190	8.2	178	0.537
23	16:59:14	135	0.1500	42	0.0467	0.218	6.0	193	7.9	185	0.548
24	17:04:14	136	0.1511	50	0.0556	0.261	6.4	191	9.0	173	0.563
25	17:09:14	148	0.1644	41	0.0456	0.232	6.6	177	11.2	140	0.718
26	17:14:14	171	0.1900	46	0.0511	0.304	8.5	151	13.3	128	0.767
27	17:19:14	187	0.2078	49	0.0544	0.360	10.3	136	10.9	151	0.681
28	17:24:14	178	0.1978	53	0.0589	0.367	9.8	145	8.5	186	0.580
29	17:29:14	166	0.1844	55	0.0611	0.351	8.9	157	5.3	306	0.566
30	17:34:14	148	0.1644	46	0.0511	0.260	6.9	177	7.1	219	0.622

APPENDIX C

PLOTS OF OBSERVED TOTAL DELAY AGAINST PREDICTED QUEUEING DELAY

TABLE C-1
STUDY 1: SAN MARCOS

total delay vs. calculated queueing delay, for 15-minute interval data

W and D in sec./veh.

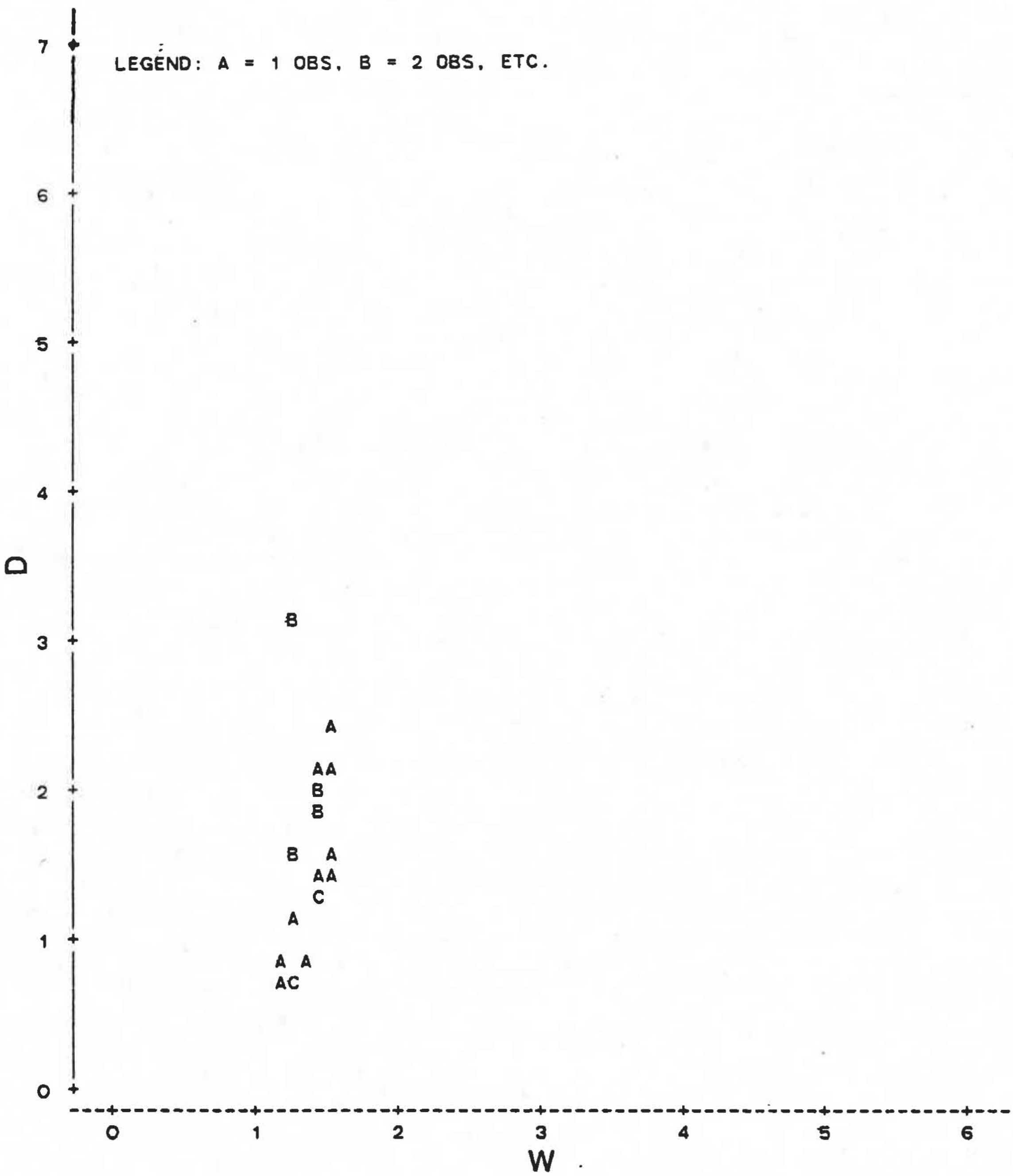

TABLE C-2
STUDY 2: COLLEGE STATION SB

total delay vs. calculated queueing delay, for 15-minute interval data

W and D in sec./veh.

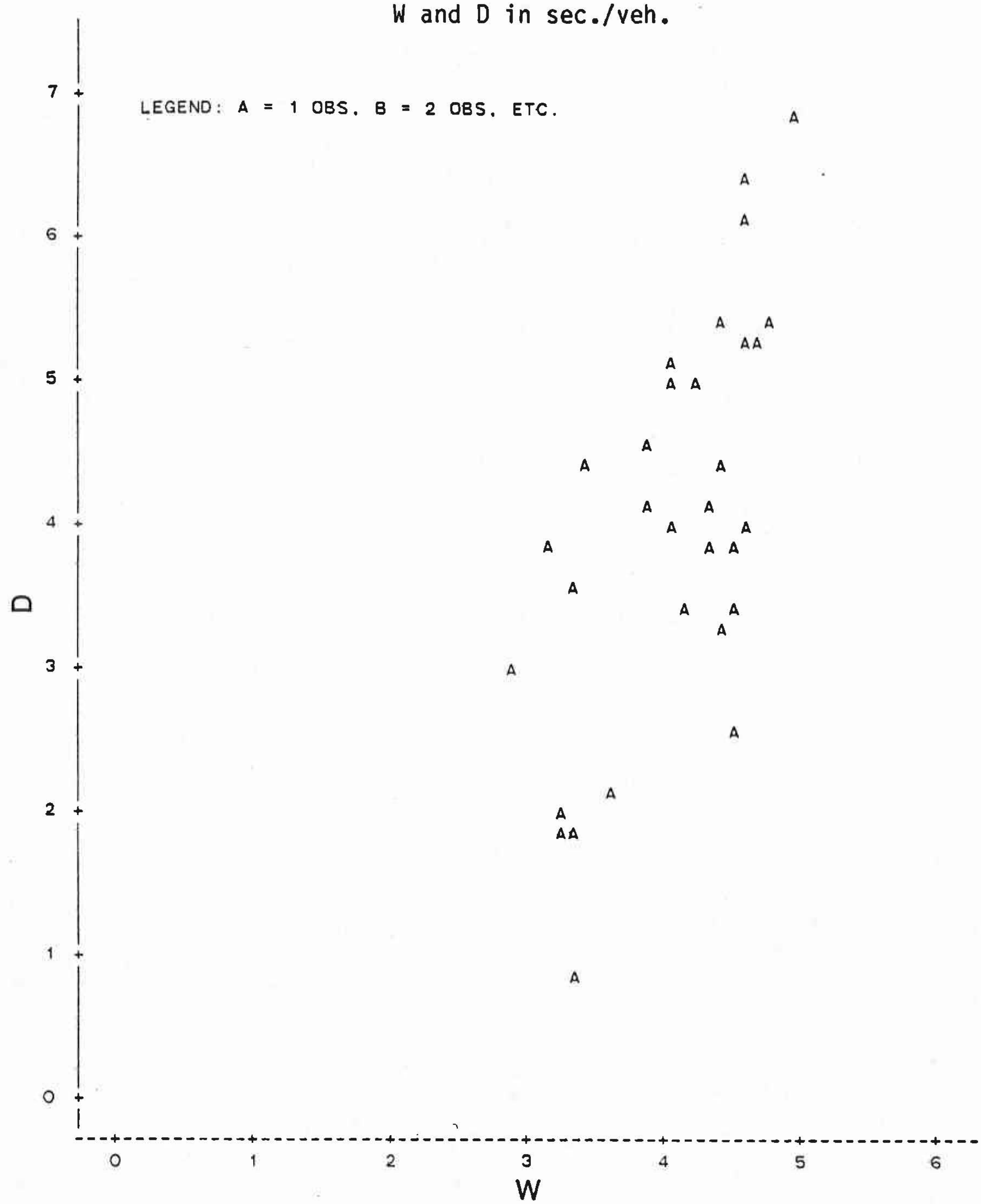

TABLE C-3
STUDY 3: COLLEGE STATION NB

total delay vs. calculated queueing delay, for 15-minute interval data

W and D in sec./veh.

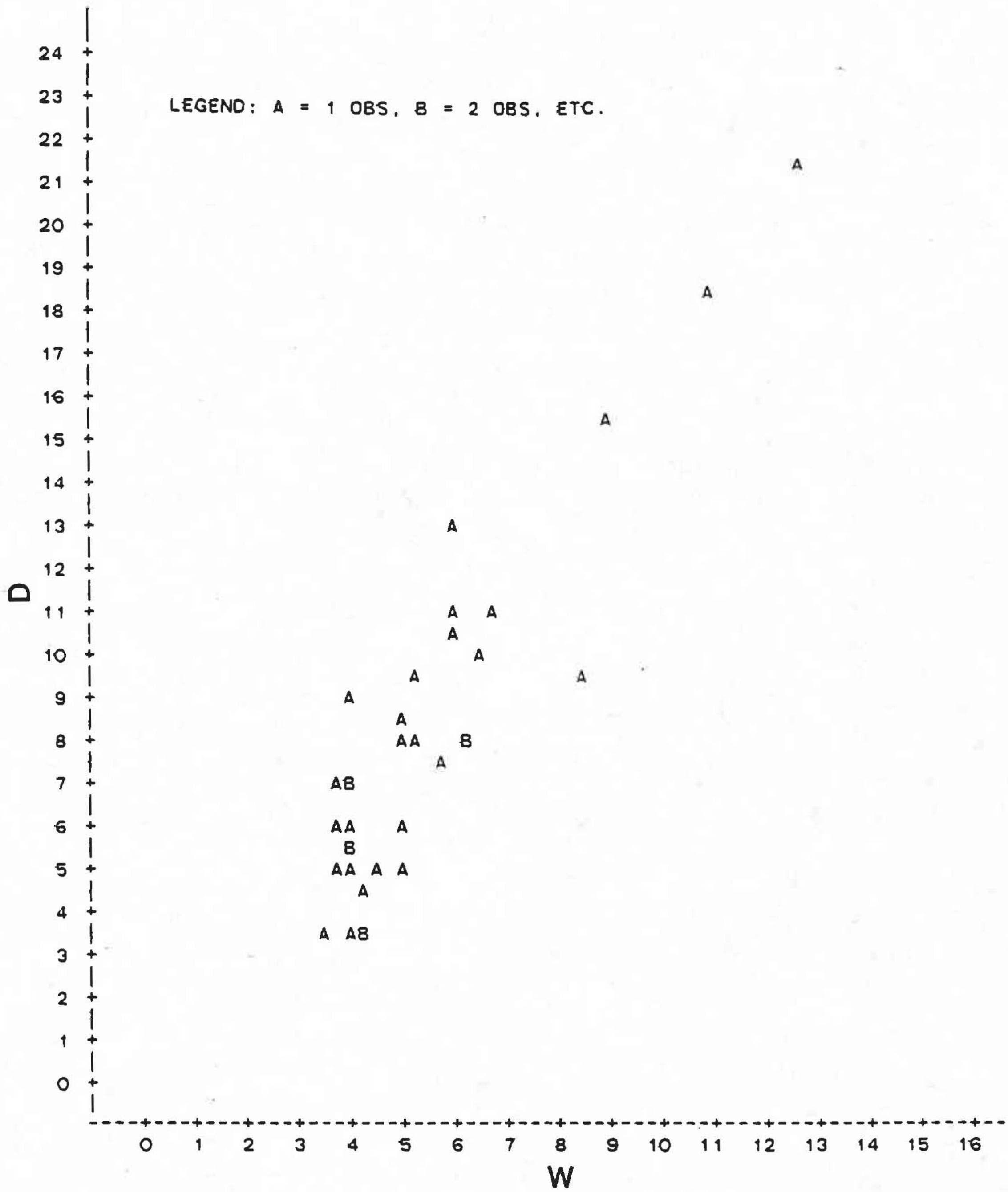

TABLE C-4
STUDY 4: BRYAN

total delay vs. calculated queueing delay, for 15-minute interval data

W and D in sec./veh.