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16. Abstract

The engineering research reports in this document resulted from the second year of the Undergraduate Transportation Engineering Fellows Program during the summer of 1991. The ten-week program, sponsored by the Advanced Institute program of the Southwest Region University Transportation Center (SWUTC), the Texas Transportation Institute (TTI), and the Civil Engineering Department at Texas A&M University, provides undergraduate students in Civil Engineering with the opportunity to learn more about transportation engineering through participation in a transportation research program. The program design allows the students to interact directly with a faculty member or TTI researcher in developing a research proposal, conducting appropriate research, and documenting the research results.

This compendium contains reports on a wide variety of transportation research. The first report discusses the development of a database and priority index for railroad-highway grade crossings. Readers interested in freeways operations will find reports of an evaluation of an inside merge at a major freeway-freeway connection and of a high volume merge operation. Another report contains a study of frontage road queuing and vehicle spacing at diamond interchanges. A study of the relationships between speed and geometric inconsistencies along rural highways is included, as well as an investigation of various operating characteristics of inductance loop detectors. One report contains information on forecasting turning flows, and another discusses lane closure guidelines for arterial street work zones. An examination of the indicators of congestion level also is included.

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1991 Undergraduate Transportation Engineering Fellows

SPONSORED BY THE

Advanced Institute in Transportation Systems Operations and Management

IN COOPERATION WITH

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Civil Engineering Department
Texas A&M University

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Foreword

The Southwest Region University Transportation Center (SWUTC) through the Advanced Institute Program, the Texas Transportation Institute (TTI) and the Civil Engineering Department at Texas A&M University recently established the Undergraduate Transportation Engineering Fellows Program. The program design allows undergraduate students in Civil Engineering to learn more about transportation engineering through participation in a transportation research program under the supervision of a faculty member or TTI researcher. The intent of the program is to introduce transportation engineering to students that have demonstrated outstanding academic performance, thus, developing a critical resource: capable and qualified future transportation professionals.

This past summer, nine students and faculty/staff mentors participated in the program:

STUDENTS	MENTORS
Richard Bartoskewitz Texas A&M University	Dr. Neilon J. Rowan Professor of Civil Engineering
P. Scott Beasley Virginia Polytechnic Institute and State University	Dr. Thomas Urbanik II TTI Research Engineer
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A special thanks is extended to the sponsors of this program--the U.S. Department of Transportation through the Advanced Institute Program of the Southwest Region University Transportation Center, the Texas Transportation Institute, and the Civil Engineering Department at Texas A&M University. Without their support and the contributions of the entire transportation engineering faculty and staff at Texas A&M University, this program could not have succeeded.

Daniel B. Fambro

Daniel B. Fambro, Ph.D., P.E.
Assistant Professor of Civil Engineering,
Undergraduate Fellows Program Coordinator

Development of an Engineering Database for Railroad-Highway Grade Crossings

RICHARD BARTOSKEWITZ

In an effort to improve the planning of interim safety improvements at railroad-highway grade crossings, research was conducted to develop an improved database of grade crossing inventory data. A comprehensive form for collecting this data was developed, incorporating aspects of the U.S. DOT/AAR National Inventory and the grade crossing database maintained by the Texas Department of Transportation. Prioritization of safety improvements was also investigated. Currently, a hazard index formula which considers highway volume, train volume, train speed, crossing protection, and accident history is used to select grade crossings for improvement. The effects of considering sight distance in this process were studied. This was accomplished by modifying the existing hazard index formula with a sight distance rating placed in the denominator. The sight distance rating was a ratio which attempted to quantify the restrictions to sight distance at the grade crossing. Four methods of calculating this ratio were investigated, but no single method could be identified as superior or preferable to the others. It was concluded that the inclusion of a sight distance rating in the hazard index formula appears to be an improvement. However, further research should be conducted into the development and application of this sight distance rating.

INTRODUCTION

In an effort to improve safety on the nation's highways, each state is required by the Federal Highway Administration (FHWA) to develop and implement a highway safety improvement program. The goals of a highway safety improvement program include reductions in the number of accidents and their severity and the elimination of potentially hazardous conditions which exist on or adjacent to the highway. Railroad-highway grade crossing safety improvements are considered to be a part of the state highway safety program.

The basic structure of safety improvement programs is outlined in the "Federal-Aid Highway Program Manual" and consists of procedures for planning, implementing, and evaluating safety improvements. The planning stage, which is the focus of the research described herein, is divided into four components:

1. data collection;
2. data analysis to identify candidate locations for safety improvements;
3. engineering studies to define safety problems encountered at specific locations; and
4. prioritization to establish the order in which safety improvements are to be implemented.

The states have developed various strategies for meeting each of these planning requirements (1).

Accident data and inventory data are the two types of information required in order to plan grade crossing safety improvements. Accident data may be considered in terms of train involved and non-train involved accidents. Any collision which occurs between railroad equipment and a highway vehicle or user at a grade crossing must be reported to the Federal Railroad Administration (FRA), regardless of the number of fatalities or injuries or the extent of property damage. A record of non-train involved accidents which occur at or in the immediate vicinity of grade crossings is useful in identifying other safety or operational problems which may be related to the presence or geometric design of the crossing. Accident data is typically maintained by several agencies, including the FRA, the National Highway Traffic Safety Administration, the state highway department, and state and local law enforcement agencies (1).

Inventory data is any information which relates to the location, function, design, and operational characteristics of a grade crossing. Inventory data for each crossing in the United States is maintained in the U.S. Department of Transportation/Association of American Railroads National Rail-Highway Grade Crossing Inventory. A cooperative effort in the 1970's between the FHWA, FRA, Association of American Railroads, state governments, and railroads resulted in the creation of the National Inventory. The end product was a uniform data pool for all the nation's grade crossings. The information for this database is provided by the state or local agency responsible for grade crossings and the railroads. The database itself is administered by the FRA (1). A typical grade crossing record from the National Inventory is shown in Figure 1. The accuracy and effectiveness of the National Inventory are limited by how its data is updated as conditions at a crossing change. Several proce-

U.S. DOT - AAR CROSSING INVENTORY FORM

Crossing Number 743228C Date 08/08/91

INITIATING AGENCY: State
REASON FOR UPDATE: Changes in Data
EFFECTIVE DATE: / /

Part I *** LOCATION AND CLASSIFICATION OF ALL CROSSINGS ***

1. Railroad Operating Company: SP
2. Railroad Division or Region: HOUSTON
3. Railroad Subdivision or District: HEARNE
4. State: 48
5. County: 041
6. County Map Ref. #: OB
7. City: 7309
8. Nearest/City: Nearest
9. Highway #: CO 0000
10. Street or Road Name: STRAUB ST
11. Railroad ID #: THD08702
12. Timetable Sta.: 682880
13. Branch or Line Name: HOUSTON-DENISON
14. Milepost: 0087V02
15. Crossing Type: Public
16. Crossing Position: At Grade
17. Private Crossing Location: Industrial
18. Private Signs/Signals: Both Signs and Signals
19. Private Sign/Signals Description

Part II. * INFORMATION FOR PUBLIC VEHICULAR AT GRADE CROSSING *

1A. Daylight (6 AM to 6 PM): Thru Trains 6, Switching 1
Night (6 PM to 6 AM): Thru Trains 6, Switching 0

1B. Less Than One Train Per Day? No

2. Maximum Timetable Speed: 40
Typical Speed Range Over Crossing: from 20 to 40

3. Type and Number of Tracks: Main 1 Other 0
Type of Other:

4. Does Another RR Operate a Separate Track at Crossing? No
Other Railroads:

5. Does Another RR Operate Over Your Track at Crossing? Yes
Other Railroads: MP

6. Type of Warning Device at Crossing
A. Signs
Crossbucks: Reflectorized 0, Non-Reflectorized 1
Standard Highway Stop Sign: 0
Other Stop Signs: 0 Other Signs: Number 0 Type
Number 0 Type

Crossing Number 743228C Page 2

B. Train Activated Devices
Gates: Red & While Reflectorized 0, Other Colored 0
Cantilevered Flashing Lights: Over Traffic Lane 0
Not Over Traffic Lane 0
Mast Mounted Flashing Lights: 0
Other Flashing Lights: 0 Type
Highway Traffic Signals: 0, Wigwags 0, Bells 0

C. Specify Special Warning Device not Train Activated

D. Is Track Equipped with any Signs or Signals? Yes

7. Is Commercial Power Available? No

8. Does Crossing Signal Provide Speed Selection for Trains? N/A

9. Method of Signalling for Train Operation:
Is Track Equipped with Signals? Yes

Part III. *** PHYSICAL DATA ***

1. Type of Development: Open Space
2. Smallest Crossing Angle: 60 to 90 Degrees
3. Number of Traffic Lanes Crossing Railroad: 2
4. Are Truck Pullout Lanes Present? No
5. Is Highway Paved? Yes
6. Pavement Markings: None
7. Are Railroad Advance Warning Signs Present? Yes
8. Crossing Surface: Asphalt
9. Does Track Run Down a Street? No
10. Is There a Nearby Intersecting Highway? Yes

Part IV. *** HIGHWAY DEPARTMENT INFORMATION ***

1. Highway System: 8
2. Is Crossing on State Highway System? No
3. Functional Classification of Road over Crossing: 9
4. Estimate AADT: 300
5. Estimate Percent Trucks: 3

FIGURE 1. U.S. DOT - AAR CROSSING INVENTORY RECORD.

dures exist by which a government agency or railroad can initiate an update of the National Inventory records for one or more locations.

In addition to the National Inventory, many state highway agencies maintain their own railroad crossing databases for use in planning grade crossing improvements. The Texas Department of Transportation grade crossing database is used to evaluate the state's crossings each year and determine how the amount of funding specified for upgrading crossings should be spent. The following information is included in the Texas grade crossing database:

- DOT crossing ID number
- county
- street name/highway number
- Federal Aid System number
- operating railroad
- number of main/other tracks
- number of thru trains
- number of switching trains
- speed of thru trains
- speed of switching trains
- existing crossing protection
- train involved accidents
- railroad milepost
- daily traffic volume
- priority index value
- crossing status

Various hazard indices and accident prediction models have been developed to aid in the identification of crossings which should be considered for safety or operational improvements. Four of the more widely-used formulas are the Peabody-Dimmick Formula, New Hampshire Index, NCHRP 50 Formula, and the U.S. DOT Accident Prediction Model. The State of Texas has developed its own hazard index formula, which is given by the following equation:

$$\text{Texas Hazard Index} = V * T * S * P_f * A_5^{1.15} * K$$

where:

V = traffic volume
T = train volume
S = train speed
P_f = protection coefficient
A_5 = five-year accident history
K = parameter

The protection coefficient, P_f, varies according to the type of protection or warning devices at the crossing. For a grade crossing with train-activated warning devices, the value of P_f is less than 1.0. For a crossing with crossbucks, signs, or no traffic control, the value of this factor is 1.0, as passive crossings are considered to be the least desirable form of grade crossing protection. The parameter K is equal to 0.001 and reduces the value of the hazard index into a manageable form, as some of the numbers occasionally become quite large.

Current funding levels for grade crossing improvements in Texas are sufficient to upgrade approximately 200 crossings per year through the installation of flashing red signals with or without gate arms. These crossings are selected on the basis of their hazard index, with those having the largest hazard index receiving top priority. It is desirable that interim measures be taken to improve the safety of crossings which are not within this "high priority" group. However, at the present time, a standard procedure for accomplishing this does not exist.

PROBLEM STATEMENT

The National Grade Crossing Inventory and the Texas Highway Department Database are both useful sources of information on the state's public railroad grade crossings. However, there exists some concern over whether or not the information in these two databases is sufficient for planning and implementing grade crossing safety improvements. Much of the information in the National Inventory, such as that relating to angle of intersection and the presence of adjacent highway intersections, is very general in nature. The Texas crossing database contains information on only a few aspects of railroad grade crossing safety, design, and operations. Neither database deals with the important issue of sight distance on the approach to and at the crossing. A more comprehensive database which combines elements of the National Inventory and Texas grade crossing databases with new features might be a more useful tool in evaluating grade crossings, determining which crossings are in need of improvements, and how they should be improved.

A second issue worthy of investigation is the problem of sight distance on the approach to a grade crossing. Sight distance is not currently a consideration in the prioritization of grade crossing safety improvement projects. There are many ways in which this may be done. One possibility is to include a sight distance variable in the existing hazard index formula. The effect and potential benefits of such a modification should be studied.

OBJECTIVES

The study was conducted in an effort to fulfill two objectives. The first objective was to develop procedures for collecting data relevant to the safety, design, and operational efficiency of passive railroad grade crossings. This data might then be used to develop an engineering database to serve as a tool in planning grade crossing improvements. The second objective of the research was to investigate the inclusion of a sight distance variable in the hazard index calculation, and to determine what, if any, effect this would have on the priority ranking.

STUDY PROCEDURE

In order to meet the requirements of the two previously stated objectives, the research was divided into four tasks.

Task 1 Development of a grade crossing data collection form, which should combine aspects of existing grade crossing databases with new material.

Task 2 Collection of data at railroad grade crossing study sites in Brazos County, including sight distance data.

Task 3 Development of a modification to the hazard index formula based upon a sight distance variable.

Task 4 Comparison of grade crossing priority rankings based upon the existing hazard index formula and the modified hazard index formula.

RAILROAD GRADE CROSSING DATA COLLECTION FORM

To satisfy Task 1 of the study procedure, a railroad grade crossing data collection form was developed. This form is illustrated in Figure 2. Its design was based upon that of the U.S. DOT/AAR Crossing Inventory Form and incorporated much of the information from this database as well as from the Texas crossing database. However, there are several significant additions which should be noted:

1. Data on passenger train operations over the crossing, including the number of daily passenger trains and their maximum speed, is included under "Railroad Operating Characteristics." Information on passenger train operations is not found in either the National Inventory or the Texas crossing database. It is felt that this data is relevant due to the much greater potential for injuries and fatalities among passengers in grade crossing accidents involving passenger trains.

2. The posted speed limit of the highway and the comfortable speed on the approach to the crossing are listed under the heading "Roadway/Highway Characteristics." Both of these are important considerations in determining safe stopping distances for highway vehicles, which in turn, are necessary to the evaluation of sight distance at the crossing. Data on highway speeds is not found in either the National Inventory or the Texas crossing database.

3. The angle of intersection and highway approach grades are included under "Grade Crossing Characteristics." The National Inventory addresses the smallest crossing angle; however, angles are simply placed into one of three categories based on their size: 0° - 29°, 30°- 59°, or 60° - 90°. The National Inventory does not require the exact angle of intersection of the highway and railroad tracks. It is felt that the exact angle is more descriptive of the crossing and is more useful than a simple classification of the angle's size. Highway approach grades are not given by either the National Inventory or the Texas database. This particular information is relevant, in some cases, to the determination of safe stopping distances for highway traffic.

4. The section entitled "Adjacent Highway Intersections" provides specific data on the location, classification, design, and operation of any intersection located within the immediate vicinity of the crossing. Such intersections may tend to distract driver attention away from the grade crossing, thereby increasing the potential for a collision to occur between an automobile and train.

5. A section entitled "Special Concerns" deals with pedestrian sidewalks, bicycle lanes, and street lighting located at the grade crossing. The presence of pedestrian and bicycle facilities may impact operations at the crossing and should be carefully considered in selecting appropriate warning devices or other forms of traffic control.

6. The final section of the data collection form, "Sight Distance Characteristics," addresses the problem of sight distance on the approaches to the crossing. Sight distance data is not included in either the National Inventory or the Texas crossing database. However, the importance of sight distance to safety at the crossing cannot be denied.

A computer database can be easily constructed based on the structure of this new data collection form. Some potential applications of the database include:

1. Placement or upgrading of railroad crossbucks and grade crossing advance warning signs;

2. Identification of crossings where advisory speed signing is appropriate, and determining advisory speeds for these locations;

3. Identification of locations for crossing surface improvement programs, such as in the case of replacing a sectional timber crossing with a rubber or prefabricated concrete crossing; and

4. Elimination of sight distance obstructions on railroad or highway right-of-way.

RAILROAD-HIGHWAY GRADE CROSSING DATA COLLECTION FORM

GRADE CROSSING LOCATION AND IDENTIFICATION	
USDOT ID NUMBER:	STREET / ROAD / HIGHWAY NAME:
STREET / ROAD / HIGHWAY TYPE:	CITY / NEAREST CITY:
COUNTY:	STATE:
OPERATING RAILROAD:	RAILROAD MILEPOST:

EXISTING TRAFFIC CONTROL DEVICES AT GRADE CROSSING	
WARNING AND REGULATORY SIGNS	
NUMBER OF ADVANCE WARNING SIGNS:	TYPE:
NUMBER OF CROSSBUCKS:	
NUMBER OF REFLECTORIZED CROSSBUCKS:	NUMBER OF DOUBLE-SIDED CROSSBUCKS:
NUMBER OF STOP SIGNS:	
NUMBER OF OTHER SIGNS:	TYPE:
TRAIN-ACTIVATED WARNING DEVICES	
NUMBER OF MAST-MOUNTED FLASHING LIGHT SIGNALS:	
NUMBER OF CANTILEVER-MOUNTED FLASHING LIGHT SIGNALS:	
NUMBER OF FLASHING LIGHT SIGNALS WITH GATES:	
NUMBER OF OTHER ACTIVE WARNING DEVICES:	TYPE:
PAVEMENT MARKINGS	
NUMBER OF STOPLINES:	
NUMBER OF RXR SYMBOLS:	

GRADE CROSSING LOCATION CHARACTERISTICS	
URBAN RURAL	RESIDENTIAL COMMERCIAL INDUSTRIAL

RAILROAD OPERATING CHARACTERISTICS	
NUMBER OF MAIN TRACKS: DAILY TRAIN MOVEMENTS:	NUMBER OF OTHER TRACKS: DAILY TRAIN MOVEMENTS:
MAXIMUM TIMETABLE SPEED FOR MAIN TRACK:	MAXIMUM TIMETABLE SPEED FOR OTHER TRACKS:
NUMBER OF DAILY PASSENGER TRAINS:	MAXIMUM PASSENGER TRAIN SPEED:

ROADWAY / HIGHWAY CHARACTERISTICS	
NUMBER OF TRAFFIC LANES:	DAILY TRAFFIC VOLUME:
POSTED SPEED LIMIT:	COMFORTABLE APPROACH SPEED:
HIGHWAY SURFACE:	
HORIZONTAL CURVES:	
VERTICAL CURVES:	

GRADE CROSSING CHARACTERISTICS	
CROSSING WIDTH:	ANGLE OF INTERSECTION:
APPROACH GRADES APPROACH 1:	APPROACH 2:
TYPE OF CROSSING SURFACE SECTIONAL TIMBER FULL WOOD PLANK ASPHALT CONCRETE PREFABRICATED	CONCRETE CAST-IN-PLACE RUBBER METAL GRAVEL

ADJACENT HIGHWAY INTERSECTIONS	
TYPE OF INTERSECTION:	TYPE OF ROADWAY:
DISTANCE FROM GRADE CROSSING:	TRAFFIC CONTROL AT INTERSECTION:

FIGURE 2. RAILROAD-HIGHWAY GRADE CROSSING DATA COLLECTION FORM.

SPECIAL CONCERNS	
PEDESTRIAN SIDEWALKS	BICYCLE LANES
STREET / HIGHWAY LIGHTING	
OTHER	

CROSSING ACCIDENT HISTORY		
ACCIDENTS	FATALITIES	INJURIES

SIGHT DISTANCE CHARACTERISTICS
QUADRANT 1
SAFE STOPPING DISTANCE (measured from stop line):
REQUIRED SIGHT DISTANCE ALONG TRACKS:
DOES AVAILABLE SIGHT DISTANCE ALONG TRACK EQUAL OR EXCEED REQUIRED?
IF NO, WHAT IS AVAILABLE SIGHT DISTANCE ALONG TRACK?
FROM WHAT POINT ALONG HIGHWAY (measured from stop line) IS SIGHT DISTANCE REQUIREMENT ALONG TRACKS MET?
LIST / DESCRIBE OBSTRUCTIONS:
QUADRANT 2
SAFE STOPPING DISTANCE (measured from stop line):
REQUIRED SIGHT DISTANCE ALONG TRACKS:
DOES AVAILABLE SIGHT DISTANCE ALONG TRACK EQUAL OR EXCEED REQUIRED?
IF NO, WHAT IS AVAILABLE SIGHT DISTANCE ALONG TRACK?
FROM WHAT POINT ALONG HIGHWAY (measured from stop line) IS SIGHT DISTANCE REQUIREMENT ALONG TRACKS MET?
LIST / DESCRIBE OBSTRUCTIONS:

SIGHT DISTANCE CHARACTERISTICS (CONTINUED)
QUADRANT 3
SAFE STOPPING DISTANCE (measured from stop line):
REQUIRED SIGHT DISTANCE ALONG TRACKS:
DOES AVAILABLE SIGHT DISTANCE ALONG TRACKS EQUAL OR EXCEED REQUIRED?
IF NO, WHAT IS AVAILABLE SIGHT DISTANCE ALONG TRACKS?
FROM WHAT POINT ALONG HIGHWAY (measured from stop line) IS SIGHT DISTANCE REQUIREMENT ALONG TRACKS MET?
LIST / DESCRIBE OBSTRUCTIONS:
QUADRANT 4
SAFE STOPPING DISTANCE (measured from stop line):
REQUIRED SIGHT DISTANCE ALONG TRACKS:
DOES AVAILABLE SIGHT DISTANCE ALONG TRACKS EQUAL OR EXCEED REQUIRED?
IF NO, WHAT IS AVAILABLE SIGHT DISTANCE ALONG TRACKS?
FROM WHAT POINT ALONG HIGHWAY (measured from stop line) IS SIGHT DISTANCE REQUIREMENT ALONG TRACKS MET?
LIST / DESCRIBE OBSTRUCTIONS:

FIGURE 2. RAILROAD-HIGHWAY GRADE CROSSING DATA COLLECTION FORM (CONTINUED).

SUMMARY OF DATA COLLECTION

To satisfy Task 2 of the study procedure, data was collected at each public grade crossing in Brazos County with passive warning devices. A list of all public crossings in Brazos County was obtained from the Texas Department of Transportation to aid in identifying and locating these passive crossings.

Sight distance data was collected at 18 public crossings with passive warning devices in Brazos County. Generally, these crossings were selected at random. However, inactive crossings and crossings located on railroad spur tracks were excluded. An effort was made to obtain data at both urban and rural sites and on low-volume and high-volume roadways.

Sight distance requirements at each study site were based upon a sight distance table developed as part of this research, illustrated in Figure 3. The format of this table is based upon the grade crossing sight distance table presented in the 1984 edition of "A Policy on the Geometric Design of Highways and Streets," published by the American Association of State Highway and Transportation Officials (AASHTO). However, the assumptions and equations used to derive the distances shown in Figure 3 differ significantly from those used by AASHTO. The values in the AASHTO table are very conservative due to the use of a design vehicle length of 65 feet and the requirement that the vehicle is able to either stop short of a hazard zone beginning 15 feet from the tracks, or that it is able to safely cross the tracks and clear the 15-foot hazard zone on the opposite side prior to the arrival of the train at the crossing. The sight distance table used for this study was developed with the requirement that the vehicle come to a complete stop short of the hazard zone prior to the arrival of the train. Thus, vehicle length was not a factor in the development of this table. The basic equations used in the derivation of this table are as follows:

$$D_H = 1.47 * V_V * (P\&R) + BD + D + d_e \qquad (1)$$

and

$$D_T = 1.47 * V_T * \left[(P\&R) + \frac{BD}{0.5 * 1.47 * V_V} \right] \qquad (2)$$

where:

D_H = distance along highway from crossing
= safe vehicle stopping distance
D_T = distance along railroad from crossing
V_V = velocity of highway vehicle
(P&R) = driver perception / reaction time
= 2.5 sec
BD = vehicle braking distance
= VV2 / (30 * f)
f = coefficient of friction
D = length of hazard zone
= 15 feet
d_e = distance from front of car to driver's eye
= 10 feet
V_T = velocity of train

Train Speed (MPH)	Vehicle Speed (MPH) 5	10	15	20	25	30	35	40	45	50	55	60
	Distance Along Railroad From Crossing (FT)											
10	45	53	62	70	81	94	105	120	133	148	159	175
20	90	107	123	140	161	188	211	240	267	296	318	349
30	135	160	185	210	242	281	316	360	400	443	477	524
40	180	213	247	280	322	375	421	480	534	591	636	698
50	225	267	308	350	403	469	526	600	667	739	794	873
60	270	320	370	420	483	563	632	720	801	887	953	1048
70	315	373	432	490	564	657	737	840	934	1034	1112	1222
80	360	427	493	560	644	750	842	960	1068	1182	1271	1397
90	405	480	555	630	725	844	948	1080	1201	1330	1430	1571
	Distance Along Highway From Crossing (FT)											
FRICT	45	70	99	132	171	221	273	338	408	486	563	659
	0.4	0.4	0.4	0.4	0.38	0.35	0.34	0.32	0.31	0.3	0.3	0.29

FIGURE 3. GRADE CROSSING SIGHT DISTANCE TABLE.

Three lines of sight were investigated in each quadrant at each study site. These are depicted in Figure 4. The first was the required line of sight from a vehicle on the highway to an approaching train and was based upon the maximum operating speed on the railroad track and the posted speed limit or comfortable approach speed of the highway, whichever was greatest. The second line of sight extended from a point on the highway located at the safe stopping distance to a point on the railroad tracks located as far down the tracks as the driver could see with an unobstructed view. The third line of sight began at the location down the tracks for specified train and vehicle speeds and extended to the point on the highway at which the driver had an unobstructed view of a train at that location. For crossings at which the required sight distance was met or exceeded, an investigation of the second and third lines of sight was not required. However, if an obstruction existed, then these were studied in order to establish the precise location of the obstruction.

Modification of Hazard Index Formula

A modification of the hazard index formula was developed in an attempt to incorporate a sight distance variable into this calculation. This modification was essentially the inclusion of a sight distance rating in the denominator of the hazard index formula, as demonstrated by the general equation:

$$\text{Modified Hazard Index} = \frac{V * T * S * P_f * A_5^{1.15} * K}{\text{sight distance rating}} \quad (3)$$

The rating was actually a ratio that varied according to the availability of the required sight distance for each of the four quadrants at the crossing. The ratio was calculated such that its maximum value was 1.0. For a crossing with severely restricted sight distance, the ratio would be very small, 0.1 for instance. At a location with fairly good sight distance, the ratio might equal 0.8 or 0.9. If the required sight distance was available, the ratio would be equivalent to 1.0. By placing the sight distance rating in the denominator of the hazard index formula, it was expected that the hazard index for locations with poor sight distance would be increased significantly, whereas the index for locations with good or excellent sight distance would change very little or none at all.

The sight distance rating was calculated by four methods. For the first two of these methods, it was necessary to determine the length of the driver's line of sight from a point on the highway defined by the safe stopping distance to the furthest unobstructed point along the tracks. This was done for each of the four quadrants at each study site. Method One defined the sight distance rating as the ratio of the available sight distance to the required sight distance for the quadrant with the most restricted sight distance. Method Two defined the sight distance rating as the average of the ratios of available sight distance to required sight distance for all four quadrants. The remaining two methods involved determination of the unobstructed area of each quadrant for each study site. The unobstructed area was defined as the four-sided area bounded by the roadway, the railroad tracks, the available line of sight from the safe stopping distance to the tracks, and the line of sight between the roadway and a

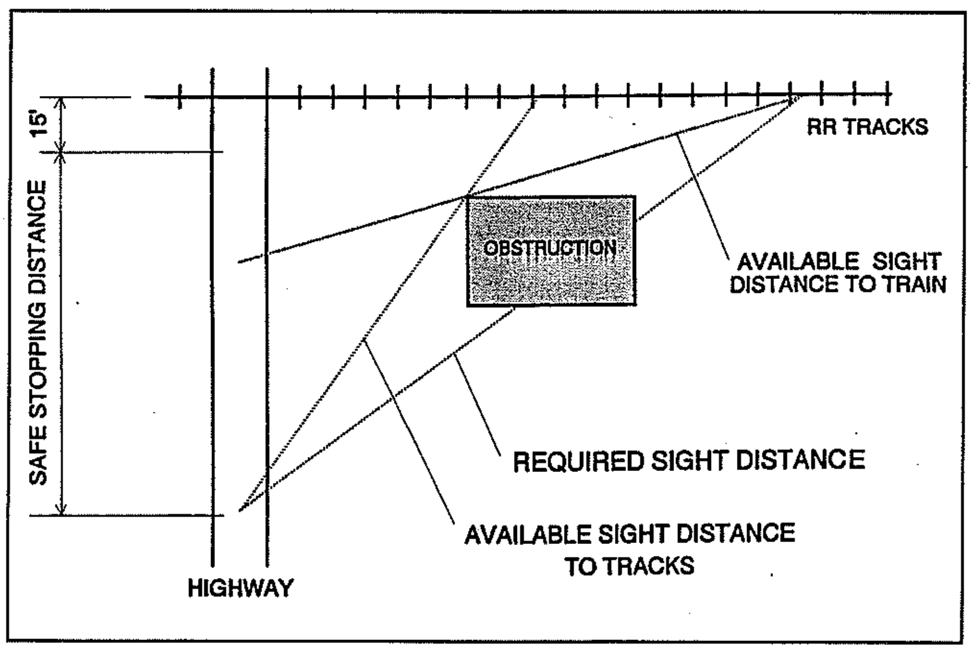

FIGURE 4. SIGHT DISTANCE TRIANGLES.

train located at the distance down the tracks defined by the vehicle's safe stopping distance and the maximum train operating speed. The required unobstructed area was defined as the area bounded by the roadway, railroad tracks, and required line of sight from the roadway to the tracks. The ratios were calculated by dividing the unobstructed area by the required unobstructed area. It was reasoned that a quadrant relatively free of obstructions would result in a sight distance rating close to 1.0, and a quadrant with many obstructions would have a very small rating due to the small unobstructed area. Method Three defined the sight distance rating as the ratio of the unobstructed area to the required unobstructed area for the quadrant with the most restricted sight distance. Method Four considered the average of the ratios for all four quadrants at each study site.

RESULTS

A modified hazard index was calculated by each of the methods previously described for each of the 18 study sites. The study sites were then ranked according to their modified hazard index values and compared to the ranking by Texas hazard index value to determine whether or not a significant change had occurred in the rankings. The results of these rankings and comparisons are presented in Tables 1 - 4.

Examination of Tables 1 - 4 reveals that none of the four methods produced a drastic change in the ranking of the 18 study sites. The position of many of the study sites within the rankings did change, but rarely by more than one or two places. There were a few exceptions to this, which indicates that the basic concept of the sight distance rating may have some merit. However, the modifications of the hazard index calculation reported herein do not appear to produce significant results.

CONCLUSIONS AND RECOMMENDATIONS

The use of a sight distance rating in the denominator of the hazard index formula may be an effective means of identifying crossings with sight distance problems and placing them higher within the priority ranking. Further research utilizing a greater number of crossings than was considered by this study is required to state conclusively whether or not this is the case. A modification of the sight distance rating, possibly through the addition of an adjustment factor, may be necessary. It is not clear which of the four methods of calculating the sight distance rating is preferable. It is recommended that the potential benefits of using a sight distance rating in the denominator of the hazard index formula and the possible means of calculating this rating continue to be investigated.

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TABLE 1. COMPARISON OF RANKINGS BY TEXAS HAZARD INDEX FORMULA AND METHOD 1.

Study Site	Texas Hazard Index	Rank	Modified Hazard Index	Rank
L	1656	1	3551	1
N	1088	2	2490	2
J	442	3	1038	3
O	396	4	640	7
H	363	5	1001	4
M	306	6	890	5
Q	236	7	374	8
K	227	8	609	6
P	200	9	289	12
E	176	10	351	9
I	136	11	338	10
G	110	12	249	11
F	85	13	248	13
D	75	14	186	14
A	33	15	83	15
R	25	16	69	16
B	8	17	13	18
C	8	17	23	17

TABLE 2. COMPARISON OF RANKINGS BY TEXAS HAZARD INDEX FORMULA AND METHOD 2.

Study Site	Texas Hazard Index	Rank	Modified Hazard Index	Rank
L	1656	1	2221	1
N	1088	2	1492	2
J	442	3	616	4
O	396	4	584	5
H	363	5	929	3
M	306	6	453	6
Q	236	7	343	8
K	227	8	330	9
P	200	9	281	10
E	176	10	351	7
I	136	11	193	11
G	110	12	157	13
F	85	13	193	11
D	75	14	102	14
A	33	15	82	15
R	25	16	43	16
B	8	17	9	18
C	8	17	10	17

TABLE 3. COMPARISON OF RANKINGS BY TEXAS HAZARD INDEX FORMULA AND METHOD 3.

Study Site	Texas Hazard Index	Rank	Modified Hazard Index	Rank
L	1656	1	9018	1
N	1088	2	6139	2
J	442	3	1366	6
O	396	4	2673	3
H	363	5	2468	4
M	306	6	1064	8
Q	236	7	2465	5
K	227	8	509	12
P	200	9	1357	7
E	176	10	363	13
I	136	11	850	9
G	110	12	581	10
F	85	13	510	11
D	75	14	189	15
A	33	15	320	14
R	25	16	78	16
B	8	17	12	18
C	8	17	17	17

TABLE 4. COMPARISON OF RANKINGS BY TEXAS HAZARD INDEX FORMULA AND METHOD 4.

Study Site	Texas Hazard Index	Rank	Modified Hazard Index	Rank
L	1656	1	2716	1
N	1088	2	1638	3
J	442	3	663	6
O	396	4	1757	2
H	363	5	1538	4
M	306	6	465	8
Q	236	7	1111	5
K	227	8	307	10
P	200	9	605	7
E	176	10	267	11
I	136	11	225	12
G	110	12	167	14
F	85	13	341	9
D	75	14	104	15
A	33	15	189	13
R	25	16	48	16
B	8	17	9	18
C	8	17	10	17

Evaluation of an Inside Merge at a Major Freeway-Freeway Connection

P. SCOTT BEASLEY

The effectiveness of the inside merge configuration as an alternative two-lane entrance ramp design has been questioned, and current literature has not sufficiently addressed the subject. This paper presents the results of an evaluation performed on a freeway to freeway connection that has an inside merge design. The evaluation includes observations from data collected on site, as well as a simulation analysis on the existing inside merge configuration, and an alternative, the exterior parallel merge configuration.

INTRODUCTION

Continuing efforts to improve operational characteristics on urban freeway facilities involves researching and understanding different designs so that new facilities can be built to better accommodate future demands. Before a new facility is constructed, the designer must choose one design among several alternatives that meets all requirements necessary for good operation. Even though one design cannot be used universally, there are usually certain designs or configurations that are considered more acceptable for particular conditions.

One example of a type of freeway facility which has more than one configuration is a multi-lane ramp terminal or freeway to freeway connection. There are different designs for multi-lane ramp or freeway connections with high volume freeway facilities. Perhaps the most simple design which poses very few operational problems is a continuation of all entrance lanes. In this case no lanes are dropped and a merge situation is avoided. However, capacity requirements may not warrant the use of this arrangement. An alternative to this design is to drop one ramp lane, resulting in a merge situation.

In the case of two-lane entrance ramps and freeway to freeway connections, lane drops can be accomplished with several configurations. Questions have arisen as to which configuration is more operationally efficient. Which lane should be dropped, the inside or the outside ramp lane? Some professionals believe the inside merge arrangement is less efficient and less safe because of the possibility of entrapment between two traffic streams. A recent study performed at the Texas Transportation Institute supports this opinion (1). Even AASHTO policy regarding these two merge configurations has changed, as is explained later in the paper. The operational characteristics of these merge configurations have not been fully evaluated. The objective of this research, therefore, is to study and evaluate a high-volume multi-lane merge configuration.

Problem Statement

The problem lies in the inadequate understanding of the operational characteristics of inside and outside merge configurations. Alternative designs for multi-lane entrance ramps and freeway-freeway connections have not been fully evaluated; conclusions concerning the effectiveness of both inside and outside merge configurations are mostly based on professional or public opinion. It is desirable, therefore, to have more information available on the operational characteristics of the alternative merge configurations for multi-lane entrance facilities.

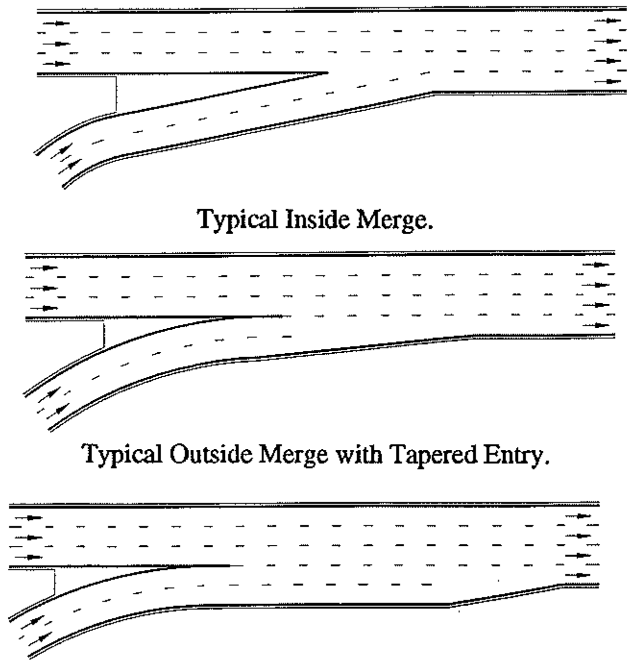
Background

There are two basic designs for merging a high volume multi-lane ramp or freeway with high volume freeway mainlanes:

1. No lanes dropped downstream of the merge: Connecting a two-lane ramp or a two-lane freeway with another freeway without dropping any lanes poses little or no problem to the operation of the freeway.

2. One lane dropped downstream of the merge: Two configurations result from dropping a ramp lane: an inside merge and an outside merge, including tapered and parallel type entry (See Figure 1).

This study examines the effects that the inside merge configuration has on freeway operations. Because there is a limited amount of information available on the effects of two-lane entrance facilities on freeway operations, design is based mostly on professional and public opinions.



Typical Inside Merge.

Typical Outside Merge with Tapered Entry.

Typical Outside Merge with Parallel Entry.

FIGURE 1. ALTERNATIVE MERGE CONFIGURATIONS FOR TWO-LANE ENTRANCE RAMPS.

Ronald C. Pfefer provided some information on two-lane merge configurations in a 1968 Traffic Engineering article indicating that the inside merge was the preferred alternative two-lane ramp design (2). The article offers a summary of comments on different merge configurations, including comments on the inside merge. The inside merge configuration was favored over the exterior merge design but was criticized most for the possibility of vehicle entrapment.

AASHTO's current recommended practice allows the use of both the inside and the outside merge arrangements. According to AASHTO, both configurations are satisfactory when used exclusively in a region or on a freeway system but should not be intermixed. However, AASHTO's design preference has changed in recent years. In the 1984 edition of "A Policy On Geometric Design of Highways and Streets," AASHTO prefers the inside taper merge design over the outside (exterior) parallel merge design (3). The current (1990) edition, however, indicates the exterior parallel design as the preferred alternative (4). Reasons for this change are not given.

A recent study entitled an "Operational Evaluation of Effects Resulting From Freeway-Freeway Interchange Geometrics" performed at Texas A&M by the Texas Transportation Institute examined three merge configurations for two-lane entrance ramps (1). These configurations included:

1. Interior taper merge,
2. Exterior taper merge, and
3. Exterior parallel merge designs (Refer to Figure 1).

The three merge configurations were simulated using INTRAS (INtegrated TRaffic Simulation), a microscopic freeway simulation model. The measure of effectiveness used to compare the simulation results was the traffic density in the merge section. To illustrate the results, density (pc/ln-mile) was plotted against traffic demand (vph) on the merge section. The results of this study indicate that both exterior merge configurations performed more soundly relative to the interior merge configuration. Acceptable levels of operation (level of service C or better) were obtained only at very low traffic demands for the interior "chicken" merge. Furthermore, the exterior parallel configuration proved to have an advantage over the other two types of merge configurations. Their results indicated that the exterior parallel merge operated more efficiently at higher demands than did the interior and exterior taper configurations.

Objectives

Because data on high-volume two-lane merge configurations is limited, there is a need for a study in which the operations of these configurations are examined. The main objective of this study is to perform an experiment of this nature. Specifically, this study is concerned with describing the operations of an inside merge configuration. As discussed above, a previous study performed at the Texas Transportation Institute (TTI) indicated that outside merge configurations were more operationally sound. Since this study was performed using simulated data, the results give only the relative benefits of one configuration over another. This study will use actual field data in the analysis of an inside merge configuration and will utilize FRESIM, a microscopic freeway simulation program, to aid in the analysis of alternative merge configurations (5).

METHODOLOGY

Field Study

Site Description - Data was collected from a study location prior to the start of this study. The site was used in a previous study; it was also used for this study because it had the required characteristics. That is, the site was chosen because of its inside merge configuration. Located in south Dallas, the site is where northbound U.S. 67 and I.H. 35 merge. Two-lane U.S. 67 merges with three-lane I.H. 35 on the left hand side (Figure 2). Although it resembles a left-handed two-lane ramp connection with a three-lane freeway, it should be noted that both roadways are high-volume, high-speed freeways. The inside merge configuration combines the right lane of U.S. 67 with the left lane of I.H. 35 over a distance of about 250 feet. The location is known to become congested during peak periods.

Data Collection - An 8mm video camera was placed downstream of the merge on the shoulder of the southbound roadway. Filming took place on Thursday, May 2, 1991 for a period of two hours beginning at 6:30 a.m. Simultaneously, a series of coupled loop detectors located approximately 600 feet downstream of the merge gore recorded raw data from each passing vehicle. Both the videotaped footage and the loop detector data were used for the operational analysis of the study site.

Data Reduction - Using the videotape, traffic counts were obtained for each lane upstream of the connection. Volumes were recorded in 15-minute intervals during one hour and forty-five minutes of the two hour period. Truck volumes were also counted.

The reduction of the loop detector data began with converting it to a summarized form through the use of the Statistical Analysis Software (SAS) program. SAS generated graphs that illustrated flow rate and speed relationships over the two hour period. Flow rates and speeds were computed for every 15-minute interval (moving one minute at a time) over the 2-hour period. Densities were also computed from the raw data. All relationships and computations were either tabulated or graphed. Visual observations and count data were used to verify the SAS graphs and the density calculations.

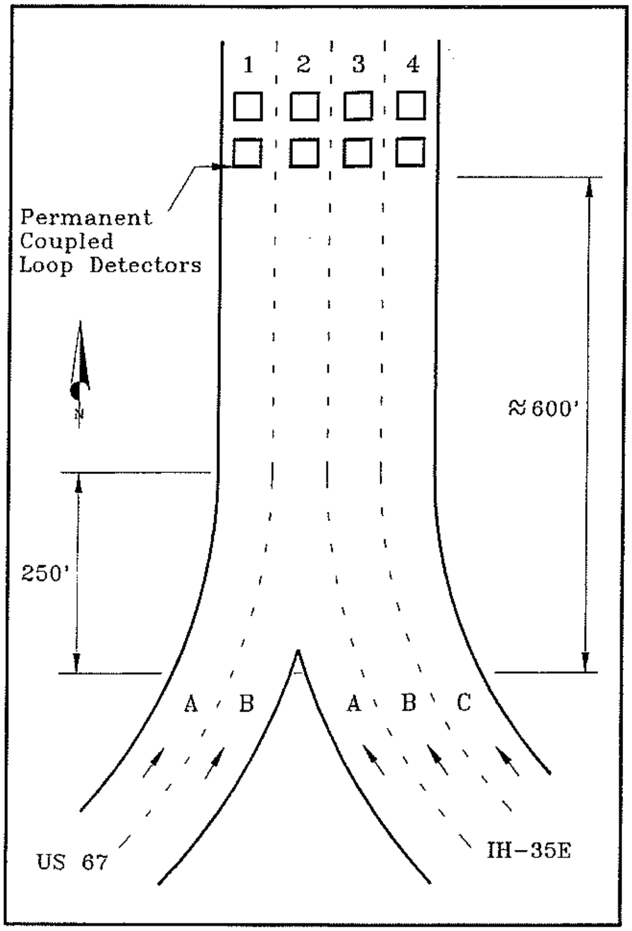


FIGURE 2. SCHEMATIC OF STUDY SITE

Observations - Although a visual observation is not enough to draw conclusions, it can give a general understanding of the operations of a particular freeway segment. An initial observation of the videotape revealed high volume conditions. Some exceptionally high flow rates were observed for the left lane of U.S. 67 during all periods and for I.H. 35 for some periods. The heaviest volumes were in the left lane of U.S. 67 and the two right lanes of I.H. 35. It appeared that drivers avoided the merge lanes (the right lane of U.S. 67 and the left lane of I.H. 35); although during the lower volume periods, merge operations were quite smooth. Flow rates for these lanes support this observation.

As time progressed and volumes increased, merge operations began to deteriorate. A queue began to form predominately on U.S. 67 around 7:17 a.m. and began to release around 7:58 a.m. The site became very congested; merge operations became inefficient, as merging vehicles could not find gaps in the traffic stream and no outlets were available due to the heavy flows on either side of the merge. The flow was reduced to stop-and-go conditions for the merging lanes, while adjacent lanes moved slowly. An accident occurred in the right lane of U.S 67 just upstream of the merge during the time period from 7:45 a.m. to 8:00 a.m. Operations were not affected by this accident, however, because the drivers moved their vehicles out of the roadway into the gore area.

One other observation deserving mention regards yield behavior. The connection of two freeways in a merge configuration introduces the question, "Who is supposed to yield?" In everyday driving when faced with a merge situation, the yielding driver will sometimes indicate a movement into a gap with a turn signal. Observation of the videotape revealed drivers on either side of the merge signaling to announce their movement into the through-stream. This observation may indicate "no answer" to the question asked above.

Field Study Findings - Table 1 summarizes the flow rates just upstream of the merge that were computed from the 15-minute counts. The maximum 15-minute flow rates recorded for a through-lane and a merge lane were 2708 vph and 1880 vph, respectively, both on U.S. 67. Examination of this table indicates a lower utilization of the two merge lanes as compared to flow rates recorded on the through lanes.

The SAS program summarized data taken from the loop detectors located downstream of the merge. Figure 3 illustrates the moving 15-minute flow rates for a typical lane. This graph shows the peak flow to occur between 7:00 a.m. to 7:30 a.m. A general trend was observed in which the peak flow rates decrease across each lane from left to right, the heaviest peak flow being in lane 1 and the lightest peak flow being in lane 4.

Figure 4 shows the 15-minute moving average speeds for a typical lane. The figure shows a clear pattern of speed deterioration over the congested periods, with free flow speeds being regained at approximately 8:00 a.m. Lane 2 (the combination of merge lanes) speeds were consistently lower than all other lane speeds during the heaviest flows.

Table 2 summarizes the calculated densities for each lane downstream of the merge, as well as the average across all four lanes. These densities were calculated from the raw data in 15-minute time intervals from 6:30 a.m. until 8:00 a.m. Examination of Table 2 shows a breakdown in operations between 7:15 a.m. and 7:30 a.m., during which time the densities recorded for three of the four lanes corresponded to an operation at level of service (LOS) F in which merging is on a stop-and-go basis, as presented in the "Highway Capacity Manual" (HCM) (6). The merge process became very turbulent at high flow rates. Densities were typically heaviest in the two left lanes (lanes 1 and 2) and somewhat lighter in the two right lanes (lanes 3 and 4) during most time periods, until all lanes broke down. The densities typically decrease from left to right across all lanes.

Computer Study

Simulation - The existing inside merge and an alternative configuration, an outside parallel merge, were simulated using Fresim. Initially, a simulation model of the study site was constructed to represent the existing inside merge configuration. Because Fresim will not accept the merging of two freeways, the model was set up as a left-handed two-lane ramp connection to a three-lane freeway. Table 3 describes the coding used to construct the model which consisted of seven links. See Figure 5 for a link-node diagram of the site.

Several simulations were performed to try to calibrate the model to make the simulated output match the actual conditions within a reasonable range. The parameters used in the calibration of the model were densities and volumes. Initial attempts at calibration revealed that the model was losing vehicles; a substantial number of vehicles input to the model never showed up on the simulation output because queueing caused vehicles to back up. The queued vehicles did not have time to reach the end of the model before simulation time expired. It was necessary, then, to adjust the model until the number of vehicles input equaled the number of vehicles output and the simulated densities reflected the actual densities.

TABLE 1. FIFTEEN-MINUTE FLOW RATES (VPHPL) UPSTREAM OF THE MERGE.

Time	Lane A	Lane B	Total	Lane A	Lane B	Lane C	Total
6:30 - 6:45	2124	1468	3592	968	1724	1396	4088
6:45 - 7:00	2072	1480	3552	776	1652	1224	3652
7:00 - 7:15	2708	1880	4588	916	1916	1456	4288
7:15 - 7:30	2306	1648	4044	964	2112	2052	5128
7:30 - 7:45	2328	1488	3816	1068	2024	2096	5188
7:45 - 8:00	2308	1844	4152	652	1900	1956	4508
8:00 - 8:15	2200	1740	3940	640	1524	1480	3644

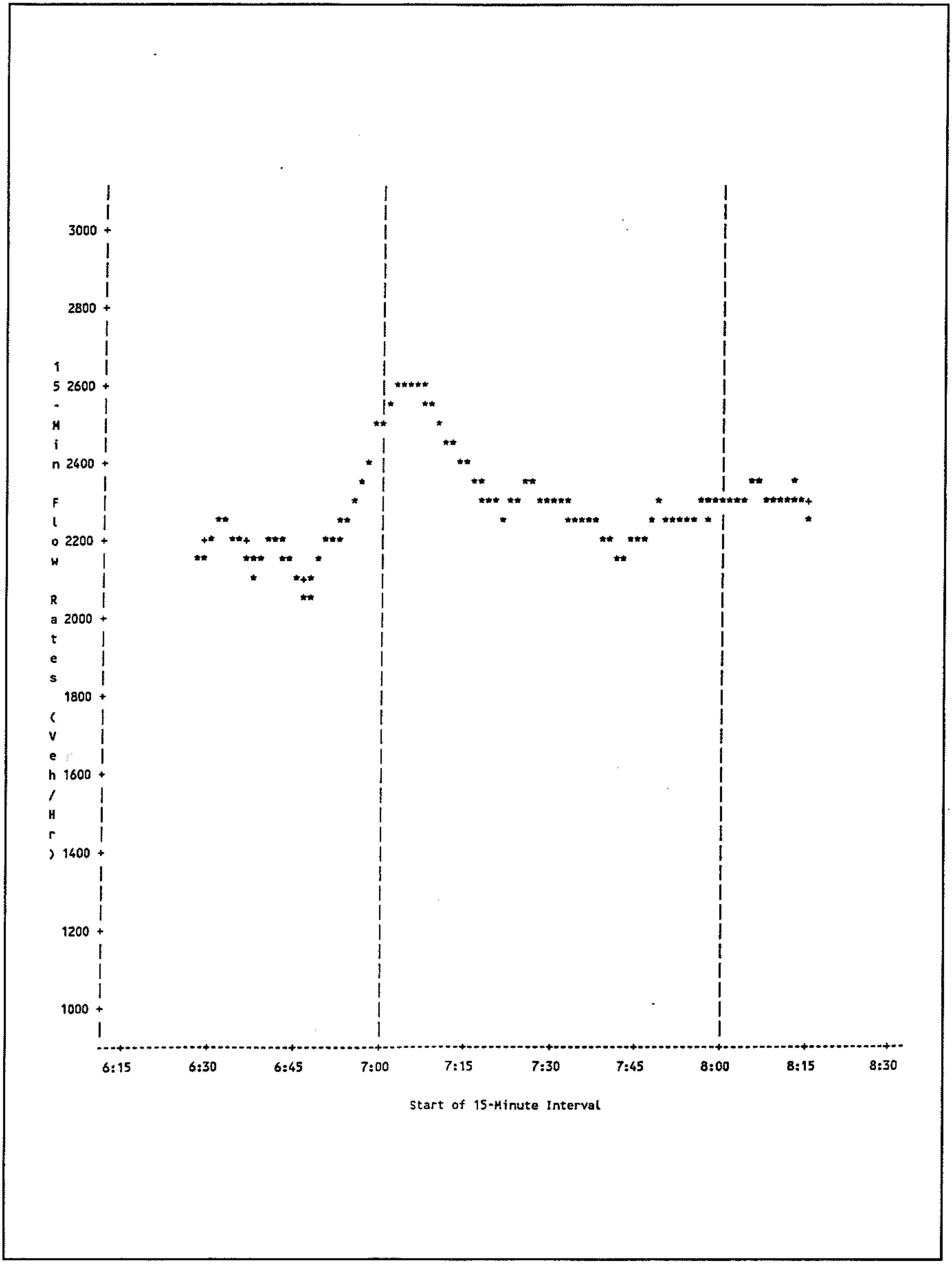


FIGURE 3. HOURLY FLOW RATES BASED ON 15-MINUTE MOVING AVERAGE FOR LANE 1.

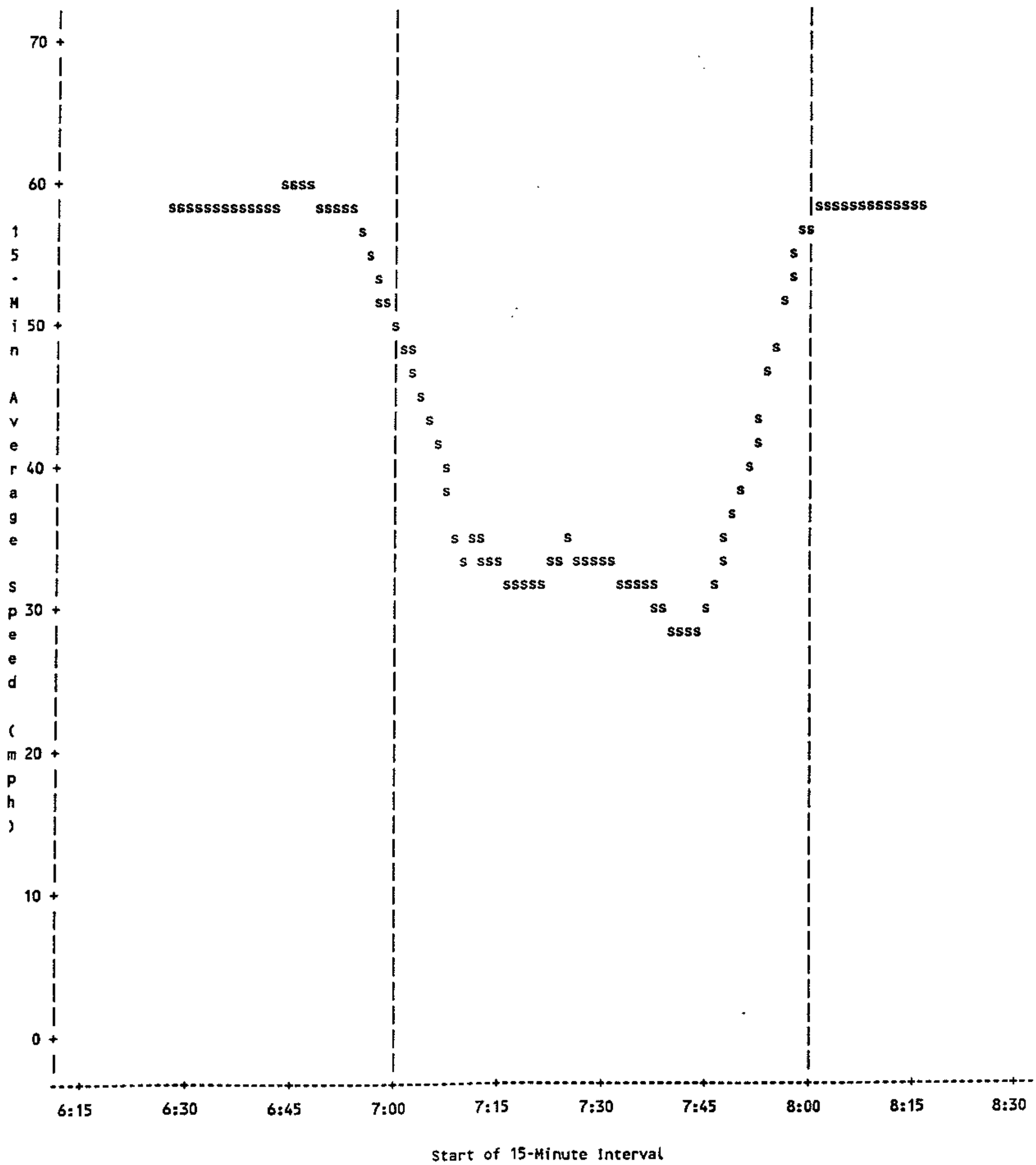


FIGURE 4. FIFTEEN-MINUTE MOVING AVERAGE SPEEDS FOR LANE 1.

TABLE 2. DENSITIES (PC/LN-MILE) FOR EACH LANE DOWNSTREAM OF MERGE.

Time	Density (pc/ln-mile) Lane 1	Lane 2	Lane 3	Lane 4	Average
6:30 - 6:45	37	34	30	23	31
6:45 - 7:00	37	34	29	23	31
7:00 - 7:15	46	49	40	32	42
7:15 - 7:30	76	76	67	65	71
7:30 - 7:45	71	82	79	75	77
7:45 - 8:00	80	83	76	82	80

TABLE 3. FRESIM LINK DESCRIPTION FOR INSIDE MERGE.

Link	Fresim Link Designation Upstream Node	Downstream Node	Description	Ramp/Freeway
1	8000	1	Freeway input link	Freeway
2	1	3	Link upstream of merge	Freeway
3	3	4	Merge link	Freeway
4	4	5	Link downstream of merge	Freeway
5	5	8004	Output link	Freeway
6	8001	2	Ramp input link	Ramp
7	2	3	Link upstream of merge	Ramp

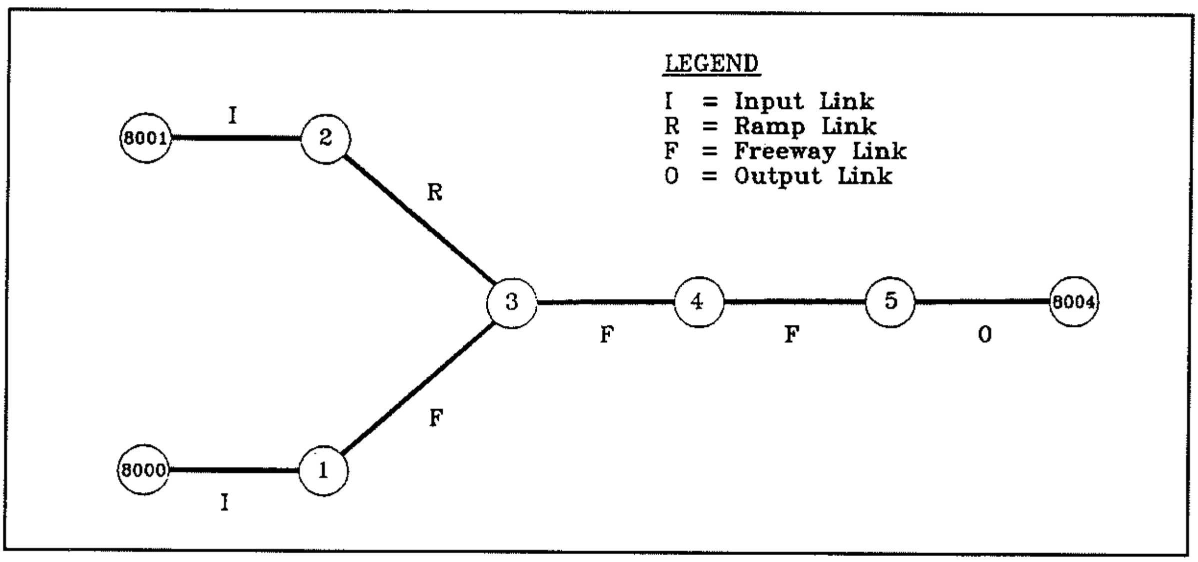


FIGURE 5. LINK-NODE DIAGRAM FOR THE INSIDE MERGE MODEL.

Simulations were performed for varying inside merge distances including 100 feet, the actual 250 feet, 1000 feet and finally 2000 feet because larger taper rates (longer merge distances) have been shown to improve merging operations.

A simulation model was also constructed for an outside parallel merge configuration. The construction of this model was more complex because there were more variables involved. Exterior lane lengths could be changed, as well as the position at which vehicles began to merge along the exterior lane. Simulations were performed using varying exterior lane lengths of 500, 1000, and 2000 feet.

Simulation Findings - The first simulation performed was the existing inside merge with the actual merge length of 250 feet. Repeated attempts to make the simulated output correspond with the actual conditions failed. Output volumes indicated that a small amount of vehicles were lost, however, the small number was not significant enough to change the computer calculations. The program calculated density cumulatively for entire sections, not by lane. Simple calculation gave the actual simulated density. A comparison of these densities to the actual densities averaged across all lanes (Table 2) revealed no similarities or patterns. The highest density calculated by the program (during the heaviest flow between 7:15 a.m. and 7:30 a.m.) was 46 pc/ln-mile. This value should have corresponded to the actual density at this time which was 71 pc/ln-mile. A comparison of the subsequent simulations using longer merge lengths showed no improvements over the shortest lengths.

Even though the original model could not be calibrated, simulations were still performed for the exterior merge configuration. Varying exterior lane lengths were used, however, the most interesting result was found by changing a parameter for an exterior lane length of 1000 feet. The model was changed to allow drivers to begin merging at three different positions along the exterior lane (at 300, 500 and 900 feet from the end of the lane). Operations improved as drivers were given more time and distance to merge.

RESULTS

Interpretation of Field Data

Data used in the analysis were flow rates, speeds and densities. Interpretation of the data were verified using observations of the videotape. These data were used to describe the operations of the facility so that interpretations could be made from them.

The most notable interpretation of the data regards the use of the merge lanes. Combining the two merge lane flows results in flows no less than 2256 vph and as great as 2796 vph. The convergence of two lanes into one lane resulted in volumes which overloaded the merge facility.

Operations began to deteriorate around 7:17 a.m. as queueing on U.S. 67 began to occur. Because flows in the lanes adjacent to lane 2 (the combination of the two merge lanes) became so heavy, merging vehicles had no outlet. Therefore merge behavior was forced, making operations inefficient. The lack of available gaps, combined with the inability of drivers to change lanes because of the heavy flows on either side of the merge, made operations of the inside merge configuration at this high volume location very inefficient. Speeds and densities recorded during the data collection support this interpretation.

Another interpretation of the results regards right-of-way in the merge area. If the study site were a two-lane entrance ramp terminal, the ramp traffic would theoretically yield to the through traffic. This study site, however, is the intersection of two high volume freeways. Merging becomes a complicated process. The merge configuration connects a high-volume freeway lane with a high-speed freeway lane. Which lane, then, has the right-of-way? Drawing any conclusions from the limited data collected for this study was impossible. The question, "Who is supposed to yield?" remains unanswered.

Interpretation of Computer Simulation

The simulation output of both models could not effectively be compared because of limitations of the FRESIM model and because differences in the models made comparing the two difficult. To begin with, because two freeway links may not feed the same link, the FRESIM program was unable to model the freeway-freeway connection (5). Freeway to ramp connections, however, could be modeled. Initially, then, the simulation output was inaccurate because the model was constructed as a two-lane left hand entrance ramp terminal with a three-lane freeway, although it is unclear how much this affected the simulation.

The model of the inside merge was insensitive to any changes in merge length. The FRESIM program, it appears, does not merge the vehicles over the merge link. Rather, the program seems to merge the vehicles at a point where the two freeway links (coded as a freeway and a ramp) connect. The exterior merge model, however, was more sensitive to changes in lane length because it has more parameters which could be altered to produce changes in operation. These limitations made comparison of the two configurations difficult.

SUMMARY

The purpose of this study was to describe the operations of an inside merge configuration at a freeway-freeway connection. The study was conducted on the basis that the information on two-lane merge configurations is limited. Also opinions on which merge configuration (inside "chicken" merge, outside taper and outside parallel) is more operationally sound are varied.

The field data and findings suggest that operations of the inside merge at the site chosen for analysis are inefficient. Flow rates, speeds, and densities indicated severe and unsatisfactory operating conditions during peak hours due to the inside merge's inability to handle the heavy flows. The inside merge design prevents the full utilization of two freeway lanes because they are converged into one. On a freeway to freeway connection the inside merge configuration violates drivers' expectancies because vehicles on the high-speed inside lane must merge with a slower moving lane.

The simulation output data of the existing insidemerge and an alternative, the exterior parallel merge, were not compared because of limitations in the Fresim programand because differences in the model's representation of the two alternative configurations made comparison difficult. The program was unable to accurately model or simulate an inside merge at a freeway to freeway connection. Operations of the exterior merge configuration, however, were more sensitive to changes in exterior lane length.

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A Study of Frontage Road Queuing, Vehicle Spacing and Applications to Freeway Exit Ramp Design Criteria

S. ROSS BLANCHARD

While extensive study has been conducted on the signal operations of urban diamond interchanges, little research has focused on frontage road/exit ramp activity. This junction of freeway exit ramp with the frontage road of a diamond interchange is a major factor in urban diamond interchange operations. Although freeway and frontage road traffic are not directly related, at the junction of the frontage road/exit ramp, the two flows interact directly. The presence of frontage road/exit ramp queues during high volume conditions demonstrates the need for research on the association between these elements. The typical approach for diamond interchange analysis includes computer simulation and field analysis of existing operations. PASSER III is one of the computer models designed for analyzing diamond interchanges. Unlike other traffic simulation models, it does not produce queuing output, a desired feature for analyzing an urban diamond interchange which experiences oversaturated conditions. It is presumed that PASSER II, a signalized arterial application, can simulate an isolated diamond; and if this is accomplished, the queuing output that PASSER II generates could be used for further study of exit ramp design criteria. For this reason, development of a queuing algorithm for PASSER III, using PASSER II techniques, would be ideal. To successfully relate computer simulation and field data analysis, it is necessary to identify a subject that comprises both aspects. In terms of diamond interchanges, there is a need to translate frontage road queue size prediction into queue length estimation for use in evaluating exit ramp separation distance design criteria. To accomplish this, a method for estimating vehicle spacing and thus queue lengths, must be developed.

INTRODUCTION

This report is a preliminary investigation into some of the aspects of diamond interchange operations, with emphasis on frontage road activity. The parent project that supported this research is an investigation of diamond interchange operations. In an attempt to offer a procedure for frontage road queue modelling, this paper will address the steps involved in the process of diamond interchange frontage road analysis, as well as the results, conclusions, and recommendations for future study into this subject. Also within the context of this report is the investigation into the concept of vehicle spacing. Results from these two areas of study will then be incorporated into an investigation of existing exit ramp design criteria.

Computer software such as PASSER II and PASSER III is being used extensively to model the operations of signalized intersections and diamond interchanges. In diamond interchange analysis, one of the areas that data is lacking, thus justifying the need for this research, is external frontage road queue modelling. The latest version of PASSER III-90, a software package designed specifically for diamond interchange analysis, currently does not perform frontage road queue calculations. PASSER II can be used to model an isolated diamond interchange and produce queuing output, but it is not specifically designed for diamond interchange analysis. For this reason a queuing algorithm could be included in future versions of PASSER III, thus providing another criteria for interchange evaluation-- frontage road/exit ramp activity.

A fundamental aspect of queuing is vehicle spacing. This is the space which a vehicle occupies while in a stationary queue. There is no value established as a vehicle spacing "constant" for queue calculations, unlike the headway between queued vehicles as given in chapter one of the "Highway Capacity Manual" section on interrupted flow (1). The PASSER III User's Guide suggests a value of twenty-five feet per vehicle for estimating interior queue storage capacity, however, this is not to be confused with queue size prediction (2). Such a value would become instrumental in urban interchange analysis, particularly analysis of frontage road activity.

If average vehicle spacing was used in conjunction with queue size estimates, an approximation could be made as to the length of vehicle queues. The significance of this result is the development of a practice for evaluating frontage road/exit ramp activity. When queues are present at this junction, concerns for safety and efficiency surface. Therefore, a need exists to investigate the use of these estimations to evaluate design standards for freeway exit ramps.

As stated previously, this report is part of a larger ongoing diamond interchange operations project. To keep within the scope of the parent project, the goal of this study is to conduct testing of queue modelling in the PASSER programs and investigate vehicle spacing. The objectives are outlined as follows:

1. Produce frontage road delay and queue size output using PASSER III-90 and PASSER II-90.
2. Investigation of a "constant" value for vehicle spacing.
3. Apply findings to exit ramp design criteria.

Lastly, as the title of this report suggests, the focus of the simulation and vehicle spacing analysis is on frontage road activity. Another aspect is the desire to differentiate between the operations of low and high volume conditions. Thus the emphasis is an investigation of oversaturated conditions on the frontage road. It is assumed (and later supported) that for all intensive purposes, the models would accurately model the key characteristics similarly during low volume conditions. What was in question was PASSER II's ability to model a diamond interchange accurately during oversaturated conditions. Also, there was a need to determine any relationships between vehicle spacing and traffic conditions, specifically the effects of high volume on vehicle spacing. It is shown later in Part 2, when queue length estimation is put into context with exit ramp design criteria, that this is an important factor.

OVERVIEW OF ANALYSIS AND METHODOLOGY

This report is presented in three sections. Part 1 will address the first objective: computer simulation using the PASSER models. Part 2 will pursue the second objective: the investigation of vehicle spacing and further applications to design criteria. And Part 3 will consist of a practical assimilation of the previous two topics by examining the applications to freeway exit ramp design standards.

Part 1 - Computer Simulation

This project involved extensive use of the latest versions(1990) of the PASSER II and PASSER III software programs. The primary objective was to generate matching output from both of the PASSER models, in order to determine how PASSER II could simulate an isolated diamond interchange in comparison to PASSER III. While it was known that this can be achieved, unanswered questions about the accuracy and correctness of the PASSER II output from this simulation remained.

The primary concern in this stage was the delay in the frontage road movements. Radwan demonstrated that both the PASSER models could produce similar delay output for various phasings using collected field data (3). In this report, however, the emphasis for the conventional diamond analysis was the comparison between four simulation models and the output generated by each model for the five standard phasing schemes. Obviously, it was a concern to accurately model all aspects of the simulated diamond; however, all other criteria remained subordinate to the frontage road characteristics.

Producing identical output from the computer models involved a two stage process. First, it was necessary to simulate the characteristics of a diamond interchange, the geometrics, and the signal control characteristics. Then it was necessary to correctly input the movement volumes and saturation flows. To limit the amount of variability in the simulation process, preliminary judgements were made as to the general input requirements. One conclusion was to use equal volume on each approach in oder to prevent any movement priority developing in the models that would affect the frontage road output. Also, the <F3> assistant was used in calculating saturation flows. Minimum phase durations used by PASSER III were assumed to five seconds for each movement. The value of ten seconds for the interior movement is attributed to the overlap present in the TTI lead phasing pattern.

PASSER III was used as the foundation for the computer simulation process, as it is designed specifically for diamond interchange analysis. Prior to the input procedure, it was determined that several assumptions were to be made for the PASSER III input data. The data for the three pertinent PASSER III input screens (General, Signal, Movement) are shown in Figures 1, 2, & 3. A brief overview of the input data is as follows; an 80 second cycle length was chosen, left turns were protected only, interior travel time was obtained from Table 2 provided in the PASSER III User's Guide, queue storage estimates were made using the suggested 25 feet vehicle length, and TTI lead phasing was used exclusively.

Output was generated using PASSER III, then after careful review, the optimum phase length data generated by PASSER III was used for input into PASSER II as the minimum phase lengths (doing so forces these phase lengths to be used by PASSER II). Input values are essentially the same as those used in PASSER III, it is the format that differs among the two. There were two steps that required careful consideration, the signal phasings and the movement volumes. This is where the ease of PASSER III input is most notable simply because it is designed specifically for diamond interchanges. Essentially, the PASSER II simulation translates into two closely spaced signalized intersections along an arterial, considered the interchange cross street in PASSER III analysis. The cross streets of the two intersections have volume in only one direction and thus represent the one-way frontage roads of the interchange. The input that was used for PASSER II is shown in Figures 4, 5, & 6.

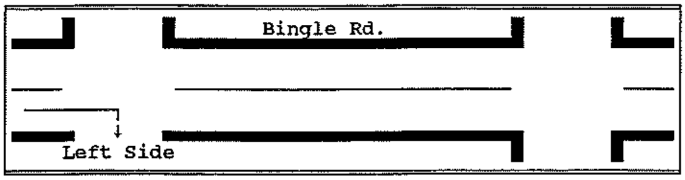
```
               P A S S E R   III  -  90            Version 1.00
Texas Department of Highways & Public Transportation

                FREEWAY IDENTIFICATION

Run Number: 01   Freeway Name: Interstate 10
District  : ??      City Name: Houston
Number of Interchanges  (1 for isolated analysis) :  1
        --------CYCLE LENGTHS----------
        Lower: 80  Upper:  80 Incr:

Calculate Band Split Proportional to Traffic?(Y/N):
"A" direction Percentage             ( 0 to 100 ):
Speed Search?                                (Y/N):


         Time/Space Diagram (Y/N):
------------PLOT SCALING / INCH------------------
Horizontal (Seconds):       Vertical (Feet):
    Note: Set to 0 for program selected values
```

FIGURE 1. PASSER III GENERAL INPUT SCREEN.

```
               P A S S E R   III  -  90            Version 1.00
   Texas Department of Highways & Public Transportation

                    INTERCHANGE  1 DATA

      Cross Street Name       Permitted Left Turns (Y/N)
        Bingle·Rd.··          Left-side: N  Right-side: N

     Interior-Travel-Time      Interior-Queue-Storage
      --Rgt-->    <--Lft--      Left-Side  Right-Side
       (Sec)        (Sec)       Left Thru  Left Thru
         6            6           8    8     8    8

                   SIGNAL PHASING DATA

Type of      Run Delay-Offset   ----- Seconds Offset -----
Phasing      Analysis (Y/N)     Forced Int.     Forced Ext.
-------           -----         -----------     -----------
Lead-Lead           N
Lag-Lead            N
Lead-Lag            N
Lag-Lag             N
TTI-Lead            Y
```

FIGURE 2. PASSER III SIGNAL INPUT SCREEN.

PASSER III - 90 Version 1.00
Texas Department of Highways & Public Transportation



Vol/ Hour	Sat Flow	Min Phase	CROSS-STREET	Vol/ Hour	Sat Flow	Min Phase
100	1665		right-turn	100	1665	
100	1665	5	straight-through	100	1665	5
100	1800		straight-then-left	100	1800	
			FRONTAGE ROAD			
100	1665		right-turn	100	1665	
100	1665	5	straight-through	100	1665	5
100	1710		left-then-straight	100	1710	
0	0		left-then-left (U)	0	0	
			INTERIOR			
100	1710	10	left-turn	100	1710	10
200	3600		straight-through	200	3600	

FIGURE 3. PASSER III MOVEMENT INPUT SCREEN.

Part 2 - Investigation of Vehicle Spacing

A study of vehicle queuing includes several factors which are associated with other aspects of traffic operations. The characteristic in question in this analysis was the distance that queued vehicles occupy. Obviously, vehicle and driver characteristics vary widely, and there was difficulty in developing general characteristics. In fact, with growing traffic activity and the changing characteristics, particularly size of passenger cars, there was a need to "update" previous research into the concept of vehicle spacing. For this project it was initially assumed that data collection and analysis would yield a standard measurement of vehicle spacing. That is, vehicle spacing would not be site specific, and the variability would be accounted for by driver tendencies.

An initial hypothesis was to examine the legitimacy of a constant value for vehicle spacing to be used in conjunction with the PASSER II maximum queue size predictions from Part 1. To facilitate analysis and applications to this research, data collection was made on the frontage roads of an urban diamond interchange rather than at random locations, ie. a mix of intersections and interchanges. An isolated urban diamond interchange in Houston was chosen as the source of data because it is the subject of a related current project as well as its urban location and traffic characteristics. The interchange was at Interstate Highway 10 and Bingle Road in Houston, shown in Figure 7. During rush periods as the volume on the freeway increases, inherently, the volume along the frontage road increases thus creating extremely long vehicle queues. Both A.M (inbound) and P.M. (outbound) periods were observed on July 18th, 1991.

The process of collecting data involved vehicle counts as well as queue length measurement. This was accomplished by selecting a random number of queued vehicles and measuring the distance occupied by the queue. The formula is shown below:

$$\text{Vehicle Spacing} = \frac{\text{Length of Queue (ft.)}}{\text{No. Vehicles in Queue}}$$

where:

Length of Queue = Distance measured from stop line to front of Nth vehicle; and
No. Vehicles in Queue = (N-1) vehicles.

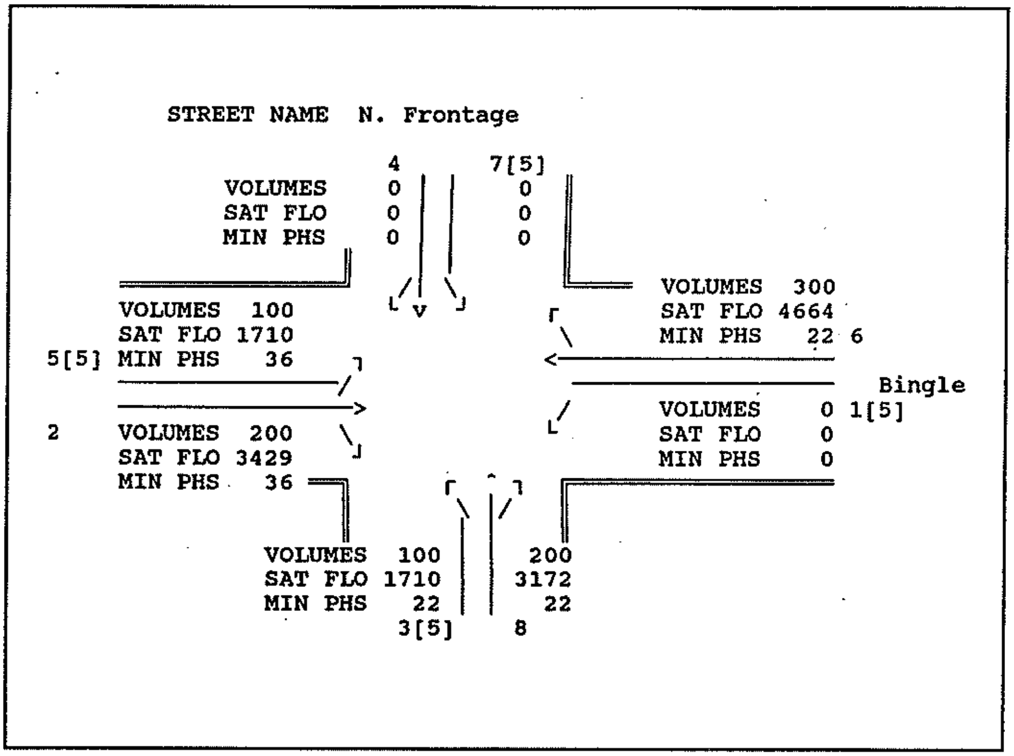
To elaborate on this procedure further it should be noted that queue length measurements and vehicle counts did not

```
[F2]                                                              <ESC>
                     PASSER II-90 Arterial Data

 Run Number                 :  1      City Name     : Houston
 Number of Intersections    :  2      Arterial Name : Bingle
 District Number            :         Date          : 08/07/91

 Lower Cycle Length :   80 | T/S Scales | Movement #2 "A" Direction  : 3
 Upper Cycle Length :   80 |            |
                           |  X :   30  | 1 = North   3 = East   0 = None
 Cycle Increment    :    0 |  Y : 1000  | 2 = South   4 = West

        Output Level :  0               |    Simulated Operation

 0 = Output All Pages
 1 = Error Exit - Cover & Error Pages
 2 = Less Input Data Echo
 3 = Less Input Echo and Best Soln
 4 = Simple - Cover, Pin.Set, T/S
 5 = Debug - All Pages, Variables

 Best Solution Format | 0 = PASSER II
     (0 or 1) : 0     | 1 = AAP P2
```

FIGURE 4. PASSER II ARTERIAL DATA INPUT.

```
[F2]                                                                 <ESC>
Bingle                      Arterial Link Geometry           2 Intersections

 "A"    Queue    Speed    Distance  |  Distance    Speed    Queue     "B"
 Link   Clear.   (MPH)      (FT)    |    (FT)      (MPH)    Clear.    Link
 1- 2     0        30       200     |    200         30       0       2- 1

        From : S. Frontage                      To : N. Frontage
```

FIGURE 5. PASSER II ARTERIAL LINK GEOMETRY DATA.



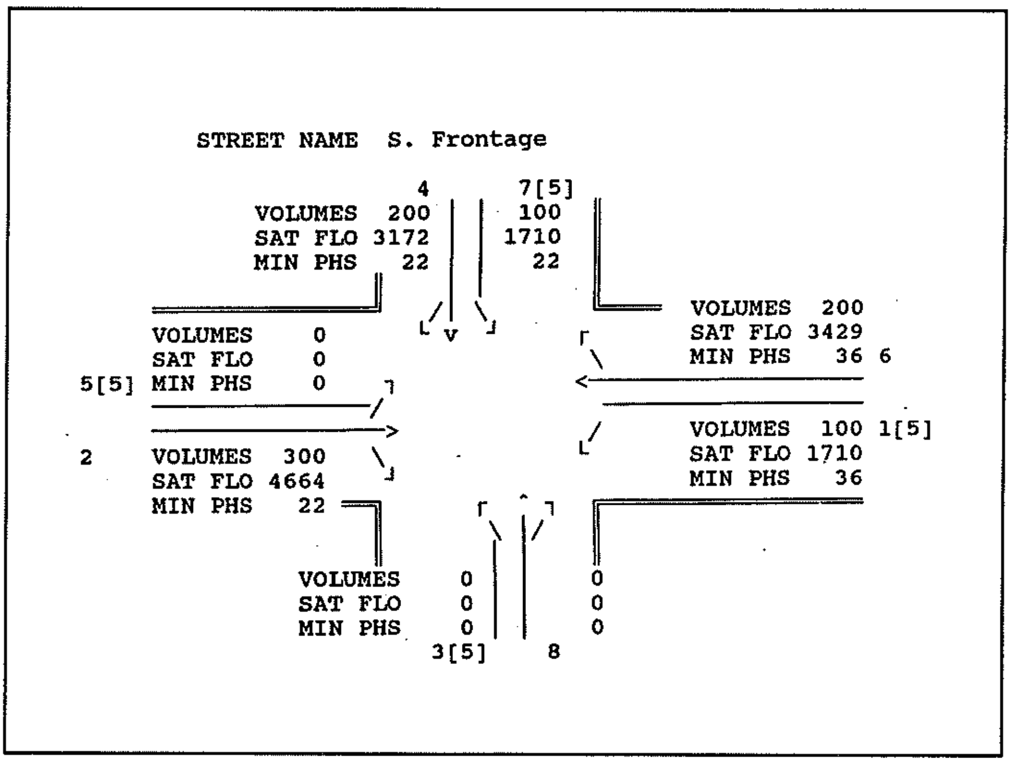


FIGURE 6. PASSER II INTERSECTION MOVEMENT DATA.

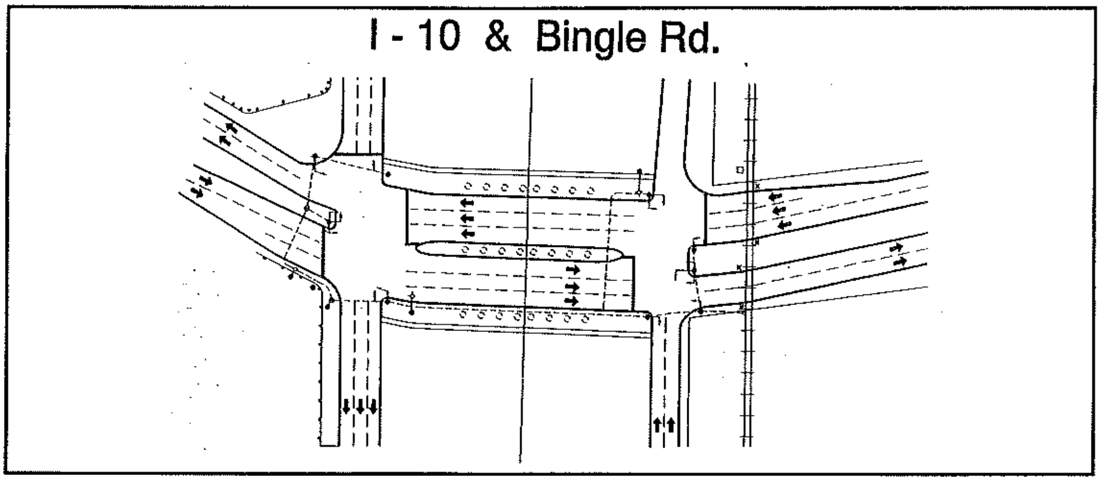


FIGURE 7. SUBJECT DIAMOND INTERCHANGE AT INTERSTATE 10 AND BINGLE ROAD.

include the entire queue for each cycle. That is, the number of vehicles used in the vehicle spacing formula was not the total number of vehicles in the queue each cycle, similarly, the queue length is not the total length of the queues that existed each cycle. In fact, for several observed queues, the number of vehicles counted and the measured distances were half the length of the queue that actually existed. Another point of consideration was the type of vehicle population. Due to the geometrics of the intersections and the demographics of the surrounding area there was little use of the interchange by trucks and heavy vehicles. As a result, the analysis involved and the resulting vehicle spacing value is in general representative of passenger cars.

Part 3 - Applications to Design Criteria

After developing an average "constant" value for vehicle spacing from all of the observed queues, the next step was to combine this value with the results from Part 1. Using computer simulation software in conjunction with actual observed data and associating the two combines the best of both worlds. The objective here was to generate queue length estimations using PASSER II output for maximum queue size and the vehicle spacing constant. This calculation was a simple product of the maximum queue size and the spacing constant.

The Texas DOT's "Highway Design Division Operations and Procedures Manual" has a graph showing three levels of design standards (4) . Once the queue length approximations were computed, the results were plotted on this graph. One data value existed for each level of service. This was done in an attempt to show the exponential pattern of growth for queue length. Demonstration of this pattern would suggest the need for an investigation into the methods of developing design standards. This stage was the practical assimilation of all the analysis involved in this research project.

SUMMARY OF RESULTS

The following synopsis is a brief outline and summary of the results that have been accomplished in this research and the fulfillment of the objectives. Most of the conclusions which are presented are visually supported in the form of graphs and tables. For this reason, this section addresses the objectives and refers the reader to the visuals for further interpretation.

The process of computer simulation and the pursuit of the first objective was successful. It was demonstrated that PASSER II-90 can successfully simulate an isolated diamond interchange. Tables 1 & 2 show the frontage road delay output from the two programs for relevant input. In Figure 8, it is possible to see the pattern that develops in the production of delay output in these models. The calculation of delay within both models is in fact a higher order function. This demonstrates the reasonable conclusion that for future diamond interchange analysis, PASSER II can be used as an additional tool, until such time that external queue analysis is included in future versions of PASSER III.

The investigation of vehicle spacing yielded interesting results both in terms of conclusions and analytical procedures. As initially desired, an average value for vehicle spacing was established, 23.5 feet/vehicle. The calculation of this value was not made in an attempt to provide a standard value, but rather in hopes that a constant would simply provide some way of translating queue size in vehicles into a measurement of length. For all intensive purposes, it can be assumed that the PASSER III user's guide suggested value of 25 feet for vehicle spacing is an appropriate estimate when considering frontage road queuing.

Finally, the applications of the aforementioned results and conclusions yield interesting insight into possible applications of computer modelling for review of design criteria. Figure 10 shows the queue size output generated by PASSER

TABLE 1. COMPARISON OF DELAY OUTPUT FOR DIFFERING LANE VOLUMES.

Computer Simulation Delay Output (5)		
Volume/Lane	PASSER II	PASSER III
100	23.90	23.88
200	25.90	25.94
300	30.40	30.38
400	46.40	46.45
500	123.30	125.27

TABLE 2. COMPARISON OF DELAY OUTPUT FOR EACH LOS.

Computer Simulation Delay Output (6)		
LOS	PASSER II	PASSER III
A	25.65	25.68
B	31.50	32.32
C	34.00	35.27
D	37.90	39.78
E	59.50	58.81
F	106.45	106.46

II-90. The result of this data, when used in conjunction with the vehicle spacing "constant" is shown in Figure 11. This data is then expanded into the results shown in Figure 12. When these results are then plotted on Figure 13, it becomes apparent that the impacts of high volume conditions are substantial in terms of design requirements. The most noteworthy result was the presence of a value for queue length that exceeded the minimum design standard. This demonstrates that there is the possibility that interchanges which experience oversaturated conditions could pose safety problems due to lack of design standards. The results from all these observations show that both field data analysis and investigative modelling, when combined, can demonstrate the need for a better understanding of the relationships between existing conditions and design conditions.

RECOMMENDATIONS

There were several underlying aspects of this field of research which warrant mention but which were not crucial to the analysis procedure of this project. As the computer simulation process was a large portion of this research, much time was spent to ensure its accuracy. When equal movement volumes on each approach were used, great success was achieved in the modelling. For this project, attempts were made to use more realistic interchange input, and when this was done, the results were considered successful, however, they were not as ideal. Great care must be taken to ensure that the PASSER II movement input is accurately modified to suit the model. For the vehicle spacing analysis, the data collection procedure came into question because it produced results that were "too good." A characteristic was noted that when volumes increased, vehicle spacing tended to decrease, a compression factor. Essentially, the point was made that perhaps the design of the data collection process was in fact what caused the compression factor to surface as opposed to the true existence of the factor. There is a need for more study into this topic, however the problems need to be more clearly researched and defined.

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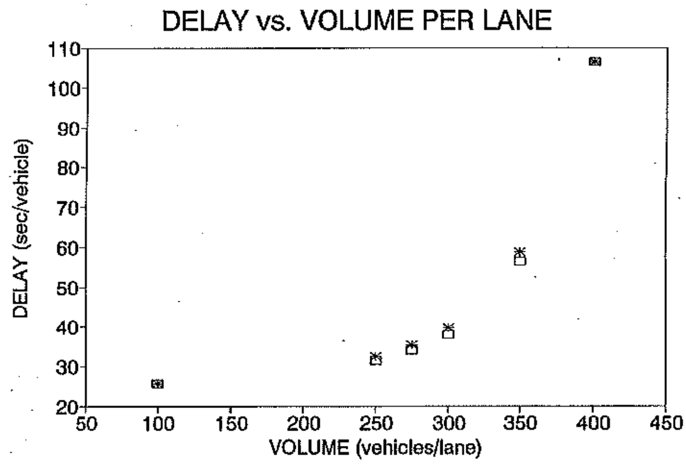

FIGURE 8. DELAY OUTPUT AS A FUNCTION OF LANE VOLUMES.

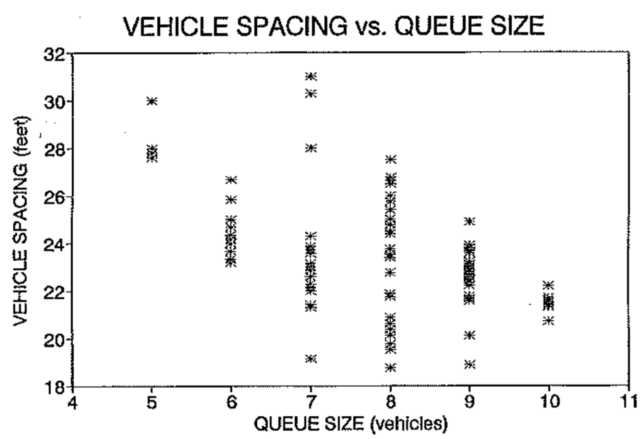

FIGURE 9. AVERAGE VEHICLE SPACING IN RESPECT TO QUEUE SIZE OBSERVATIONS.

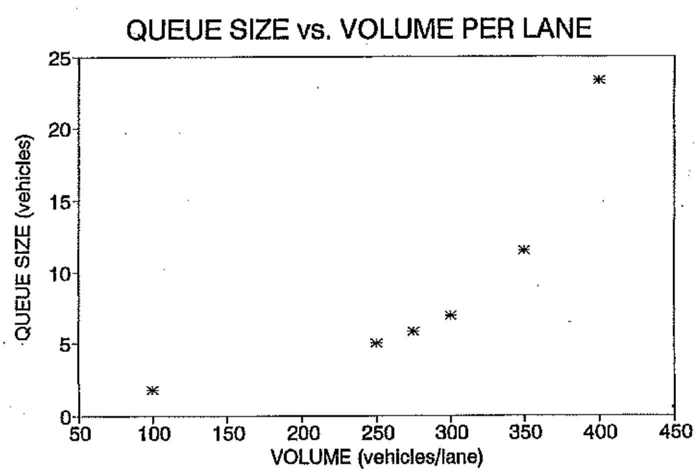

FIGURE 10. MAXIMUM QUEUE SIZE OUTPUT FROM PASSER II.

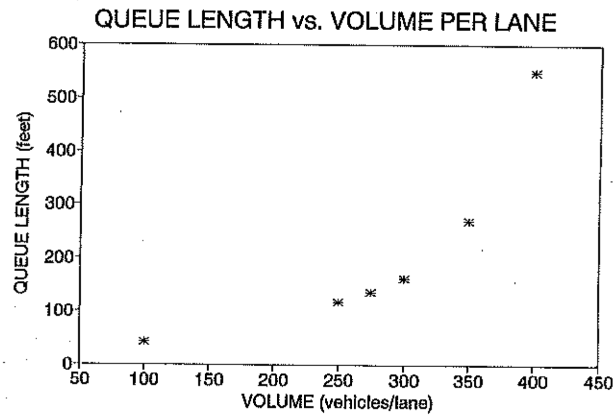

FIGURE 11. QUEUE LENGTH ESTIMATION USING RESULTS FROM PART 1&2 .

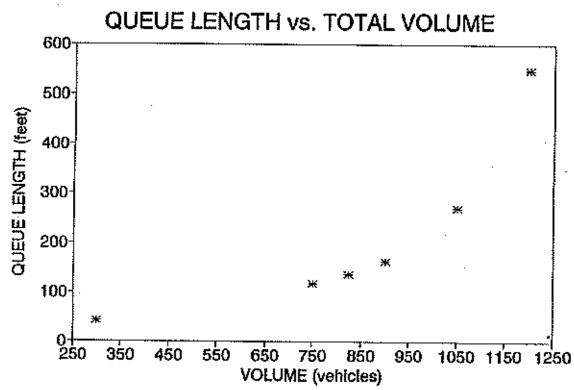

FIGURE 12. QUEUE LENGTH ESTIMATION FOR TOTAL FRONTAGE ROAD VOLUMES.

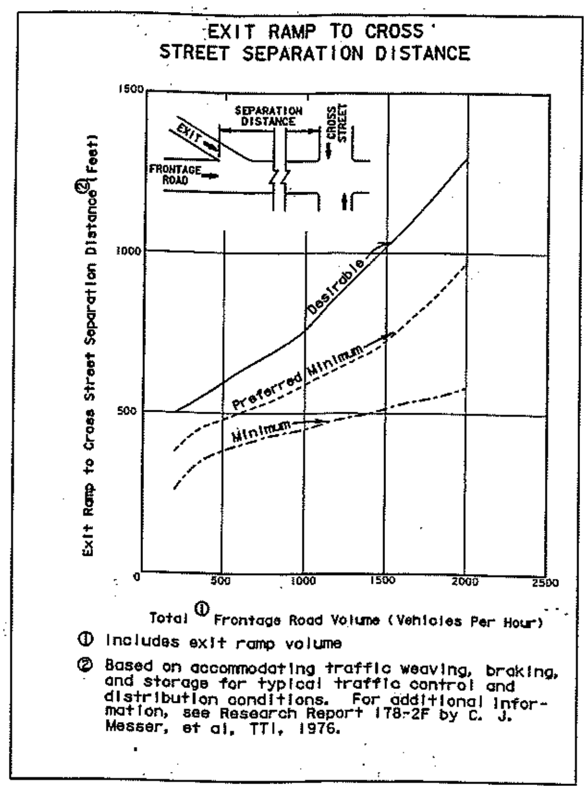

FIGURE 13. TEXAS DOT DESIGN CRITERIA FOR EXIT RAMP SEPARATION DISTANCE.

Relating Speed and Geometric Inconsistencies on Two-Lane Rural Highways

KENNETH L. FINK

This research paper evaluates the relationship between Dr. Carroll J. Messer's workload procedure for quantifying geometric inconsistencies and the change in operating speeds on horizontal curves. Sixteen sites were examined. The sites were chosen from two Farm-to-Market roads in the State of Texas. The data collected at each site was the change in operating speed associated with the feature. Regression equations were formulated to test the relationship between Dr. Messer's workload values and the change in operating speeds for the sites. Also evaluated was the correlation between Dr. Messer's workload values and the mean change in operating speeds for each site. Finally, relationships were established between both the degree of curvature versus the change in operating speed and the degree of curvature versus the mean change in operating speed. The results showed that all the relationships tested were statistically significant. Considerable variability was observed among the drivers at each site. As a result, the r^2 values were relatively low (0.23 - 0.56). Additional data should be collected on a larger number of vehicles at a larger number of sites to account for the observed variability and thereby develop more reliable regression equations.

INTRODUCTION

Almost everyone, from the best transportation engineer to the beginning driver, realizes that accidents are more likely to occur on curves than along tangents on a roadway. However, the reasons for this are not clear. There is, of course, some intrinsic logic that tells us that the more severe a horizontal curve is the greater the chance of driver error. But, what makes one curve more severe than the other, and how do we quantify this severity? The most obvious, and common, factor used to describe and determine the severity of a horizontal curve is its degree of curvature. This is an excellent indicator and has been used in speed-based-procedures for analyzing horizontal curves by Leisch and Lamm (1, 2). However, the degree of curvature is not the only factor that contributes to the severity of a curve. Other factors such as sight distance, driver expectancy, driver unfamiliarity, and preceding elements also can affect a curve's severity. Therefore, the accident potential at curves cannot be quantified based solely on their degree of curvature. An alternative method must be used that rates the seriousness of a curve not only by its degree of curvature but also by other variables that the road presents to the driver, as well as variables the driver brings to the road.

Curves that create problems for the driver by being inconsistent with either the roadway or what the driver would expect of the roadway are labeled as geometric inconsistencies. "A geometric inconsistency in rural highway design is defined as a geometric feature or combination of adjacent features that has such unexpectedly high driving task workload that undesirable driving is a distinct possibility by those surprised motorists" (3). For example, problems often arise when a driver is encouraged by a sufficiently long or inviting section of more gentle alignment to increase speed momentarily, though vehicle speed may need to be reduced further along the road (1). Geometric inconsistencies increase a driver's workload and thereby increase the likelihood of an accident occurring. "Driver workload is the time rate at which drivers must perform a given amount of work or driving task" (3). Therefore, if driver workload can be quantified, the severity of geometric inconsistencies, including horizontal curves, also can be measured.

Workload may be estimated via the Messer procedure (3, 4, 5). The Messer procedure incorporates driver expectancy, driver unfamiliarity, sight distance, and separation distance with other elements, such as degree of curvature, to assign workload values to geometric inconsistencies. Messer contends that the higher the workload value, the more severe the feature.

PROBLEM STATEMENT

A concern about Messer's procedure is that workload can not be measured directly. Therefore, to better measure the severity of horizontal curves and to verify Messer's procedure it becomes necessary to relate Messer's workload values with a more directly measurable value. This relationship also will help to determine if Messer's procedure is preferable for use in quantifying geometric inconsistencies over operating-speed-based procedure such as proposed by Leisch and Lamm (1, 2). The relationship that will be explored in this paper is that between Messer's workload values and the change in vehicle operating speeds. Although there are many types of geometric inconsistencies, due to limited time and resources for data collection, the scope of this study will be limited to horizontal curves, which are among the most common geometric inconsistencies and the most likely to demonstrate changes in operating speeds.

OBJECTIVES

The overall objective of this research was to determine the correlation between workload and changes in operating speed, as well as to test the relationship between degree of curvature and changes in operating speed. These relationships will enable us to determine the validity of Messer's workload values and form other inferences about workload and how it is measured.

BACKGROUND

Messer introduced his method of quantifying geometric inconsistencies, via use of workload values, in a report submitted to the Federal Highway Administration in 1979 (3, 4, 5). Messer's equation for measuring workload values is as follows:

$$WL_n = U * E * S * R_f + C * WL_{n-1}$$

Where WL_n is the workload value that is being obtained for the feature. R_f is the workload potential rating, and the base value for determining workload. The workload potential ratings were determined by a group of 21 experienced highway design engineers and research engineers having experience in highway design, traffic engineering, and human factors (3). The group was asked to rate a variety of geometric features on a 7-point rating scale. A rating of 0 was given to a feature they felt would pose no problem to the driver, and a rating of 6 was given to features that they would term as critical. Factors such as width of shoulders, degree of curve, deflection angle, and many other variables are used to determine a specific Rf for the feature that is being analyzed.

The U value is the unfamiliarity factor. Values for U range from 0.4 to 1.0 depending on the percentage of drivers unfamiliar with the roadway, and classified by the type of roadway. The U value increases as the percentage of unfamiliar drivers increases. For example, a rural principal arterial would receive an unfamiliarity factor of 1.0, a rural local road would receive a 0.4. For this study all roads were Farm-to-Market roads and classified as rural collector roads and assigned an unfamiliarity factor of 0.6.

The E variable is defined as the feature expectation factor. This factor adjusts for the potential confirmation of driver expectancy where the prior feature is similar to the current (3). If the feature is similar to the previous feature, E = 1 - C. If the feature is not similar to the prior feature, E = 1.

The S variable describes the effect of sight distance and is termed the sight distance factor. The sight distance factor is determined by use of a graph that incorporates sight distance and 85th-percentile speed. As the sight distance to the feature decreases, the S value increases and, therefore, adjusts the workload value upward.

The C variable is known as the carryover factor. The purpose of the carryover factor is to determine the percentage of workload that has been carried over from the preceding feature. In other words, it is a measure of how much the driver has recovered from the workload of the previous feature. Carryover factors are determined from a graph, much like the sight distance factor, which incorporates the 85th-percentile speed and the separation distance between the features. Values for the carryover factor can range from one to zero. As the separation distance between features increases the carryover factor decreases thereby lowering the effect of the preceding feature on the workload for the current feature.

Finally, WL_{n-1} is the workload of the preceding feature. This value is combined with the carryover factor to determine the amount of workload that is carried over from the prior feature and contributes to the total workload for the feature.

Messer went on to define levels of consistency, or LOC's, for his workload values. These Levels of Consistency are very similar to the Levels of Service used commonly in highway capacity analysis. LOC's range from the letter A to the letter F with F being the most severe and representing a workload greater than six.

The first attempt at correlating these workload values with other data was recently done by Glascock in his masters thesis at Texas A&M University (6). In his thesis Glascock explored a relationship between Messer's workload values and accident experience on two-lane rural highways. He evaluated the accident history for five roads, and then calculated the workload values for all the features along each road. He then formulated regression equations with his data. By doing this he was able to determine a direct correlation between workload and accident rates. Glascock developed models that produced r^2 values from 0.61 to 0.88; which means that 61 to 88 percent of the variability in accident rates among highways was accounted for by the mean workload value. This was a very impressive finding as previous studies have only been able to explain up to 50 percent of accident variability.

Jack E. Leisch and Joel P. Leisch proposed an operating-speed-based procedure in an attempt to identify problems in roadway alignment (1). The objective of their procedure was to "meet driver expectations and to comply with his or her inherent characteristics to achieve operational consistency and to improve driving comfort and safety" (1). They proposed a 10 mile-per-hour rule that consists of three considerations. First, reductions in design speed should be avoided if possible, but if required should be no more than 10 mph (1). Second, within a given design speed, potential automobile speeds along the highway normally should vary no more than 10 mph (1). Finally, potential truck speeds generally should be no more than 10 mph lower than automobile speeds on common lanes. To avoid these situations, Leisch proposes a speed-profile technique. The technique entails determining speeds on an existing highway at close intervals along the road and plotting these speeds against the distance along the highway. This enables the engineer to see variations in speeds and where they occur along the roadway.

Lamm presented a procedure for measuring the consistency of horizontal design as defined by operating speed and accidents expected (2). Lamm contends that operating speeds and accident rates can be predicted for various lane widths based on degree of curvature and posted recommended speeds. Lamm defines a poor design as having a change in degree of curve between successive design elements that exceed 10° and/or a change in 85th-percentile speed of greater than 12 mph. He goes on to define a fair design, or a design having at least minor inconsistencies, as having a change in speed between 5° and 10° and/or a change in 85th-percentile speed between 6 and 12 mph. Lamm describes a good design as one having a change in degree of curvature of less than or equal to 5° and/or a change in operating speed of less than or equal to 6 mph. Lamm feels these criteria should be considered when evaluating designs and especially in the resurfacing, restoration, and rehabilitation of existing roadways.

Glennon believed that to create a good design one must conceptualize a relationship between driver performance and highway system demand in creation of accident circumstances (7). Glennon demonstrated this by plotting driver performance and system demand versus time. He proposed that as time went on and the two curves converged the opportunity for accident increased. Glennon stated "in developing design criteria that are functionally related to the design constraints, the real solution is one of matching the limited sensory and motor capabilities of the driver to the requirements of the driver task for various combinations of vehicle, roadway, traffic, and environmental constraints" (7). Glennon's approach for evaluating design features for design consistency consists of six countermeasures:

1. Improve driver detection.
2. Increase driver perception and response time.
3. Eliminate "false cue" designs.
4. Decrease driver guidance and control demands.
5. Increase driver expectancy.
6. Build "relief valve" designs.

Another research report that was used for reference was Alexander's 1986 report to the Federal Highway Administration (8). Alexander's report is based on the idea of driver expectancy and how violations thereof affect the driver. According to the author, "expectancy relates to a driver's readiness to respond to situations, events and information in predictable and successful ways" (8). Alexander asserts that when the road meets the driver's expectancy, performance tends to be error free. This was an important study because it dealt with more than what the roadway presents, but with what the driver expects the roadway to present and how violation of this expectancy can increase the chance for driver error.

STUDY DESIGN

Sixteen features with varying workload values and degrees of curvature were chosen for data collection and analysis. The curves were selected from two roads that were studied by Glascock: FM 391 and FM 1179 (6). Four curves were selected from FM 391 and the remaining twelve from FM 1179. The rational for the larger number of sites on FM 1179 was that although the AADT on FM 391 was 995 vehicles, attempts at data collection at sites along FM 391 failed to provide enough vehicles for continued collection. The four curves studied on FM 391 were included in the data, however, although the number of vehicles was much lower than the studies on FM 1179. The other three roads in Glascock's thesis (FM 2154, FM 1362 and FM 3058) were not used due to either low AADT's or lack of significant curves. The workload values for the curves had been previously calculated by Glascock. However, these values were verified and in some case slightly revised for consistency sake. Differing workload values and degrees of curvature were selected to formulate a good distribution of values for the regression analysis. The sites and their characteristics are summarized in Table 1.

Speeds were taken at two locations at each feature. First, each vehicle's speed was taken approaching the curve. All the curves selected were preceded by a relatively long tangent so speeds could be taken at the midpoint of the tangent, before the vehicle began to decelerate for the curve. Each vehicle's speed was then taken at the midpoint of the curve. The operating speed of the vehicle in the curve was then subtracted from its speed in the tangent to determine its change in operating speed. Data collection personnel used radar guns to measure the vehicles speed in both the tangent and the curve. At each site data collection personnel attempted to conceal themselves as best as possible from the motorists, so their presence would not affect the vehicle's operating speeds.

TABLE 1. SUMMARY OF SITES USED FOR DATA COLLECTION AND ANALYSIS.

ROAD	STATION	DEGREE OF CURVE	WORKLOAD
391 Eastbound	210 + 79.3	18° 00'	9.09
391 Westbound	280 + 70.2	5°00'	14.50
391 Eastbound	602 + 54.2	10°00'	2.53
391 Westbound	675 + 62.5	10° 00'	2.53
1179 Northbound	66 + 32.0	6°00'	4.94
1179 Northbound	120 + 13.0	12°00'	9.54
1179 Southbound	126 + 00.0	12° 00'	8.07
1179 Northbound	161 + 27.3	5°00'	4.69
1179 Southbound	161 + 27.3	5°00'	4.65
1179 Northbound	172 + 30.2	4°00'	6.05
1179 Southbound	172 + 30.2	4° 00'	0.92
1179 Northbound	194 + 61.0	1°00'	1.32
1179 Northbound	232 + 70.5	3°00'	0.64
1179 Southbound	243 + 38.3	3° 00'	2.33
1179 Northbound	265 + 05.5	6°00'	1.44
1179 Southbound	265 + 05.5	6°00'	2.33

RESULTS

The field data was then brought back into the office where it was analyzed with SAS, a statistical software package. After entering all the data into SAS, the software was used to perform various statistical procedures. The procedure of greatest interest for this research was the regression analysis performed between workload and change in operating speed. Also of interest was the analysis between workload and the mean change in operating speed at each site. The relationship between degree of curvature and change in operating speed was also explored and compared with the relationship between workload and change in operating speed. Finally, the relationship between degree of curvature and the mean change in operating speed was studied and compared to the previous analyses. The results from these regression analyses are shown in Table 2.

The first regression equation that was formed was to test the correlation between workload and change in operating speed. The results of this model are shown in Table 2 and the scatter plot is displayed in Figure 1. Workload was assigned as the independent variable and change in operating speed as the dependent variable. Unfortunately the r^2 that was computed came out to be lower than expected at 0.23. Although this is a rather low value, the F- and p-values for the test were 77.84 and 0.0001 respectively. The high F-value means there is a correlation between the variables, and the p value tells us that the test is significant. The low r^2 value indicates that a large percentage of the variability in the data remains unexplained.

There are several reasons the r^2 value would be so low. The first reason might be that the variables do not have a strong enough correlation to form a substantial relationship through regression. Another reason is the large inherent variability among drivers within a site and the lack of data from enough sites to override the effect of that variability. This is believed to be the case for this study. Since there were only sixteen sites studied with an average of just seventeen vehicles per site, the chance for variability within the data was very large. If one or two sites were inconsistent with the rest of the sites, the r^2 value could have been adversely affected. This is thought to be the reason for the low r^2 value calculated in this study.

Out of the sixteen sites, two had a standard deviation in their changes in operating speeds of greater than five miles per hour and three had standard deviations greater than seven miles per hour. This certainly demonstrates a large amount of the variability in the data. The reasons for this variability are varied. It might have been the time of day that the study was done. For instance, if the study was done at peak hours the drivers would have been primarily commuters, who would be more familiar with the road thus affecting the data; the opposite circumstance would of course happen during off-peak hours. The drivers may have been affected by something following the curve, such as another curve or an intersection, and slowed down a greater amount than they normally would have under usual circumstances. Finally, the most probable cause would be that the motorists saw the data collection personnel. Although they hid themselves as best as possible, some sites did not provide good "hiding places," and detection by the motorists could have been possible. This would have caused the motorist to slow down either out of curiosity, to see why a man was attempting to hide in the bushes along the road, or because they mistook the data collection personnel for police officers and were more cautious with their speed.

TABLE 2. REGRESSION EQUATIONS AND CORRESPONDING RESULTS.

Model	F-Value	P-Value	R-Square	Root MSE
CHGSPD = 0.4 + 1.1 * WL	77.82	0.0001	0.2270	6.7925
MCHGSPD = 1.4 + 0.9 * WL	7.53	0.0158	0.3499	5.0760
CHGSPD = -3.8 + 1.5 * DOC	220.96	0.0001	0.4546	5.7051
MCHGSPD = -1.4 + 1.0 * DOC	8.00	0.0008	0.5624	5.7969

CHGSPD = Change in operating speed
WL = Workload
DOC = Degree of Curve
MCHGSPD = Mean change in operating speed

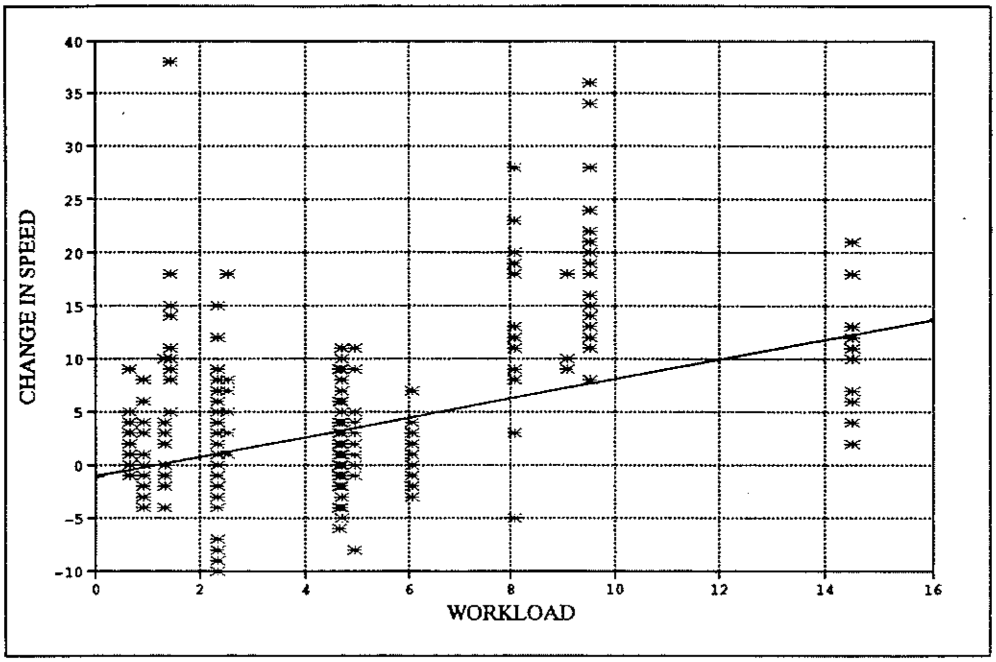

FIGURE 1. SCATTER PLOT OF THE CHANGE IN SPEED VS. WORKLOAD.

In an attempt to somehow compensate for the variability within the data, the mean change in speed at each site was calculated and these values then regressed against the workload values for the sites. The results from this analysis are shown in Table 2 and the scatter plot is displayed in Figure 2. This analysis showed a significant increase in the r^2 value to 0.35. Although this value is still rather low, it is promising in that it demonstrates that some of the variability is among the drivers within the sites. If the r^2 value for the means of the sites were equal to or less than the r^2 value for the entire data, it would show that the problem was probably within the relationship between the variables. However, since the r^2 value increased, it demonstrates some correlation within the data, although there is still some variability.

Another relationship that was studied was between the change in operating speed and the degree of curvature for the curve. The output from this analysis is shown in Table 2 and plotted in Figure 3. This relationship appeared to be stronger (r^2 value was 0.45). The F- and p-values were 220.96 and 0.0001 respectively. It is intuitive that as the degree of curve increases, the change in operating speed increases, and therefore less data is needed to confirm this relationship. Also, when workload was regressed against change in operating speed, there were more variables involved. Although it only appeared to be two, workload and change in operating speed, the workload values are actually calculated from several variables, including degree of curvature. Since degree of curvature is a directly measured quantity, its inherent variability is less, thus creating less variability in the relationship with the change in operating speed.

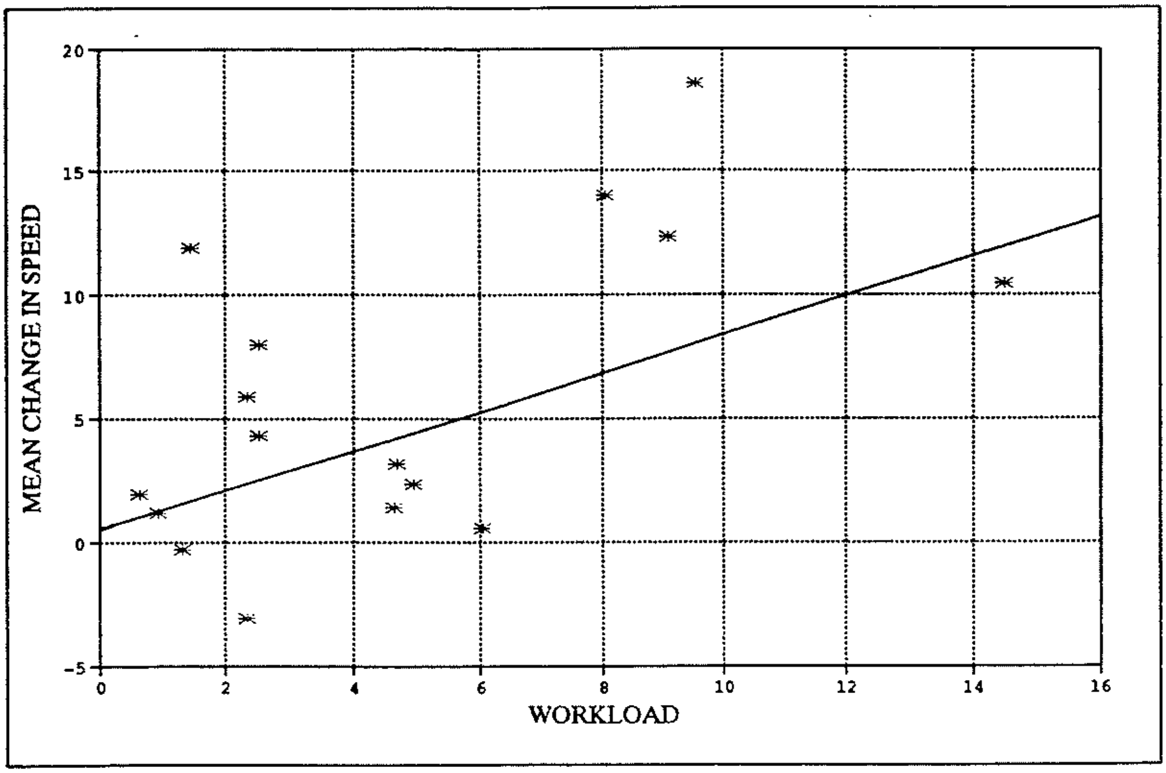

FIGURE 2. SCATTER PLOT OF THE MEAN CHANGE IN SPEED VS. WORKLOAD.

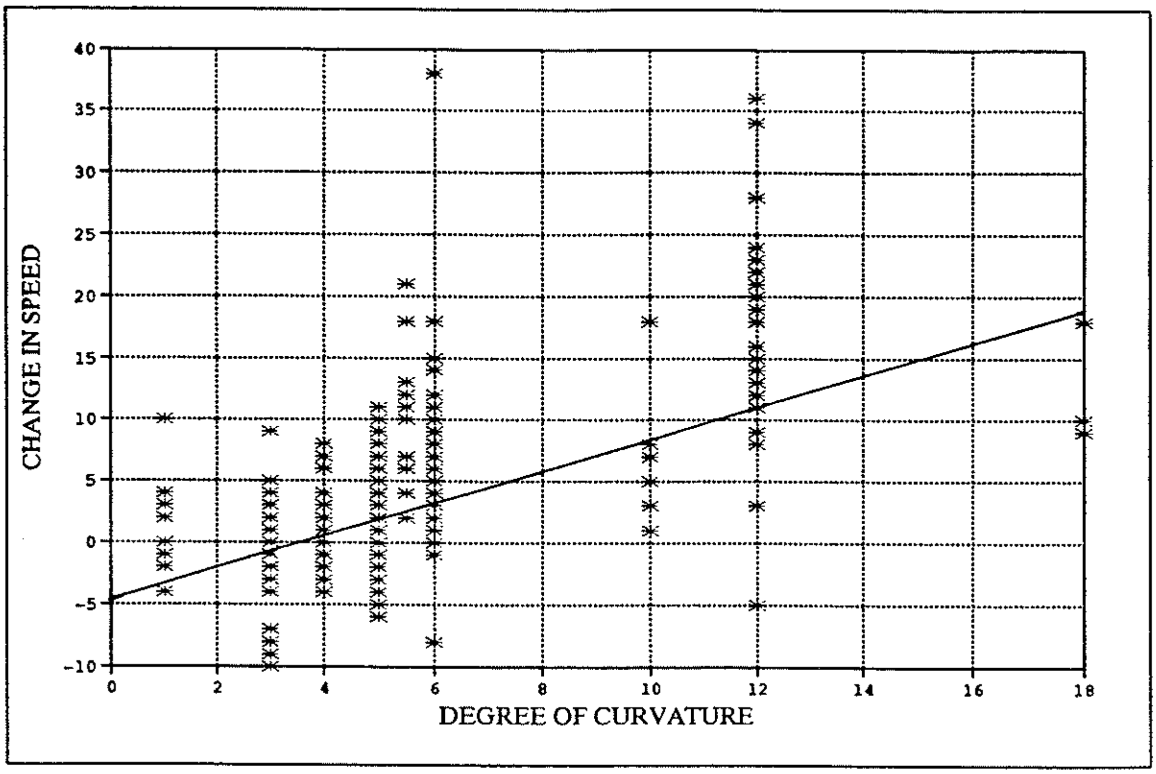

FIGURE 3. SCATTER PLOT FOR THE CHANGE IN SPEED VS. DEGREE OF CURVATURE.

The mean change in operating speed was then regressed against the degree of curvature to determine if it would lessen some of the variability as it did previously with workload. Again the r^2 value increased, this time to 0.56. These results are summarized in Table 2 and Figure 4. This also helps to prove the previous claim, that there was considerable variability among drivers at individuals sites.

CONCLUSION

These results show there is a correlation between both workload versus change in operating speed and degree of curvature versus change in operating speed. Although the r^2 for the relationship between workload and change in operating speed was low, the results are still promising. The F-value was high and the p-value very low. Also the correlation became stronger when the mean change in operating speed was used at each site. The use of the mean change in operating speed allowed for the reduction of variability thereby demonstrating that the collected data were not as consistent as it could have been, or, in other words, the low r^2 value was probably not due to a lack of significant relationship between workload and change in operating speed.

The r^2 value for the relationship between degree of curvature and change in operating speed was higher than that of the relationship between workload and degree of curvature. This, however, does not mean that degree of curvature is a better measure of the severity of a curve than workload. As explained before, the degree of curvature is a simpler and more direct measure than workload, and therefore has a lower inherent variability. The r^2 value for the degree of curvature versus the mean change in speed also increased, which helped to substantiate the claim that there is considerable variability in driver behavior at individual sites.

RECOMMENDATIONS

If further study is to be done in this area, the first consideration must be the collection of data. As much data should be collected at as many sites as possible. The data should be taken at each site twice, at different times of the day. Finally, data should be collected from as many different roads as possible.

These recommendations should help to eliminate the excess variability in the data and assist in determining the validity of the relationship between workload and change in operating speed.

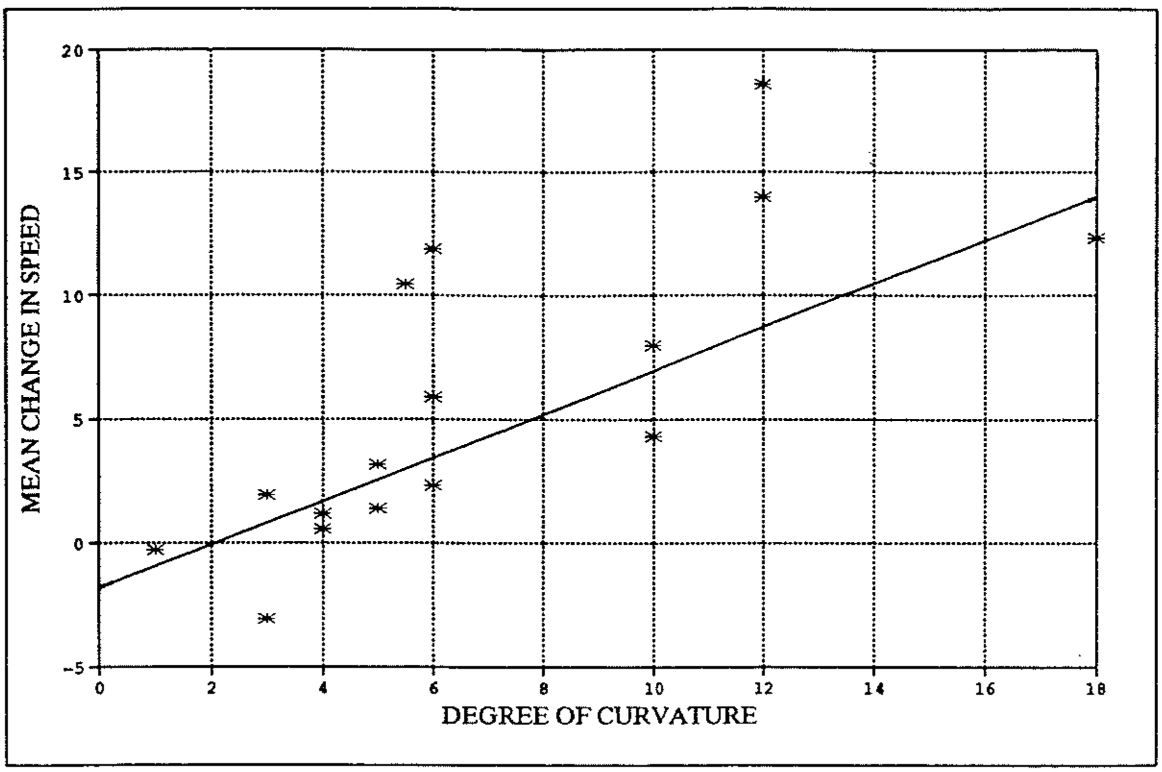

FIGURE 4. SCATTER PLOT FOR MEAN CHANGE IN SPEED VS. DEGREE OF CURVATURE.

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Analysis of Inductance Loop Detector Sensitivities

ROBERT A. HAMM

This report presents the results of a group of tests on the accuracy and sensitivity of inductance loops. The detection ability of deep buried loops to bicycles, motorcycles, MOPEDs, and high profile trucks are discussed. Also included is the effect of water and the effect of a manhole on the detector's sensitivity.

Deep buried loops were found to be unsuitable for detecting bicycles and MOPEDs. However, deep buried loops up to a depth of 15 inches did successfully detect and hold the call for high profile trucks.

Water did create a slight change in the loop inductance, but the Texas series detector was able to adjust around the change. No effect on vehicle detection was found when the loop was flooded with water.

A manhole within the loop area did not detract from the loop's ability to accurately detect vehicles. The loop performed essentially the same as a loop located in an area without a manhole.

INTRODUCTION

Purpose

Inductance loops play a major role in vehicle detection systems that provide data for use in traffic-actuated signal control and other surveillance systems. Induction loops detect the presence or passage of automobiles and trucks. Considering the wide variety of vehicles on the road today, it has become increasingly important to find a loop configuration that will accurately detect odd-sized vehicles. Odd-sized vehicles include motorcycles, mopeds, bicycles, and especially high profile trucks. This loop type should also be easy to install and relatively maintenance free.

Most loops in use today are rectangular surface loops and are not efficient enough to detect these vehicles. A recent study found that deep buried loops are as accurate in detecting automobiles as surface loops (1). However, since damage by pavement movement, traffic flow, and construction equipment does not exist, deep buried loops require less maintenance. It is therefore desirable to determine if deep buried loops are sensitive enough to detect bicycles, MOPEDs, motorcycles and high profile trucks.

Several common practices used today during loop installation use up valuable time and may not even be necessary. One practice requires sealing the loop for protection from water, and another is to avoid placing the loop near a manhole. Many professionals believe that water around a loop wire or the presence of a manhole near a loop has an adverse effect on a loop's accuracy. Therefore, it is desirable to determine what effect, if any, these conditions have on accurate inductance loop operation.

Objectives

The broad objective of this research effort is to establish a better understanding of induction loop detector sensitivity. The specific objectives of this research are:

1. To determine how sensitive deep buried skewed loop detectors are to bicycles, MOPEDs, and motorcycles for two, three, four and five turns of wire.

2. To determine how sensitive deep buried loop detectors are to high profile trucks for two, three, four and five turns of wire.

3. To determine how the presence of water around the loop wire in the conduit affects the detection ability of the loop for two, three, four, and five turns of wire.

4. To determine how the presence of a manhole in the loop area influences the sensitivity of a surface loop.

BACKGROUND

Principles of Detection

An inductance loop is composed of a detector unit, a lead-in cable, and an insulated wire (the loop) wound one or more times (each wind is known as a turn) in the pavement surface. The detector unit energizes the loop system RF circuit at a certain frequency. The loop becomes an inductive element, and, as a vehicle travels over the loop, the inductance of the loop decreases. The detector unit senses this change in peak frequency and interprets this as a vehicle presence if its magnitude exceeds the threshold value set on the detector.

Loop Inductance Measurements

A loop should contain more than 50 microhenries of inductance to operate effectively, and, depending on the size of the loop and the lead-in cable length, the loop requires a certain number of wire turns. The minimum number of turns needed for a certain loop size can be approximated as follows:

$$L = \frac{5PN^2}{N+10} \qquad (1)$$

where:
- L = Loop inductance in microhenries
- P = Perimeter of the loop in feet
- N = Number of turns of wire used

The actual inductance is determined in several ways. It is measured with an inductance meter or determined from the frequency by using the manufacturer's frequency verses inductance plot on the frequency meter. The inductance can also be calculated from the following relationship:

$$f = \frac{1}{2\pi\sqrt{LC}} \qquad (2)$$

where:
- f = frequency of the loop in hertz
- L = loop inductance in henries
- C = capacitance of the loop in picofarads

A loop's sensitivity can be adjusted. The detector unit has high, medium, and low settings which adjust a loop's sensitivity. The number of turns of wire also affects the sensitivity. A greater number of wire turns results in a greater detector sensitivity.

Loop Configurations in Bicycle Detection

Although many loop designs exist, the most common surface loop design used in the field is the 6' x 6' square. This design effectively detects automobiles and is easy to install. A recent study has shown, however, that this design does not accurately detect bicycle and motorcycle traffic. The study found that a 6' x 6' loop skewed 45 degrees to the direction of traffic is the best design for accurate detection of bicycles, MOPEDs, and motorcycles (2). Other designs primarily used for bicycle and MOPED detection are shown in Figure 1 (3).

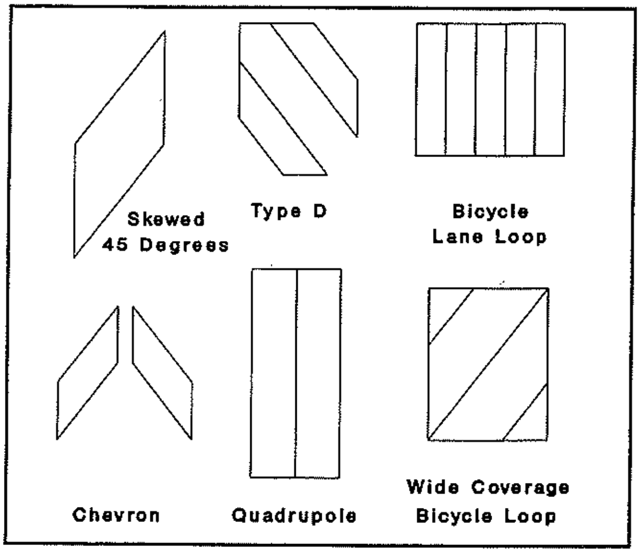

FIGURE 1. TYPICAL BICYCLE LOOP DESIGNS.

Deep Buried Loops and Bicycle Detection

A loop installed below the pavement surface is known as a deep buried loop. A recent study showed that a 6' x 6' square twenty inch deep buried loop detects automobiles as well as a 6' x 6' square surface loop (1). However, neither the surface loop nor the deep buried loop accurately detected bicycles, MOPEDs, or motorcycles possibly because of the square loop shape. For deep buried loops to be effective, small vehicle detection must be achieved at a sensitivity level that will not cause false detections from adjacent lanes. This is especially true if the loop is located in a bike lane.

Deep Buried Loops and High Profile Truck Detection

One type of vehicle which may be hard to detect with a deep buried loop is a high profile truck. High profile trucks sit at or about 51 inches off of the ground. With the loop buried at 20 inches below the surface, the truck bed is 71 inches (5.9 feet) away from the inductance loop. This distance may be too large for the loop inductance to change, so presence detection may not be possible.

For deep buried loops to be considered an acceptable loop alternative, high profile truck detection is necessary. If the detector only detects each axle, it falsely detects the passage of two vehicles. Large vehicles must be detected and allowed to pass through on the green phase. Non-detection of these vehicles can cause jack-knifing, a longer start-up time delaying other traffic, and increased noise and air pollution resulting from high profile truck start-up (3).

Effect of Water on a Loop

Common practice in surface loop installation is to seal the loop for protection from water and for holding the loop in place and away from traffic damage. Many believe that water affects the inductance of a loop, thereby degrading the loop's detection ability. However, a recent study in a laboratory proved that the presence of water around a loop produces only a small change in capacitance and frequency (1). With the current state of modern detectors, this small change in frequency does not affect the ability of the detector to function accurately.

The Federal Highway Administration (FHWA) questioned the study because the study did not consider the effect the soil will have on the water saturated loop. The FHWA believes that water around a deep buried loop will change the inductance enough to trigger a false detection (4). If water does not have any affect on a loop, sealing the loop 100% will not be necessary, thereby saving time and money.

Effect of a Manhole on a Loop

The FHWA recommends that traffic detectors should not operate within 10 feet of a manhole located in the roadway. This distance is required to allow for maintenance work on the manhole without disturbing the detector (3.) No studies, however, have been completed to determine the effect of metal objects, such as manholes, on detector sensitivity. This research effort examines the effect of a manhole on the sensitivity and detection ability of a surface loop.

STUDY DESIGN

Installation of Induction Loops

Two deep buried 6' x 6' square loops exist on University Drive near Avenue A. The loops are 15 inches and 20 inches below the pavement surface. Installation of a deep buried 6' x 6' loop skewed 45 degrees to the direction of traffic occurred in the 1200 block of Neal Pickett Drive in College Station, Texas. Due to the compaction of the subbase course prior to installation, the loop was placed only 10 inches below the surface.

Installation of a temporary 6' X 6' square surface loop occurred on Villa Maria Road between Texas Avenue and Wayside Drive in Bryan, Texas. The loop was placed around a 24 inch diameter manhole located in the far right hand eastbound lane as shown in Figure 2.

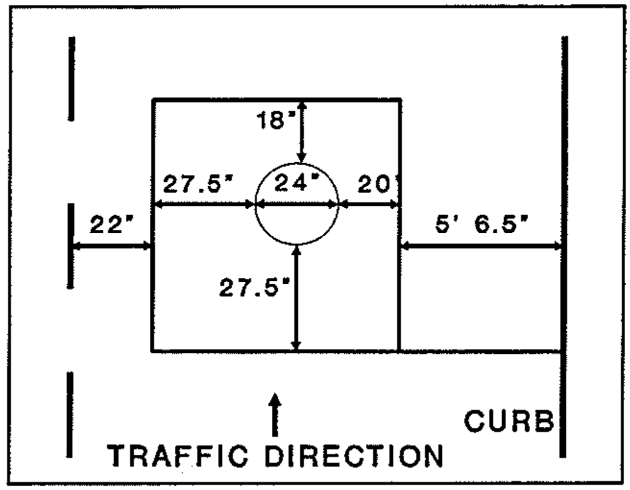

FIGURE 2. SURFACE LOOP MANHOLE LAYOUT.

Small Vehicle Sensitivity Study

Manual observations using a Detector Systems Digital Loop Detector tested a MOPED, motorcycle, and two bicycles passing over the 45 degree deep buried skewed loop. The vehicles traveled on paths across the loop in both directions as shown in Figure 3.

Each vehicle made several passes on each path for each number of turns and sensitivity tested. Testing began with five wire turns and a high sensitivity level and slowly decreased. A medium frequency level remained throughout testing. A previous study has proven that no effect on vehicle detection exists for different frequency settings (1). Due to time constraints, testing stopped when a vehicle made one pass on every path and no detections occurred. The number of turns and/or the sensitivity level then increased.

Observations also included the base frequency of the loop and the frequency as a vehicle passed over the loop. The frequency was used to determine the loop inductance and the percent change in inductance caused by a vehicle presence.

High Profile Truck Sensitivity Study

Manual observations using the Detector Systems Digital Loop Detector observed the high profile truck passing over both the 15" deep and the 20" deep 6' X 6' square loops buried on University Drive. Observations included two, three, four and five turns of wire with low, medium, and high sensitivity settings. A medium frequency level remained throughout testing. The TTI water truck was used as the high profile truck.

While the truck study was performed, the percent change in inductance of different sized vehicles was determined. The base frequency of the loop and the frequency as the vehicles passed over the loop were recorded.

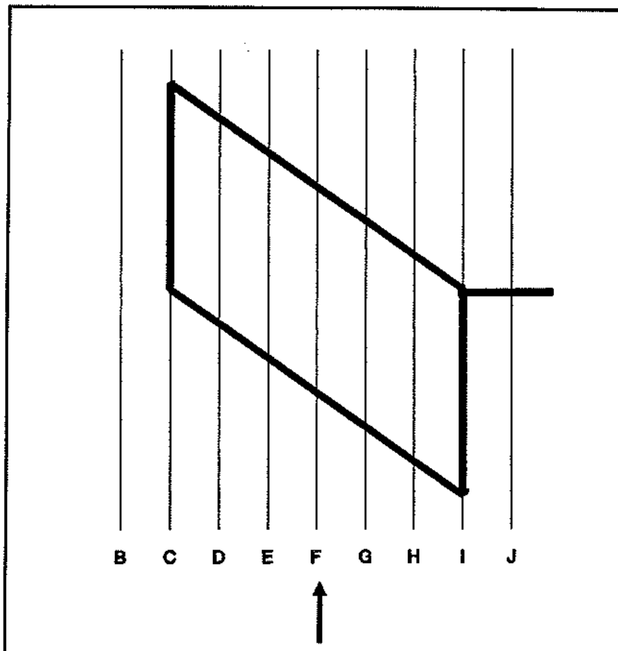

FIGURE 3. SKEWED LOOP TEST PATHS.

Manhole Sensitivity Study

Using the Detector Systems Detector Unit, manual observations determined if the manhole affects a loop's accuracy. Observations began with two turns of wire and a low sensitivity setting. A medium frequency level remained throughout testing. Observations included 50 passes of random vehicles for each number of turn and sensitivity combination tested. The number of false detections from the adjacent lane (spillovers), and number of vehicles travelling in the adjacent lane were also recorded.

Water Sensitivity Study

Since an inductance meter could not be located, the inductance of the loop was directly calculated using the RF circuit frequency. Capacitance and frequency measurements were taken on the University Drive 20" deep buried loop for all turns of wire. About two gallons of regular tap water then filled the loop PVC conduit. Capacitance and frequency measurements were then remeasured for all turns of wire.

An incorrect hookup of the MICRONTA multimeter resulted in invalid capacitance measurements. Remeasurements revealed that the multimeter's scales were too high to register a capacitance. Measurements were then attempted using a CIRCUITMATE Capacitance Meter, but the scales were still not small enough to read the loop capacitance. Therefore, no capacitance measurements appear to be valid.

Loop observations included at least 50 passes of random vehicles each for two, three, four, and five turns of wire. Following each observation, the frequency was measured again. All measurements and observations were then repeated on the 15" deep buried loop.

RESULTS

Bicycles, MOPEDs, and Motorcycles

The 10 inch deep buried loop did not perform as well as expected for these small vehicles. The bicycle and MOPED were generally very hard to detect, with 5 turns and a high sensitivity level the only combination producing 100 percent detection, as shown in Table 1.

One-hundred percent detection includes paths B through J (one foot outside the loop perimeter). It is considered necessary for the bicycle to be detected one foot outside of the loop boundary because that is generally where the bicyclist will ride.

If only paths located within the loop perimeter are considered, the MOPED achieved 100 percent detection on high sensitivity and five turns. However, no MOPED detections occurred on any other number of turns or sensitivity combination.

Much better results were achieved with the motorcycle than the other small vehicles with 100 percent detection occurring on high sensitivity with both four and five turns of wire. If only paths located within the perimeter are considered, the motorcycle also achieved 100 percent detection with five turns and a medium sensitivity and also three turns and high sensitivity.

Figure 4 shows the percent change in inductance the small vehicles achieved. Only the motorcycle had a large enough change in inductance to be detected by both types of detectors on medium sensitivity. The moped and bicycle registered very small percent changes in inductance and are very hard to detect.

High Profile Truck

The 15" deep buried loop performed much better than the 20" deep buried loop in detecting high profile trucks. Low sensitivity did not accurately detect the trucks on either loop or on two, three, four, or five turns of wire. On medium and high sensitivity, however, there was a significant difference between the 15" deep loop and the 20" deep loop, as shown in Table 2. The 15" deep loop performed significantly better in all cases except five turns and high sensitivity.

An accurate detection consisted of the detector holding the signal for the entire length of the truck. Many

TABLE 1. PERCENT ACCURATE DETECTION OF SMALL VEHICLES WITH 10 INCH DEEP BURIED LOOP.

Number of Turns	Sensitivity Setting	Bicycle	Moped	Motorcycle
2	Medium	0.0	0.0	0.0
3	Medium	0.0	0.0	0.0
4	Medium	20.0	0.0	42.4
5	Medium	5.6	0.0	78.4
2	High	0.0	0.0	0.0
3	High	0.0	0.0	70.4
4	High	55.6	0.0	100.0
5	High	100.0	77.8	100.0

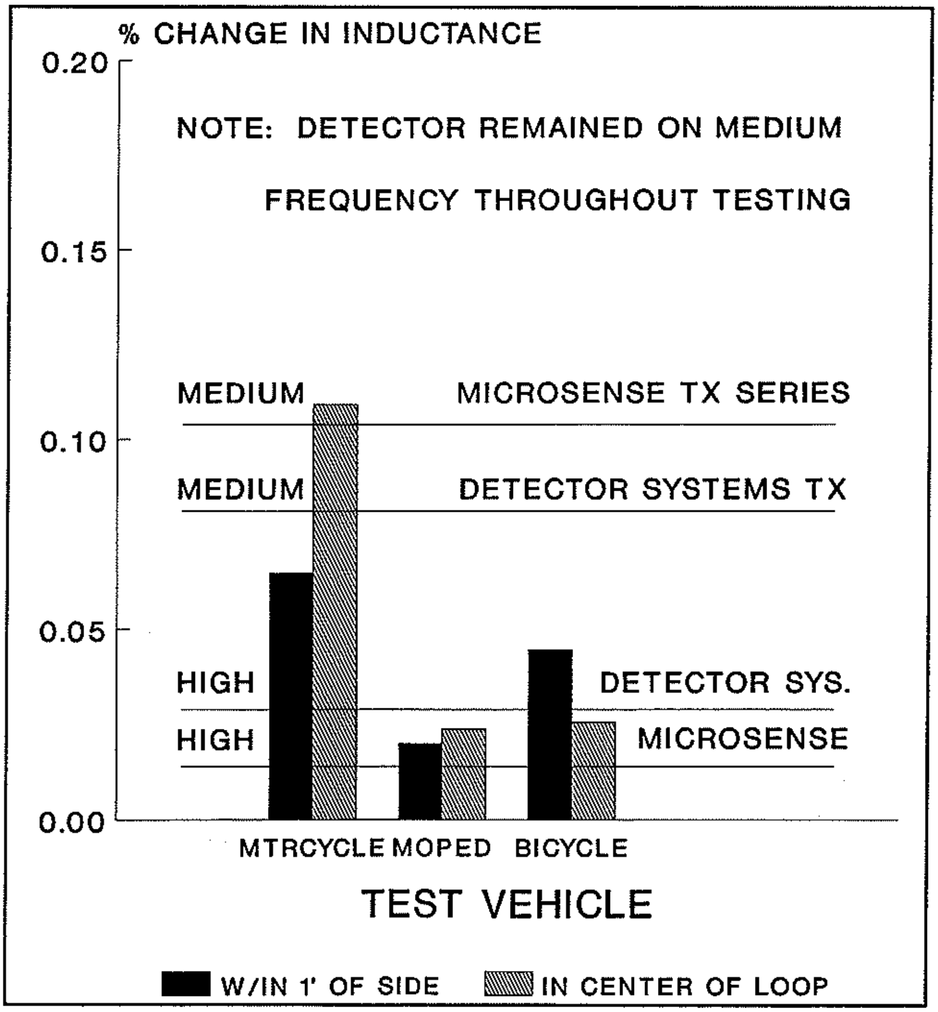

FIGURE 4. PERCENT SHIFT IN INDUCTANCE FOR SMALL VEHICLES.

TABLE 2. PERCENT SIGNAL HELD OF THE HIGH PROFILE TRUCK.

Number of Turns	Sensitivity Setting	15" Deep Loop	20" Deep Loop
2	Medium	0.0	0.0
3	Medium	100.0	33.3
4	Medium	100.0	50.0
5	Medium	100.0	71.4
2	High	100.0	50.0
3	High	83.3	33.3
4	High	100.0	83.3
5	High	100.0	100.0

detections showed the presence of two vehicles instead of one truck. Three wire turns and high sensitivity performed questionably on both loops. In March 1989, during loop installation, construction equipment accidentally cut the third wire of the 15" deep buried loop. An emergency splice on that wire is probably the cause of the questionable results. Due to time constraints, the sample size was smaller than desirable, but the data did present an accurate trend of high profile truck detection.

The frequency measurements taken on many vehicles travelling over the loop indicate a large variation in percent change of inductance between different vehicle sizes and types and within different vehicles, as shown in Figure 5. The water truck was consistently picked up on high sensitivity, sometimes on medium sensitivity, but rarely on low sensitivity. The sample size of frequency was less than desirable for all vehicles except the water truck due to time constraints.

Despite being a large vehicle, a concrete truck caused a lower percent change in inductance than all other vehicles except the Bronco. A large variation in percent inductance change also was noted for the same vehicles. The data do incorporate samples from many of the number of turns from both loops.

Manhole Sensitivity

The manhole had virtually no effect on the detection ability of the loop. Two turns of wire on low sensitivity accurately detected 100 percent of the vehicles. As the number of turns of wire increased, many false detections were recorded from the adjacent lane, as shown in Table 3. With five turns of wire, only 82.3 percent of the detections on high sensitivity were from the right lane, with the other 17.7 percent of the detections being spillovers. This rate of spillovers is common with loops only 22 inches from the lane line and is not caused by the manhole. Regular surface loops less than two feet from the adjacent lane have a similar percent of false detections.

Water Sensitivity

Water had virtually no effect on the detection ability of either the 15 inch or the 20 inch deep buried loops. The inductance did slightly increase, as shown in Figure 6, but the detector unit adjusted and remained very accurate. The only problem in detection accuracy was for two turns of wire and low sensitivity. However, this problem also occurred in the dry condition, so water cannot be considered the cause.

Percent detection accuracy is shown in Table 4 for low sensitivity only. All other sensitivity and number of turn combinations had a 100 percent detection accuracy except medium sensitivity and four turns on the 20" deep loop. In this case, one motorcycle was not detected resulting in a detection accuracy of 97.9 percent.

CONCLUSIONS

Deep Buried Loop Recommendations

Deep buried loops are not recommended to be used for bicycle or moped detection. Motorcycle detection is possible with 10" deep buried loops using four or more turns and a high sensitivity. However, spillover effects for a 10" deep buried loop are not known. Therefore, surface loops or deep buried loops much closer to the surface are recommended for use in bicycle, MOPED, and motorcycle detection.

Deep buried loops accurately detect high profile trucks. Any deep buried loop can provide accurate detections up to a maximum depth of 15". For a 15" deep loop, three or four turns of wire with a medium sensitivity setting will achieve 100% detection without any spillover effects if reasonable care is taken to keep the loop edge away from the lane line.

TABLE 3. MANHOLE SURFACE LOOP ON HIGH SENSITIVITY.

Number of Turns	Percent of Spillovers
2	0.0
3	3.4
4	13.1
5	17.7

TABLE 4. PERCENT DETECTION OF WATER SATURATED LOOP ON LOW SENSITIVITY.

Number of Turns	15" Deep	20" Deep
2	61.4	52/6
3	100.0	94.3
4	100.0	97.9
5	100.0	89.8

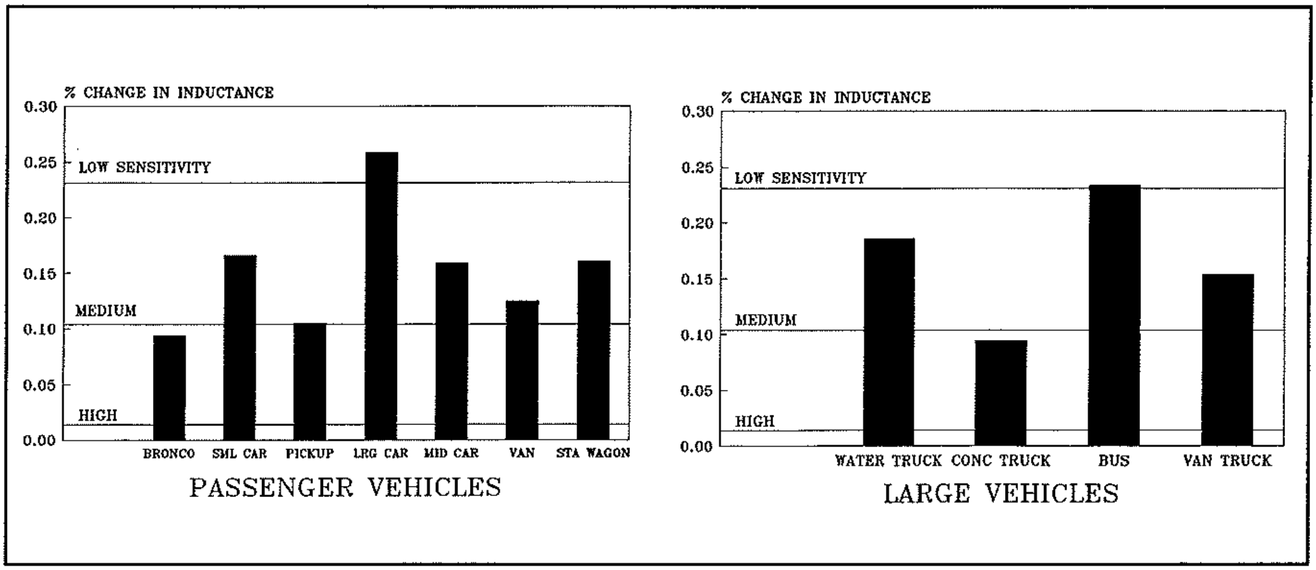

FIGURE 5. PERCENT SHIFT IN INDUCTANCE BY VEHICLE TYPE FOR DEEP BURIED LOOPS.

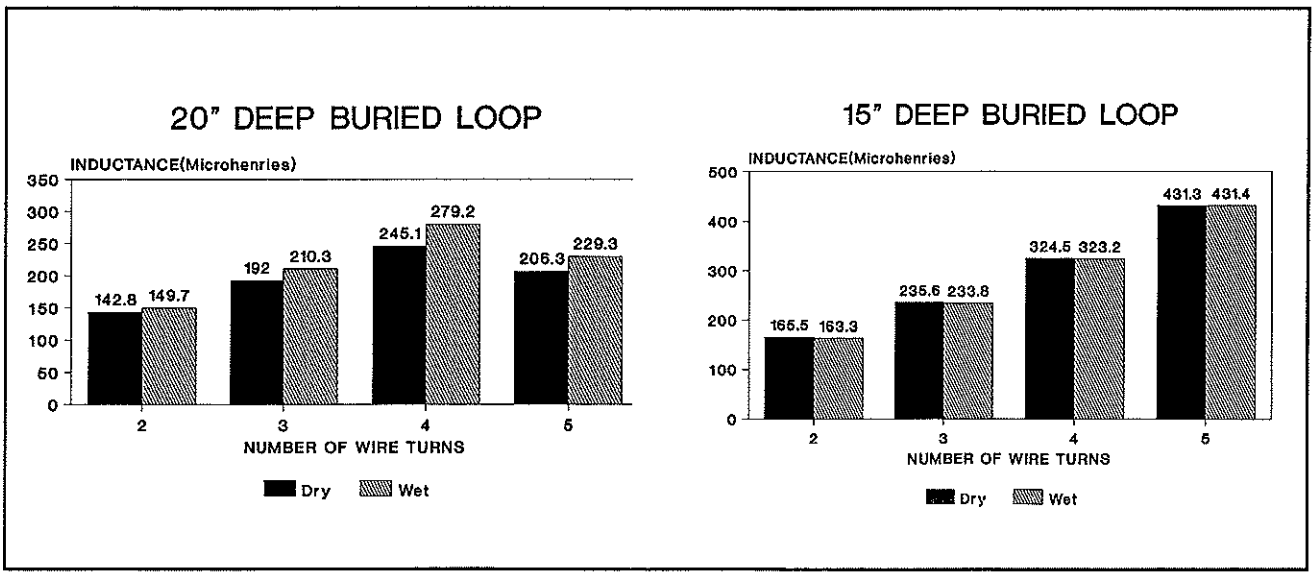

FIGURE 6. WET DRY INDUCTANCE OF A DEEP BURIED LOOP.

Water and Manhole Comments

Water had virtually no effect on loop detector performance. A slight increase in the inductance did occur between the dry and wet conditions, but the TX series detector was able to adjust. The same detection accuracy resulted in both the dry and wet conditions. It is not worth the added expense to insure that a loop is 100 percent sealed from water since water has no effect on loop detector accuracy.

The manhole had no effect on the detection accuracy of the surface loop. The loop detected just as well with the manhole as a loop without a manhole. The only consideration to be given when placing a loop near or around a manhole is the amount of maintenance and construction which will occur at the manhole. Damage to the loop by construction equipment should be avoided.

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The State of the Practice in Forecasting Turning Flows

JANIS L. PIPER

The purpose of this project was to develop a better understanding of the process of forecasting turning flows. Review literature provided information about the state of the research in the area of turning flow forecasts and provided information about the models available for use in making turning flow forecasts. A telephone survey was performed to obtain information about the state of the practice in forecasting turning flows in the United States. Turning flow proportions were analyzed to show a correlation between turning flow proportion and functional classification, and in doing so, average turning flow proportions were developed.

INTRODUCTION

Accurate estimates of turning flows are important to the development and design of new or expanded facilities. Errors in estimates can lead to over- or under-design, and these errors could cost the agency involved both time and money.

Turning flow estimates have an impact on the design process. Evaluating the need for one or more left-turn bays or for three- or four-phase signal timing, adding right-turn bays, or constructing a grade-separated intersection are all considerations which, in some way, are based on turning flow volumes.

This research attempted to offer a better understanding of the methods available for making turning flow forecasts as well as the methods currently in use. By providing information on the methods available in the literature for forecasting turning flows and the current state of the practice, and providing estimates of standard turning proportions based on the functional classification of the intersecting roads, it is possible to reduce some of the risks involved in forecasting turning flows for new facilities and to improve the intersection design process as a whole.

The main objective of this study was to compile information in the area of intersection turning flow forecasts. The state of the practice for forecasting turning flows was determined by interviewing representatives from 10 states in order to achieve diversity, as well as develop information that would relate to the needs of the state of Texas. The results of this survey are reported below in the section entitled, "State of the Practice." The second major area covered in this study is the development of average turning flow proportions based on the functional classifications of the intersecting roadways, while showing a correlation between functional classification and turning flow proportion. Data were collected and separated by functional classification. The turning proportions were analyzed in three different ways. The results are reported below in the section titled "Development of Average Turning Proportions." A recommendation is made to use one set of proportions and the reasoning is described. The purpose of the second part of the study is to show a relationship between turning flow proportion and functional classification and provide some general form of historical information that can be used as initial input for the various models that estimate turning flows.

This study represents a small portion of a larger study involving the corridor analysis process. In the corridor analysis process, groups of intersections are evaluated and turning flows at each are determined. Turning flows determined in this manner take into consideration not only forecasted approach and departure volumes but also the effect that each intersection has on the others around it and the effect of nearby facilities and developments on the operation of the facility in question.

PROBLEM STATEMENT

Analyzing turning flows is a requirement when designing or upgrading intersections. In the case of an upgrade, existing conditions can be analyzed and future turning flows can be predicted. When considering the develoment of a new intersection where existing information is not available, it is necessary to use other methods for forecasting turning flows. This may also be the case in an area where an agency's budget limits the ability to obtain physical counts at an existing location. The methods available to make these predictions are diverse and little is known about state of the practice in this realm of traffic forecasting.

LITERATURE REVIEW

Many attempts have been made to reduce or eliminate the need for labor-intensive, manual counting of intersection turning flow volumes. Some of these methods are very simple mathematical solutions, while others are complicated algorithms and iterative processes. Few of the methods were designed with forecasting specifically in mind.

Some of the methods documented are based on simple algorithms, while others are extremely complex. This complexity, however, does not necessarily relate to improvements in accuracy. Many of the procedures involve iterative processes, and most require some knowledge of expected turning proportions. Some of the procedures have been field tested, and the results can be obtained. Others are theoretical with no testing documented outside the laboratory.

Marshall offered an option for reducing the number of observations necessary for counting intersection turning flow volumes (1). The method requires one-way volumes into and out of the intersection. By using the one-way traffic volumes and the manual turning flow counts at some of the approaches to the intersection, the remaining turning flow counts can be estimated by following a series of simple mathematical equations. Marshall offered a method for reducing the need for observers to count turning traffic but did not address the problem of forecasting turning flows.

Jeffreys and Norman published articles that discuss a non-iterative method which works on the principal of developing a "realistic" set of turning flows (2, 3). The method uses linear programming and elementary "rook's tours" to develop a set of turning flows for an intersection. The method was referred to as the ordered rook's tour method. The first article primarily presented the method and examples of its application. The second article, which further developed the ideas of the first paper, presented two alternatives related to this idea and performed a comparison between the methods and the entropy maximization method. Conclusions were that the methods yielded similar results when the prior information available was close to balancing the given situation. Otherwise, it was felt that the method could be improved by going through a few iterations using the Furness balancing factor model before applying the method (4).

National Cooperative Highway Research Program (NCHRP) Report #255 presents several methods for predicting turning flow volumes which are dependent on the available information (5). Three factoring procedures--the ratio method, the difference method, and the combined method-- are available. Each requires the following directional or nondirectional information: future year turning flow forecast, base year turning flow assignment, and base year turning flow counts. If base year turning flow volumes are not available, approach link volumes taken from traffic forecasting models may be substituted in the ratio method only, making it the only method of the three which offers a solution to the forecasting problem. Iterative procedures are offered for four-way intersections for use when either directional or nondirectional future year link volumes are known. Non-iterative procedures are offered for the development of turning flows at three-way intersections, for either directional or nondirectional link volumes.

Mekky discussed a log-linear method for estimating turning flows at intersections (6). The forecasting matrix developed can be solved through a series of iterations similar to the Furness iteration method or the bi-proportional method (4, 7). Mekky introduced his method and stated that it "may be worth considering and testing by experimental evidence." The method was later referred to as the entropy maximization method (3). Bell further discussed Mekky's procedure, offering standard errors and confidence intervals for the estimates developed using this model (8). The article discusses the sampling approach to obtaining prior information and looks at estimating, rather than forecasting, turning flows.

Articles by van Zuylen, Hauer, et al, and Schaefer discuss the use of the iterative technique developed by Kruithof (Kruithof's algorithm) to balance possible turning flows at an intersection (9, 10, 11). Van Zuylen offered an information minimizing method while Hauer offered a maximum likelihood method, also referred to as the bi-proportional method. Schaefer summarized the efforts of van Zuylen and Hauer in his article which concentrating mainly on the work of Hauer. Schaefer concluded that Hauer's method was a "useful tool for developing intersection turning movement estimates," but that "selection of an appropriate estimate of the intersection turning proportions is key to developing an accurate estimate of the actual flows."

Maher presented a non-iterative method in which the development of turning flow estimates is approached by using Bayesian Statistical Inference (12). At the time of the article, no detailed tests had been performed to compare the method to other methods available, but it was thought to be comparable to the maximum entropy approach and minimum information approach previously discussed. Maher published a second article where he compared the information minimizing method and the maximum likelihood method with his own Bayesian method (13). In presenting the maximum likelihood method, he stated that "Hauer, et al, claimed to have presented a maximum likelihood formulation of the same method [information minimizing method], but this is incorrect; the estimates produced should properly be described as modal values." Conclusions were that the Bayesian model appeared to be the most appropriate choice to estimate turning flows at intersections. A third article comparing the information minimizing method, the Bayesian method, and a modification of the Bayesian method reached a similar conclusion (14).

More recently, Furth has developed a method which works on the principal developed by Hauer (15). Furth detailed the development of a turning propensity model and the factors affecting turning flows. The propensity matrix produced by this model can then be applied as the initial input of expected turning flows in another model. He stated that "the overall performance of the propensity model is very encouraging," and that the average prediction error in the model was of similar magnitude to the day-to-day variations in turning flows.

Other research includes work by Luk on the bi-proportional solution to the information minimizing procedure (16). Adebisi and Buehler offered comparisons between the various models and provided some results of testing performed on the models for accuracy (17, 18).

Most of the methods described above require an estimate of the approach and departure volumes at the intersection as well as some historical information about turning proportions at the location. Work involved in many of the papers included methods for acquiring this historical information.

Standard turning flow proportions are a form of historical information that can be provided for an intersection that is in the development stages. The "1965 Highway Capacity Manual" indicated that estimates of 10 percent left, 10 percent right, and 80 percent through traffic is considered the average condition for an urban intersection (19). Hauer reported that differences in turning flow proportions could be attributed to the functions of the intersecting roads as well as time of day, direction of movement, and location in the urban area. A major portion of the differences in turning flow proportions, Hauer thought, could be attributed to the functional classification of the intersecting roads.

STATE OF THE PRACTICE

In order to develop an understanding of the state of the practice for forecasting turning flows, a telephone survey was conducted to determine the methods being used by various state transportation agencies. The literature review provided an overview of the methods available but gave little indication of the acceptance of the methods in actual practice. Transportation agencies from ten states were surveyed, including the Texas State Department of Transportation, and the results of the interviews are summarized below. Some questions may not have applied to the state being interviewed and discretion was used to determine whether the response was complete or if further questioning was necessary in order to acquire a full understanding of the methods being described.

Arizona (20, 21)

Following an interview with representatives from the Arizona Department of Transportation (DOT) and the city of Phoenix, it was determined that there were two methodologies used in Arizona. The Arizona DOT evaluates turning flows from a regional planning perspective. The department currently uses the Urban Transportation Planning System (UTPS) as a traffic model and uses the turning flows directly from the model output as the future turning flow forecasts. The flows were often adjusted based on existing information or some professional judgment, but the output of the traffic model was the sole source of turning counts outside of physical counting of the intersections in question. The accuracy of the output was unknown and was considered suspect by the department. Previously, a model developed by the Federal Highway Administration (FHWA) called PlanPak was used as the traffic forecasting model. This model allowed the planner to input constraints on the turning flows. The representative of the Arizona Department of Transportation interviewed considered these turning flow estimates to be more accurate than the current estimates being produced by UTPS.

The city of Phoenix looked at turning flows in a more localized manner. The geometry and signal timing at intersections were used with the turning flow estimates to develop a level of service (LOS) estimate. Two programs are currently being used by the city to develop turning flow forecasts. The first is a program called TURNFLOW which can be purchased through the Center for Microcomputers in Transportation (McTrans) in Gainesville, Florida. The second is a mathematical algorithm which uses an iterative process to determine the turning flows from the approach and departure volumes and initial turning estimates input into the program. The program was developed on a Lotus spreadsheet by a staff member working for the city of Phoenix and is based on the algorithm reported in Transportation Research Record (TRR) 795 (10). Both programs require that initial estimates of turns be input, as well as average daily traffic (ADT) approach and departure volumes.

The ADT volumes were obtained from the Arizona DOT's Transportation Planning Office and were the output of the UTPS model. Different turning flow proportions are used as the initial input to the intersection analysis program. The initial estimates vary based on the peak hour approach volume in question and the quadrant in the city where the intersection is located. On the average, these proportions were 10 to 12 percent left and right turns based on the total approach volume. Turning flows were considered to be heavier in the peak and lighter in the off-peak hour, assuming that when the directional distribution was heavier in one direction the turning flows would also be heavier in that direction. The proportions were based on historical information and actual turning counts and were developed by the

Phoenix Department of Transportation Planning. Field turning flow counts were also available for a number of intersections and the proportions from an intersection with similar characteristics may have been used in the absence of any other information.

Both TURNFLOW and the mathematical algorithm were used by the city of Phoenix, and neither was considered more accurate than the other. If the results of either program were not considered plausible, they were adjusted manually taking into consideration the impact of related facilities and other environmental considerations. Most of the work was limited to the intersections of arterial streets. Analysis of intersections with collector and local streets was generally considered less critical. The two major cities, Phoenix and Tucson, where the analyses were conducted had arterial streets spaced approximately one mile apart. This provided for very regular traffic flow patterns. Because the predictions being made were 20-year projections, it was difficult for the city of Phoenix to judge the accuracy of the output, but the transportation planning department considered the results to be acceptable.

California (22, 23)

Interviews with a representative of the California Department of Transportation (CALTRANS) and a representative from one of the state's regional planning agencies provided information on the practices of turning flow forecasting in California.

At the state level, TRANPLAN and UTPS were used as traffic forecasting programs. The output of turning flows from the models was analyzed and often required adjustment. In order to "smooth out" the output, a very localized hand assignment was done. At the state level, the planning analysis encompassed large areas and generally was not localized to a single intersection. Base year calibration was completed on the model output by comparing the base year output to actual counts; and the future traffic volumes were, therefore, considered to be accurate.

At the regional level, the turning flows were estimated by several traffic forecasting programs, including the Maximum Entropy Matrix Estimation (ME2) program, TRANPLAN, UTPS, and others. Traffic engineering studies, including link forecasts as well as turn predictions, were frequently subcontracted out to an engineering firm. The firm generally used a traffic forecasting model to analyze the traffic volumes, including turning flows. The turning flows generated by the traffic forecasting programs were then manipulated to account for predicted land use and other localized factors. The volumes were looked at for "reasonableness" and manually corrected until the engineer considered them reasonable. Any more specialized form of turning flow forecasting was considered to be trivial since there were very few new intersection developments in California.

Florida (24, 25)

Conversations with representatives of District 4 of the Florida Department of Transportation provided an overview of the procedures used by that district to forecast turning flows. Although each district acts independently, the representatives interviewed concurred that the methods used in the other districts would be similar if not identical to those described below.

Turning flow forecasts were used to determine whether an intersection would be applicable for the given location or if a grade separated interchange or a partial interchange would be required. The geometry of the intersection was also determined using the turning flow forecasts. The traffic modeling program used by the state was the Florida Standard Urban Transportation Model System (FSUTMS) which was a modification of PlanPak. Turning flows were developed through the program's assignment model. If existing counts were available, the output was analyzed and manually adjusted to reflect the existing conditions. If it was a new development or existing counts were unavailable, the model output was used directly. The turning flow estimates were then delivered to the design department where adjustments may have been made.

District 4's planning section delivered the FSUTMS output to the traffic operations section. The data were analyzed and compared to existing conditions. A growth factor was developed, and it was assumed that the entire corridor would grow at the same rate. If there was an isolated development nearby, volumes were approximated for that development using the Institute of Transportation Engineer's (ITE) manual, Trip Generation, and these volumes were added to the future count estimates at the affected locations.

Illinois (26)

A representative of the Illinois Department of Transportation, District 5, responded to the survey questions. There are nine districts in Illinois; but Lee Bates, Traffic Policy Engineer of Central Bureau of Traffic, Illinois DOT, felt that one response would adequately represent the procedures used throughout the districts.

Turning flow estimates were used to determine intersection geometry and signal timing. Illinois did not conduct isolated intersection analyses unless they were preparing an isolated site impact study. For the small scale, local situation, use of a computer program was not felt to be necessary. For the large scale situation, they were using the Quick Response System (QRS) Site Impact Analysis program. The Transportation Planning Modeling Software (TRANPLAN) developed by The Urban Analysis Group was expected to be implemented in urban areas; but assignment and analysis was done by hand. The Illinois DOT used historical trends, trip generation rates, and other known factors to predict link volumes and then distributed the volumes manually throughout the system. Intersections were balanced by hand, first analyzing two-way link volumes, then analyzing the directional distribution to develop turning flows. Environmental factors were taken into account at the same time. Once traffic volumes and turning flows had been generated for the base year, a straight-line increase in volume was used to generate 20-year predictions. It was assumed that if the directional split and the total volume at the intersection approaches were known, it was possible to calculate one, and only one, reasonable set of numbers that would balance the intersection. It was also assumed that the current manual assignment process accomplished this goal. In some cases, when an existing count was not available, a nearby intersection having similar characteristics was used to approximate the turning traffic for the base year. The respondent personally recorded the results of this process and said that "it (the manual turning flow forecasting process) is as accurate as the traffic counting process." By checking the results of the manual calculations against actual traffic counts, the respondent refined the skills necessary to make accurate predictions.

Indiana (27, 28)

Representatives of the Indiana Department of Highways were interviewed to determine the state of the practice in forecasting turning flows in that state. Turning flow forecasts were used to determine intersection geometry, signal system timing, capacity analysis, and LOS determination. Indiana did not have a state-wide traffic forecasting model. In rural areas, forecasting was done as a straight-line projection. In urban areas, Metropolitan Planning Organizations (MPO's) provide the state with main line link volumes, and these were manipulated manually to develop flows through the corridors. In the future, the state plans to implement TRANPLAN as a forecasting model.

For existing intersections, turning flows were forecasted using a growth rate consistent with the link projections. If a major traffic generator existed in the area, traffic from that generator was forecasted separately and added to the rest of the corridor projections, and the turns at the intersections were adjusted accordingly. For intersections which were developed as a part of new construction, the traffic forecasts were reviewed and an attempt was made, using professional judgement, to predict how traffic would move through the system. Turning flows were developed based on the path the traffic was expected to follow through the system. The Indiana Department of Highways often relied on manual counts at similar intersections to forecast how traffic would move at the new intersection. No records were kept to assure the accuracy of any of the estimates.

Massachusetts (29, 30)

Interviews with two representatives of the Central Transportation Planning Staff (CTPS) of the Massachusetts Department of Transportation yielded information on turning flow forecasting from two viewpoints. In terms of network modeling (i.e., analyzing projects at a regional level) it was considered difficult to convert the traffic volume at an existing intersection to future turning flows when the network was changing drastically. The best approach was to make the models as accurate as possible and give the network modeling output to the traffic engineers to do a more localized analysis. There was an ongoing project in Boston, the Central Artery Project, where I-93 was to be completely removed and relocated, resulting in the need to redesign 100 to 150 intersections. The methods used to develop the turning flow counts for the new intersections, some of which would be completely relocated, were to look at the base year counts compared to the traffic forecasting model output, develop a correction factor for the model output to match the existing counts, then apply the same correction factors to the future year outputs with the new intersections in place in the model. The two main traffic forecasting models used at this level were UTPS and TRANPLAN.

For corridor planning studies and more localized studies, trip tables were generated directly using The Highway Emulator (THE). The THE model was developed by Edward Bromage while employed by CTPS. The program uses the maximum entropy approach of Willumson and van Zuylen, and is available through McTrans. Daniel Beagan, Deputy Director of CTPS, stated that standard errors for this program were in the two to three percent range (31).

There were also programs available that had been developed to analyze turning flows specifically. The most basic of these programs looked at one intersection only. It was developed in part by E. Pagitsas formerly of Toronto, Canada, and currently employed by CTPS and described in TRR 795 (10). Old turning flow counts or small sample count were used to develop turning flow percentages for input, and directional volumes were available through actual counts or model output. Another program was developed by Peter Furth at Northeastern University and funded by CTPS to expand on the methodology of Pagitsas' program, adding the ability to input information about the geometry and the location of the site in the urban area. The program also gave the user the ability to describe the relationship of the site to other facilities. Although complete and available for purchase through McTrans, the program had not been implemented by CTPS.

If no counts were available, 10 percent left and right turns were sometimes used as a rule of thumb as input to the model developed by Pagitsas (10). The results of the Toronto work included some average turning proportions based on functional classification. The method was considered to be fairly accurate based on citation in two separate articles (10, 18). Typically the transpose of the AM peak counts was used for the PM and vice versa in using the program to develop turning estimates. This eliminated the additional effort required to get counts for both periods.

Michigan (32)

An interview with a representative of the Michigan Department of Transportation indicated an effort was underway in Michigan to develop better turning flow estimates. In areas where there were congestion problems, accurate turning information was considered to be imperative. It was thought that use of an average turning proportion or any other standardized estimate might be a costly mistake, and for that reason counts were ordered as a matter of course on all major projects. It was felt that using average turning flow proportions opened an avenue for the project to be challenged during the public hearing process and that it would be less expensive in the long run to have crews make manual counts at the intersections. Turning flows, for urban projects, were considered to be important in the design of the intersection from the lane configuration all the way to the signal timing. For rural projects, through volumes were more important than turning flows. Turning flows were checked only at high volume intersections along a rural route.

TRANPLAN is currently being used as a traffic modeling program. For a new or proposed intersection in an urban area, the model output provided a prediction of turning flows as well as through traffic. For rural intersections assumptions were made based on the nearest intersection to predict how the traffic would flow. Michigan had implemented an extensive traffic counting program in order to develop growth factors. Growth factors were applied to the turning flow estimates to get future year turning flows.

A program is currently being developed in the Bureau of Transportation Planning, Michigan Department of Transportation, to refine the process of forecasting turning flows. The NCHRP Report #255 software was modified to use an iterative process to balance base year ADT turning flows (5). A Lotus-based spreadsheet then applied the growth factors and allowed for the increase or decrease of any of the turning flows due to nearby developments or other environmental influences on the traffic. The results were verified by checking them against the results of manual estimates. The results of the two methods were within 5 to 15 vehicles of each other. Otherwise, the accuracy of the counts was not checked, but the estimates were kept as historical information.

New York (33)

A telephone interview with a representative of the New York State Department of Transportation provided information on the methods used for forecasting turning flows. Turning flow estimates were considered necessary in the design of new intersections (including items such as provision for turning lanes), and better estimates of vehicle queue lengths could be developed based on a more accurate turning flow estimate.

Turning flow forecasts for a given highway intersection came from the system forecasts developed through the traffic modeling process. In general, link level forecasts were used in combination with existing turning flow distributions to generate future turning flow distributions.

Traffic modeling was previously done by the state using a state developed mainframe program. Currently, the State of New York encourages its MPO's to use the Transportation Modeling System (TMODEL2) available through McTrans. This program allows the user to record and recall turning flow information at any intersection. The program also has a corresponding software package that does the "Highway Capacity Manual" (HCM) signalized intersection analysis. Other programs used throughout the state included TRANPLAN and UTPS.

TMODEL2 was considered inaccurate for turning flow estimates on a regional planning basis, because in regional planning, little was done to calibrate the turning flows. In the corridor analysis process, more emphasis was put on the calibration of the turning flows. For site impact studies, trip generation and manual turn assignment were used. The traffic was then added to the existing traffic in order to study the impact. For future highway development, reasonable judgment was used, and traffic volumes were followed throughout the system to develop turning flows. Some effort was being made to analyze the relationship between functional classification and the capacity of intersections, but the respondent was not aware of any work in the area of turning flows per se. No effort was being made to check the accuracy of the estimates.

Ohio (34, 35)

The Ohio Department of Transportation currently uses turning flow estimates in the development and planning phases for new intersections, especially in the area of intersection geometry.

The traffic modeling program used by the state was the FHWA version of PlanPak. The program assigns traffic volumes on a minimum time path through the network, and turning flows are estimated based on the assignment of the traffic volumes. By using the model in conjunction with existing ground counts, the Ohio DOT would calibrate the model to balance the estimated turns with the existing conditions. If a new development was being considered, the model estimates were more likely to be used. A professional judgment of how traffic would flow through the system was developed by the user, and the turning flows were adjusted accordingly. According to the respondent they "haven't found anything better than PlanPak [to provide them with turning flow forecasts]." In order to check the accuracy of the estimates, the original output was kept as historical information, and traffic volumes were checked against it. The Ohio DOT considered the results of this type of turning flow forecasting to be very good.

Texas (36)

A telephone conversation with a representative of the Texas State Department of Transportation (TxDOT) in Texas revealed that turning flow counts were forecasted for use in the design process. The turning flow estimates were computed manually. Each approach at the intersection was studied separately and was assigned a different turning proportion. If there was known information available for any approach at the intersection, that approach was analyzed first. Known information included observed turning flows, familiarity with the area, or information from previous, similar projects. If nothing at all was known about the intersection, a proportional method was used. In the proportional method, a ratio was used where each approach was assigned a ratio equal to the ratio of its volume over the total volume at the intersection. Turning flows were assigned by applying these ratios. For instance, if the first approach carried one-third of the total traffic volume at the intersection, then one-third of the traffic at each of the other approaches would be assigned to turn onto the first approach. Some engineering judgment was used to adjust these numbers, particularly at new intersections.

When turning flows were generated through a traffic model, they were manually adjusted by a traffic engineer familiar with traffic in general, how intersections operate, and how to compare roadways in terms of their operation. Considerations when adjusting the model output included where development was located, access points to the existing system, and the type of traffic that the location was expected to generate.

Computer programs for generating turning flow forecasts that were promoted by FHWA at their workshops and short courses were considered, but manual methods were considered to be quicker and more efficient. The respondent considered the manual estimates to be accurate to within plus or minus 10 percent and stated that work has been done to check the accuracy of the estimates.

Summary

It can be concluded from the results of the telephone survey that the state of the practice in forecasting turning flows is widely diversified. The information on alternative methods for forecasting turning flows is available, and many of the agencies involved in the survey were aware of this availability. It appears that although a few states are implementing turning flow forecasting programs, most rely heavily on professional judgement to develop turning flow forecasts. Interest in the area was varied, and it is felt that as interest increases, more of the methods will be tested, implemented, and improved.

DEVELOPMENT OF AVERAGE TURNING PROPORTIONS

The objective of this portion of the research is to demonstrate a correlation between turning flow proportions and functional classification. Average turning flow proportions were also calculated for possible use in turning flow forecasting models. In relating turning flow proportions to functional classification, the assumption was made that turning flow proportions were not directly related to approach volume. This assumption was tested and the results are shown in the section, "Turning Proportion vs. Approach Volume."

AM and PM peak turning flow proportions were compared to look for differences in the mean turning flow proportions by functional classification. The AM peak counts were then analyzed in three different ways, yielding three slightly different turning proportions. The three methods are described and an explanation is given why one set of proportions is recommended over the other two. The analyses were made on four-way signalized intersections only and, with no further information, can be considered valid only for four-way signalized intersections.

Functional Classification

Four functional classifications were analyzed in this research: major arterial, minor arterial, collector, and local road. Data were collected from a number of urban areas, mainly in Texas, based on the following definitions (37):

1. *Major Arterial* - serves major through movements between important centers of activities in a metropolitan area and a substantial portion of trips entering and leaving the area. In smaller urban areas (under 50,000), its importance is derived from the service provided to traffic passing through the urban area. Service to abutting land is very subordinate to the function of moving through traffic.

2. *Minor Arterial* - is a facility that connects and supports the major arterial system. Although its main function is still traffic mobility, it performs this function at a somewhat lower level and places more emphasis on land access than the major arterial.

3. *Collector Street* - provides both land access and traffic circulation service within residential, commercial, and industrial areas. Access function is more important than that of arterials.

4. *Local Street* - any roadway not described in the other categories.

It is assumed that the roads were accurately classified. Table 1 shows the distribution of the data collected from the various urban locations over the range of classifications.

Turning Proportion vs. Approach Volume

Although by definition, functional classification gives some indication of the relative volume of traffic on the roadway, a major arterial in a city of two million people would be expected to carry a higher volume than a major arterial in a city of 70,000. Major arterials are classified as such because of the type of traffic they carry and the amount of access provided to abutting properties. To analyze turning flow proportions on the basis of functional classification, it was necessary to demonstrate that the approach volume and turning flow proportion were not directly related. This allows the use of the same turning flow proportion for the same functional classification regardless of the size of the city or the traffic volumes at that location.

Two methods were used to demonstrate that turning flow proportion was not directly related to approach volume. The first was to simply plot the approach volume vs. turning flow proportion for each functional classification and each type of turning flow. The second method was to calculate the coefficient of correlation between approach volume and turning flow proportion for each functional classification and each turning movement (63). The results are shown in Table 2. The correlation coefficient, a number between -1 and +1, demonstrates how closely the data approximates a linear relationship. As the coefficient approaches the lower or upper limit of the range, the data approximates a negative or positive linear relationship, respectively. The results shown in Table 2 indicate a non-linear relationship between approach volume and turning proportion. It was, therefore, assumed that the average turning flow proportions discussed in subsequent sections could be considered valid in any metropolitan area, regardless of the approach traffic volumes.

Average Turning Flow Proportion Analysis

The data analyzed were from various urban areas (mainly in Texas) and consisted of 988 intersection approaches, from 247 different intersections. A list of the data sources and the number of approaches from each can be found in Table 1. All of the data analyzed were from four-way signalized intersections, and the analysis was done without regard to the size of the city, traffic volumes, or location in the urban system (e.g., Central Business District). AM and PM peak counts were compared using the Student's t-Test and were found to be similar populations (63). The results allow the assumption to be made that the turning proportion generally remains the same regardless of the directional distribution of the traffic, and this eliminates the need to analyze the PM peak counts separately. Further investigation would be required to assure that the proportions are also valid for the noon peak hour or the off-peak hour traffic. Three methods of analysis were used to develop turning proportions. Standard deviations and confidence intervals were calculated for two of the methods. The third method provides another option but time limitations prohibited the calculation of any information besides the mean.

Method 1

It was shown in the previous section that the turning proportions were not related to the approach volume. In order to eliminate regard for the approach volume, the data were converted to turning proportions. Left turns, right turns, and through movements were analyzed separately. The population analyzed was a set of proportions for each functional classification. One proportion was calculated for each turning flow movement at each intersection approach. Histograms were plotted for each turning flow and each functional classification. Because the sample sizes were considered large, a normal distribution was assumed for analysis purposes. The main potential error in assuming normality is that the limits of the proportion are 0 to 1 (i.e., it is a fixed-ended distribution). The limits of a normal distribution are negative to positive infinity. There is a method available for converting a fixed-ended distribution to a normal distribution and it is discussed as Method 2.

The calculated proportions were analyzed as a normal distribution. Mean, variance, standard deviation, and the 90 percent confidence interval were calculated for each turning flow movement for each functional classification. The results are listed, along with the number of approaches analyzed for each functional classification in Tables 3 to 8.

TABLE 1. DATA SOURCES - NUMBER OF APPROACHES ANALYZED PER CATEGORY.

Source	Number of Approaches Analyzed													
	M-M	M-A	M-C	M-L	A-M	A-A	A-C	A-L	C-M	C-A	C-C	L-M	L-A	L-L
San Antonio, TX (38, 39)	36	28	50	12	28	12	6	0	50	6	0	12	0	0
Duncanville, TX (40)	12	2	2	0	2	0	0	0	2	0	0	0	0	0
Euless, TX (41)	4	4	6	0	4	0	0	0	6	0	0	0	0	0
Garland, TX (42, 43)	20	16	2	4	16	0	10	4	2	10	0	4	4	0
Corpus Christi, TX (44, 45)	0	2	0	0	2	32	10	2	0	10	0	0	2	0
Fort Worth, TX (46, 47)	12	10	20	6	10	0	0	0	20	0	0	6	0	0
Hurst, TX (48, 49)	0	2	0	0	2	0	6	0	0	6	0	0	0	0
College Station, TX (50, 51)	4	4	0	4	4	0	0	0	0	0	0	4	0	0
Arlington, TX (52, 53)	8	10	10	2	10	8	14	0	10	14	16	2	0	0
San Angelo, TX (54, 55)	0	12	12	6	12	4	14	6	12	14	20	6	6	12
Austin, TX (56, 57)	12	8	18	16	8	0	0	0	18	0	0	16	0	0
Addison, TX (58, 59)	8	6	8	0	6	8	10	0	8	10	0	0	0	0
Corsicana, TX (60, 61)	0	0	0	0	0	0	2	4	0	2	0	0	4	16
Dauphin County, PA (62)	0	0	0	0	0	0	2	0	0	2	0	0	0	0

TABLE 2. CORRELATION COEFFICIENTS OF APPROACH VOLUMES COMPARED TO TURNING FLOW PROPORTIONS.

Functional Classification	Left Turn	Through	Right Turn
Major Arterial to Major Arterial	-0.18	0.32	-0.29
Major Arterial to Minor Arterial	-0.21	0.28	-0.18
Major Arterial to Collector	-0.37	0.43	-0.28
Major Arterial to Local Road	-0.09	0.21	-0.20
Minor Arterial to Major Arterial	-0.02	-0.05	0.09
Minor Arterial to Minor Arterial	-0.13	0.36	-0.50
Minor Arterial to Collector	-0.09	0.16	-0.13
Minor Arterial to Local Road	-0.27	0.36	-0.33
Collector to Major Arterial	0.01	0.18	-0.25
Collector to Minor Arterial	0.03	-0.09	0.10
Collector to Collector	-0.16	0.34	-0.30
Local Road to Major Arterial	-0.28	0.10	0.20
Local Road to Minor Arterial	0.26	-0.12	-0.11
Local Road to Local Road	-0.04	0.03	-0.02

TABLE 3. TURNING FLOW PROPORTION ESTIMATES - LEFT TURNING FLOW.

Functional Classification	Mean Proportion	Weighted Average	Mean Developed Through Transformer
Major Arterial to Major Arterial	0.1662	0.1522	0.1489
Major Arterial to Minor Arterial	0.0868	0.0790	0.0759
Major Arterial to Collector	0.0697	0.0521	0.0559
Major Arterial to Local Road	0.0502	0.0474	0.0397
Minor Arterial to Major Arterial	0.2546	0.2518	0.2346
Minor Arterial to Minor Arterial	0.1494	0.1395	0.1247
Minor Arterial to Collector	0.0971	0.0925	0.0804
Minor Arterial to Local Road	0.0746	0.0551	0.0632
Collector to Major Arterial	0.2614	0.2625	0.2457
Collector to Minor Arterial	0.2066	0.2112	0.1824
Collector to Collector	0.1460	0.1300	0.1229
Local Road to Major Arterial	0.3464	0.2922	0.3337
Local Road to Minor Arterial	0.2603	0.3026	0.2470
Local Road to Local Road	0.1303	0.1283	0.1111

TABLE 4. TURNING FLOW PROPORTION ESTIMATES - THROUGH TRAFFIC FLOW.

Functional Classification	Mean Proportion	Weighted Average	Mean Developed Through Transformer
Major Arterial to Major Arterial	0.6719	0.7097	0.6803
Major Arterial to Minor Arterial	0.8110	0.8290	0.8237
Major Arterial to Collector	0.8627	0.8931	0.8768
Major Arterial to Local Road	0.9082	0.9171	0.9204
Minor Arterial to Major Arterial	0.5202	0.5123	0.5180
Minor Arterial to Minor Arterial	0.6973	0.7400	0.7124
Minor Arterial to Collector	0.8109	0.8217	0.8246
Minor Arterial to Local Road	0.8152	0.8626	0.8285
Collector to Major Arterial	0.4454	0.4764	0.4357
Collector to Minor Arterial	0.5311	0.5119	0.5246
Collector to Collector	0.6671	0.7162	0.6780
Local Road to Major Arterial	0.2990	0.3171	0.2757
Local Road to Minor Arterial	0.4591	0.4367	0.4525
Local Road to Local Road	0.6669	0.6699	0.6762

TABLE 5. TURNING FLOW PROPORTION ESTIMATES - RIGHT TURNING FLOW.

Functional Classification	Mean Proportion	Weighted Average	Mean Developed Through Transformer
Major Arterial to Major Arterial	0.1619	0.1522	0.1424
Major Arterial to Minor Arterial	0.1022	0.0920	0.0865
Major Arterial to Collector	0.0676	0.0547	0.0554
Major Arterial to Local Road	0.0416	0.0355	0.0320
Minor Arterial to Major Arterial	0.2252	0.2359	0.2066
Minor Arterial to Minor Arterial	0.1532	0.1205	0.1364
Minor Arterial to Collector	0.0921	0.0857	0.0760
Minor Arterial to Local Road	0.1102	0.0823	0.0995
Collector to Major Arterial	0.2932	0.2611	0.2794
Collector to Minor Arterial	0.2623	0.2768	0.2446
Collector to Collector	0.1869	0.1538	0.1697
Local Road to Major Arterial	0.3546	0.3907	0.3453
Local Road to Minor Arterial	0.2806	0.2607	0.2540
Local Road to Local Road	0.2028	0.2018	0.1922

TABLE 6. NINETY PERCENT CONFIDENCE INTERVALS - LEFT TURNING FLOW.

Functional Classification	Average Proportion		Mean Developed Through Transformation	
	Lower Limit	Upper Limit	Lower Limit	Upper Limit
Major Arterial to Major Arterial	0.1471	0.1853	0.1312	0.1676
Major Arterial to Minor Arterial	0.0767	0.0971	0.0664	0.0861
Major Arterial to Collector	0.0613	0.0831	0.0456	0.0672
Major Arterial to Local Road	0.0380	0.0624	0.0302	0.0503
Minor Arterial to Major Arterial	0.2235	0.2857	0.2043	0.2664
Minor Arterial to Minor Arterial	0.1229	0.1759	0.1000	0.1516
Minor Arterial to Collector	0.0790	0.1152	0.0657	0.0965
Minor Arterial to Local Road	0.0502	0.0990	0.0415	0.0891
Collector to Major Arterial	0.2366	0.2870	0.2206	0.2716
Collector to Minor Arterial	0.1759	0.2373	0.1527	0.2142
Collector to Collector	0.1115	0.1805	0.0917	0.1580
Local Road to Major Arterial	0.3003	0.3925	0.2451	0.4287
Local Road to Minor Arterial	0.1973	0.3233	0.1375	0.3762
Local Road to Local Road	0.1019	0.1587	0.0759	0.1520

TABLE 7. NINETY PERCENT CONFIDENCE INTERVALS - THROUGH TRAFFIC FLOW.

Functional Classification	Average Proportion		Mean Developed Through Transformation	
	Lower Limit	Upper Limit	Lower Limit	Upper Limit
Major Arterial to Major Arterial	0.6438	0.7000	0.6499	0.7101
Major Arterial to Minor Arterial	0.7929	0.8289	0.8054	0.8412
Major Arterial to Collector	0.8381	0.8725	0.8229	0.9220
Major Arterial to Local Road	0.8911	0.9255	0.9040	0.9353
Minor Arterial to Major Arterial	0.4841	0.5563	0.4780	0.5579
Minor Arterial to Minor Arterial	0.6577	0.7369	0.6695	0.7535
Minor Arterial to Collector	0.7877	0.8339	0.8016	0.8465
Minor Arterial to Local Road	0.7710	0.8594	0.7801	0.8719
Collector to Major Arterial	0.4191	0.4865	0.3996	0.4721
Collector to Minor Arterial	0.4879	0.5743	0.4743	0.5747
Collector to Collector	0.6176	0.7168	0.6249	0.7288
Local Road to Major Arterial	0.2550	0.3432	0.1546	0.4166
Local Road to Minor Arterial	0.3879	0.5303	0.2603	0.6523
Local Road to Local Road	0.6164	0.7174	0.5901	0.7566

TABLE 8. NINETY PERCENT CONFIDENCE INTERVALS - RIGHT TURNING FLOW.

Functional Classification	Average Proportion		Mean Developed Through Transformation	
	Lower Limit	Upper Limit	Lower Limit	Upper Limit
Major Arterial to Major Arterial	0.1416	0.1814	0.1241	0.1617
Major Arterial to Minor Arterial	0.0866	0.1178	0.0737	0.1002
Major Arterial to Collector	0.0613	0.0837	0.0451	0.0667
Major Arterial to Local Road	0.0292	0.0540	0.0240	0.0413
Minor Arterial to Major Arterial	0.1968	0.2536	0.1800	0.2347
Minor Arterial to Minor Arterial	0.1309	0.1755	0.1151	0.1592
Minor Arterial to Collector	0.0749	0.1093	0.0620	0.0913
Minor Arterial to Local Road	0.0823	0.1379	0.0721	0.1308
Collector to Major Arterial	0.2635	0.3131	0.2545	0.3049
Collector to Minor Arterial	0.2311	0.2935	0.2129	0.2778
Collector to Collector	0.1490	0.2248	0.1350	0.2075
Local Road to Major Arterial	0.3122	0.3970	0.2618	0.4338
Local Road to Minor Arterial	0.2131	0.3483	0.1423	0.3852
Local Road to Local Road	0.1677	0.2381	0.1399	0.2508

Method 2

In order to eliminate the fixed-ended distribution and approximate a normal distribution, the following formula was used to transform each proportion (64):

$$\text{Arcsin}\ (\sqrt{p})$$

where:
p = the turning flow proportion

The transformed data were analyzed as in Method 1, and the resulting mean and confidence interval were transformed back to the original distribution by reversing the process of the transformation. Results are shown in Tables 3 to 8. One problem encountered with this transformation was that the summation of the left, through, and right turning proportions was not equal to one. Time limitations did not allow for the necessary investigation to determine the method to correct the proportions.

Method 3

The third estimate of a mean proportion is actually a ratio estimate (i.e., weighted average). This method used the turning volumes directly rather than the turning proportions. By adding up the turning volumes for each approach and dividing the sum by the sum of the total of all the approach volumes, a ratio estimate was calculated. This estimate gives more weight to the intersections with higher approach volumes. Although it was already pointed out that the approach volume and turning proportion are unrelated, it is still valid to weigh the approaches in this manner. A larger sample of vehicles should more closely approximate the mean turning proportion, and the turning proportions for the approaches with heavier volumes are not statistically different from those of approaches with lower volumes. It is reasonable, therefore, to weigh the higher volumes more to approximate the true mean. Results are shown in Tables 3 to 5.

Summary

In summarizing the results, it is necessary to qualify the validity of the mean turning proportions calculated for the functional classification categories containing few observations, particularly those with less than 30 approaches. In the local road to local road intersection category, 28 approaches represent only seven intersections. The results could be influenced by the qualities of a data set this small.

TABLE 9. LEFT AND RIGHT TURNING FLOW PROPORTIONS AND ACCURACY OF THE ESTIMATES.

Functional Classification	Number of Approaches Analyzed	Average Turning Proportions			
		Left	Accuracy	Right	Accuracy
Major Arterial to Major Arterial	116	0.1662	0.019	0.1619	0.020
Major Arterial to Minor Arterial	104	0.0868	0.010	0.1022	0.016
Major Arterial to Collector	128	0.0697	0.011	0.0676	0.011
Major Arterial to Local Road	50	0.0502	0.012	0.0416	0.012
Minor Arterial to Major Arterial	104	0.2546	0.031	0.2252	0.028
Minor Arterial to Minor Arterial	64	0.1494	0.027	0.1532	0.022
Minor Arterial to Collector	74	0.0971	0.018	0.0921	0.017
Minor Arterial to Local Road	16	0.0746	0.024	0.1102	0.028
Collector to Major Arterial	128	0.2614	0.025	0.2932	0.025
Collector to Minor Arterial	74	0.2066	0.031	0.2623	0.031
Collector to Collector	36	0.1460	0.034	0.1869	0.038
Local Road to Major Arterial	50	0.3464	0.046	0.3546	0.042
Local Road to Minor Arterial	16	0.2603	0.063	0.2806	0.068
Local Road to Local Road	28	0.1303	0.028	0.2028	0.035

Due to the limited time available to complete this study, the statistical analysis of the data was not as complete as initially intended. An analysis of the turning proportion distributions may have yielded a type of non-normal distribution that the data more closely approximated. The ratio estimates could have been further analyzed to determine the variance, standard deviation, and confidence intervals. The results obtained from the transformation could have been corrected to sum to one for the approach.

The average of the proportions, Method 1, is recommended for use as the average turning flow proportion estimate. The left- and right-turn proportions and their accuracies are shown in Table 9. This set was chosen for a number of reasons. There is no statistical proof that the distributions do not approximate a normal distribution. The limited testing was inconclusive, and no tests were attempted to approximate any other type of distribution. The results of the transformation were not considered adequate because the left, through, and right proportions did not sum to 1. Without a correction, these were considered invalid. Choosing the average proportion eliminates the approach volume as a factor in the calculation.

Further statistical analysis might prove that another proportion is better than that recommended. However, the proportions from the three methods were not very different, based on visual observation. Having clarified the choice of the average proportion as the recommended value, it is possible to respond to the question raised in the objective portion of this research; "Is there a direct relationship between turning flow proportion and functional classification?" Although limited statistical analysis yielded inconclusive results in comparing the mean proportions with each other, observation of the data in Table 9 shows an obvious trend which appears to indicate that there is a strong correlation between functional classification and turning flow proportion.

RESULTS AND RECOMMENDATIONS

The literature review revealed a variety of approaches to predicting turning flows at intersections. Some of these can be applied to forecasting, while others simply offer an option for reducing the labor-intensive effort of counting turning flows. In the state of the practice portion of this research, the telephone survey revealed that the amount of interest in and knowledge of the availability of these methods was diversified. It is felt, however, that interest will increase and the methods will be used and improved as the necessity is recognized. Further testing of the available methods is recommended, specifically with respect to forecasting turning flows for future intersection developments.

Analysis of turning flow proportion with respect to functional classification revealed that turning flow proportions appear to be related to functional classification. The average turning flow proportions presented in the report are an option when an engineer is interested in making a quick estimate of the turning traffic at an intersection or as initial input into one of the available turning flow forecasting models. The most important benefit of this information is the potential elimination of the labor-intensive manual counting of intersection turning flows. Further statistical analysis is recommended to verify the proportions recommended. It is also recommended that they be tested in the turning flow forecasting models for accuracy.

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Arterial Lane Closures Downstream of Signalized Intersections

ISABEL S. SIU

The objective of this ten-week study was to determine the critical distance between the intersection and the downstream lane closure needed to prevent intersection blockage for a given set of parameters. The parameters that were considered for the development of the lane closure guidelines are: number of open lanes, number of closed lanes, lane closure capacity of open lanes, traffic volume on the arterial and cross street, and traffic signal timing and phasing. If the lane closure needs to be located closer to the intersection than the critical distance, lane closure should be relocated upstream of the intersection in order to prevent intersection blockage.

This topic has not been properly addressed in current research reports, and some information on arterial lane closure capacity was able to be gathered for the analysis and preparation of this research report. Lane closure guidelines were based on theoretical aspects of traffic signal operation, queueing analysis, and lane closure capacity. Mathematical computations were then performed to obtain the maximum number of vehicles queueing at a given intersection.

INTRODUCTION

The primary function of traffic signals is to assign right-of-way to traffic movements at intersecting streets or highways. These traffic signals exert a great influence on traffic flow, and have a significant impact on the vehicular movement through an intersection (1). When maintenance or construction activities are required near signals, lane closures are often necessary, and they can have a significant impact on traffic flow and lane capacity. Lane closures downstream of an intersection may cause a queue to back up and block the upstream intersection. Congestion will often occur and can impede traffic movements. This congestion will increase travel time, driver frustration, and traffic signal violations. Driveways and crossovers will be blocked, and gridlock can occur (2). Lane closure guidelines are needed to reduce the impact of this situation.

In this study, guidelines were developed for the safe and efficient implementation of lane closures near signalized intersections. It focused on pre-timed signals, as the detection capability of actuated devices is normally lost in work zones due to the fact that lane widths are reduced and lane positions shifted.

Objective

The objective of this study was to determine the critical distance between the intersection and the downstream lane closure needed to prevent intersection blockage for a given set of parameters. The parameters considered were:

1. Number of open lanes.
2. Number of closed lanes.
3. Capacity of open lanes.
4. Traffic volume on the arterial and cross-street.
5. Traffic signal timing and phasing.

If the lane closure needs to be located closer to the intersection than the critical distance, that is, lane closure distance is less than the critical distance, lane closure should be relocated upstream of the intersection in order to prevent intersection blockage.

Background

In order to develop guidelines for the solution of the proposed problem, it was necessary to review material addressing traffic signal operations, work zones, and queueing analysis.

Related literature on work zones in urban arterials was found in a research report on "Traffic Control Guidelines for Urban Arterial Work Zones" by Hawkins, Ogden, and Crowe (2). The report provides a brief description of research activities performed, and included the preliminary findings and preliminary guidelines developed during the study. A number of problem areas related to urban arterial work zones, including traffic signals, left turns, lane widths, accidents, construction activities, driver needs, and public relations, were addressed in the study. The report also emphasized the fact that current research and guidelines do not adequately address the topic.

The "Manual on Uniform Traffic Control Devices for Streets and Highways" (MUTCD) and the "Traffic Control Devices Handbook" contain standards for the use of all traffic control devices including those devices used in work zones (3, 1, 2). Part VI of the MUTCD entitled "Traffic Control for Street and Highway Construction and Maintenance Operations" establishes principles to be observed in the design, installation, and maintenance of traffic control devices, and prescribes standards where possible. These principles and standards are directed to the safe and expeditious movement of traffic through construction and maintenance zones and to the safety of the work force performing these operations (3). However, this part of the MUTCD only provides a small amount of information on work zone traffic control on urban arterials. The Handbook addresses urban arterial work zones in greater detail than the MUTCD. Diagrams of typical work zone layouts for different situations are provided and some of the major concerns are briefly mentioned (2).

Relevant information on capacity, signalized intersections, and urban and suburban arterials was found on the "Highway Capacity Manual" (4). Chapter 9 provided information on signalized intersections, and capacity of signalized intersections. No information was found in the manual for capacities on urban arterial work zones. Chapter 6 entitled "Freeway Systems" provides measured values for capacities on freeway work zones.

A report entitled "Traffic Flow Theory" provided information on the fundamentals of queueing theory (5).

DATA COLLECTION AND REDUCTION

Paving operations for a center lane on South Cooper Street in Arlington, Texas forced a lane closure on northbound traffic. The lane closure extended approximately 1100 feet and affected the signalized intersection in Sublett Street. Data was collected for AM, Noon, and PM peak hours on July 11, 1991.

Site Description

South Cooper is a four lane arterial street and one of the main roadways in Arlington. The area is surrounded by small shopping areas, fast food restaurants, and a large Hypermart. Therefore, a large amount of queueing and congestion was expected to take place during the AM, Noon and PM peak hours.

Advance road construction signs were placed on the street shoulder to inform motorists of the work zone ahead, and an advance warning arrow panel was set near the taper to indicate that traffic must merge. Eastbound traffic on Sublett was forced to merge with southbound traffic on the arterial, and many vehicles did illegal U-turn movements in the work zone to enter the northbound traffic in S. Cooper. The vehicles either continued moving downstream or turned into Sublett.

Data Collection Procedure

Data was collected using a video camera which was affixed to a 12-foot high telephone post as shown in Figure 1. During the AM peak, the camera was located approximately 500 feet from the start of the taper and traffic was observed to queue around 700 feet from start of the taper. It is assumed that this queueing was caused by a traffic signal located downstream which was still working at this time and also by construction activity taking place near taper. However, during the Noon peak, the camera was set up inside the work zone, approximately 270 feet from the end of the taper. The traffic signal was changed to a yellow/red flashing operation and a flagger helped direct traffic. This affected the queueing by providing a continuous flow of vehicles through the open lane, making random stops when cross street traffic needed to merge into arterial.

Data Reduction Procedure

A taped line was positioned in the monitor's screen as a reference, and every vehicle that passed through the taped line was counted in 5 minute intervals. The 5 minute flow rates were then grouped in 15 minute flow rates.

Data for AM peak hour was recorded from 7:41:30 to 8:25:00. Vehicle counting did not start until 7:43:30 after the camera was properly adjusted. A five minute period from 8:03:30 to 8:08:30 was discarded due to unusual traffic maneuvers where a large number of vehicles tried to enter the northbound traffic stream by cutting in through the work zone. Another factor affecting traffic activity may have been the movement of trucks into the work zone unloading concrete, thus blocking the open lane for a short period of time.

The most useful data was obtained during the Noon peak hour starting at 11:05:45 until 12:46:38. A complete 1 hr. flow rate could not be obtained, approximately 2.5 minutes of data was lost due to transfer of power source for video camera.

The PM peak was completely discarded since no queueing was present during this time, and traffic flow was continuous.

Data Analysis Procedure

All flow rates were then grouped together; the mean, median, and mode were calculated, and the lane closure capacity was estimated to be approximately 720 vph. Results are summarized in the following Tables 1, 2, 3, and 4.

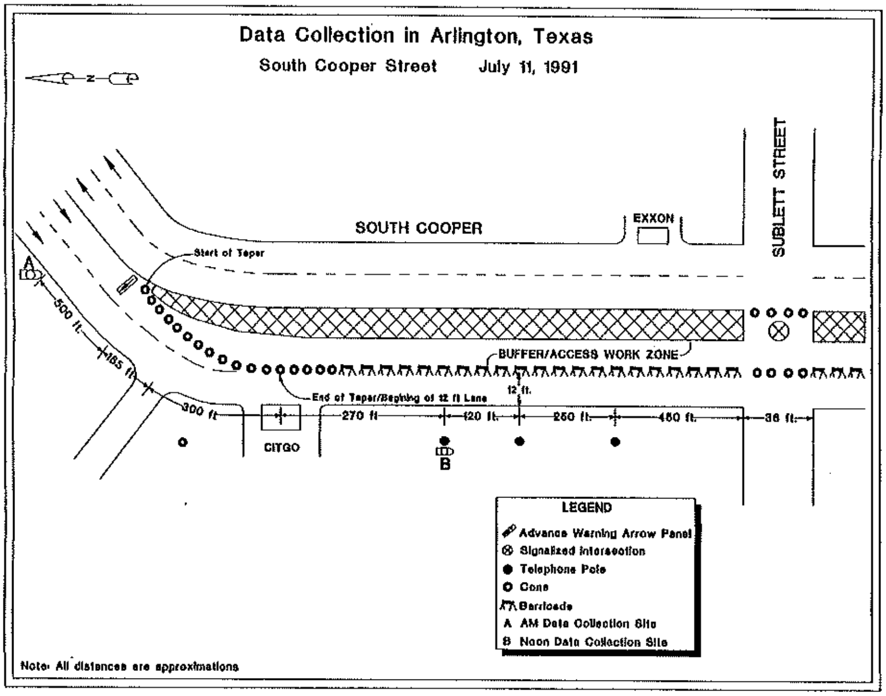

FIGURE 1. DATA COLLECTION SITE.

TABLE 1. FIVE MINUTE FLOW RATES FOR AM PEAK.

Time	No. of Vehicles
7:43:30 7:48:30	57
7:48:30 7:53:30	54
7:53:30 7:58:30	60
7:58:30 8:03:30	62
8:08:30 8:13:30	52
8:13:30 8:18:30	51
8:18:30 8:23:30	60

Factors Affecting Data Results

The following conditions at the intersection were noticed during data collection that might have affected data results:

1. Use of flashing yellow light during the Noon and PM peak instead of regular phasing.
2. Flagger guiding traffic.
3. Merging of vehicles before reaching lane closure.
4. Vehicles performing illegal U-turns inside work zone.
5. Paving operation vehicles entering or leaving work zone.
6. Vehicles entering or leaving nearby shopping centers.

STUDY DESIGN

The development of lane closure guidelines was based on theoretical aspects of traffic signal operation, queueing analysis and lane closure capacity. Mathematical calculations involving cycle length, traffic volume, effective green time, and capacity were performed to calculate the critical distance. The maximum number of vehicles that will form a queue in the intersection was estimated, and with this information, the critical distance was computed. In cases where lane closure distance is less than the critical distance, the lane closure needs to be placed upstream of the intersection.

TABLE 2. FIVE MINUTE FLOW RATES FOR NOON PEAK.

Time	No. of Vehicles
11:09:00 11:14:00	51
11:14:00 11:19:00	53
11:19:00 11:24:00	65
11:24:00 11:29:00	68
11:29:00 11:34:00	68
11:34:00 11:39:00	56
11:39:00 11:44:00	63
11:44:00 11:49:00	63
11:49:00 11:54:00	54
11:57:00 12:02:00	68
12:02:00 12:07:00	65
12:07:00 12:12:00	68
12:12:00 12:17:00	61
12:17:00 12:22:00	62
12:23:00 12:28:00	72
12:28:00 12:33:00	68
12:33:00 12:38:00	62
12:38:00 12:43:00	60

TABLE 3. FIFTEEN MINUTE FLOW RATES FOR AM PEAK.

Time	No. of Vehicles
7:43:30 7:58:30	171
8:08:30 8:23:30	163

TABLE 4. FIFTEEN MINUTE FLOW RATES FOR NOON PEAK.

Time	No. of Vehicles
11:09:00 11:24:00	169
11:24:00 11:39:00	192
11:39:00 11:54:00	180
11:57:00 12:12:00	201
12:28:00 12:43:00	190

Study Variables

In order to develop guidelines for the solution of the proposed problem, a set of conditions at the signalized intersection had to be established. The guidelines involve the following parameters:

1. The volume and distribution of traffic movements.
2. Traffic composition.
3. Geometric characteristics of the intersection.
4. Intersection signalization.

Major street approach volumes analyzed were 300, 500 and 700 vph. The minor street volume for each way was determined to be 100 and 300 vph with 30 percent right-turn/left-turn turning movements; and 500 and 700 vph with 12 percent left-turn/right-turn turning movements. These average turning proportions were obtained from two sources: "Estimation of Intersection Turning Movements from Approach Counts" and "Estimation of Turning Flows from

Automatic Counts" (6, 7). Saturation capacity for arterial was set at 1700 vphpl, and saturation capacity for minor street was obtained by multiplying the saturation capacity by the LT, RT turning proportion. Lane closure capacity was based on the value obtained in study site, and also from the "Highway Capacity Manual," though this information is only related to work zone capacities on freeways (4).

Traffic composition is addressed as vehicles or passenger cars. Information provided by S. Ross Blanchard on his research report entitled "A Study of Frontage Road Queueing, Vehicle Spacing and Applications to Freeway Exit Ramp Design Criteria" determined passenger car spacing to be 23.5 feet, front bumper to front bumper (8). For this research, a spacing of 24 feet is used.

The geometric characteristics of the intersection analyzed in this report were the following:

Lanes in Major Street	Open Lanes	Closed Lanes
2	1	1
3	2	1
3	1	2

Signal timing for intersection involved 60, 90, and 120 seconds cycles with cycle splits of 50, 67 and 75 percent of the green time on major street. The signal phasing was 2-phase, with unprotected left turns on minor street.

Study Limitations

Conditions at intersection were limited to the following characteristics:

1. Four-legged intersection.
2. Pre-timed control.
3. Work zone located downstream of signalized intersection.
4. Directional distribution.

Study Assumptions

The conditions present at signalized intersection included the following:

1. Twelve-foot lane widths.
2. Level grade.
3. No curb parking on the intersection approaches.
4. All passenger cars in the traffic stream, including no buses stopping within the intersection.
5. Intersection located in a non-CBD area.
6. No pedestrians or bikes on intersection.
7. No start-up or clearance lost time.
8. Random arrival of vehicles at intersection.
9. No travel time to work zone.
10. No work zone length given.

DATA ANALYSIS

A set of formulas was needed for the mathematical calculation of the maximum amount of vehicles that will queue in the intersection. The formulas developed are derived from simple relationships between capacity and traffic signal operation.

Lane Closure Guidelines

The following calculations were performed to obtain the queue at the work zone:

1. Calculate effective red and green times.

$$g = C * p \tag{1}$$
$$r = C - g \tag{2}$$

where:

g = effective green time, in seconds
C = cycle length, in seconds
p = percentage of green time on major street
r = effective red time, in seconds

2. Calculate maximum number of cars that will queue in the arterial during red signal

$$Q_{MS} = V_{MS} * r \tag{3}$$

where:

Q_{MS} = number of vehicles queued in intersection during effective red time
V = volume in major street, in vehicles per second

3. Determine the saturation green time for arterial street.

$$G_s = (V_{MS} * r)/s/(1 - (V_{MS} * r)/s) \tag{4}$$

where:

G_s = saturation green time, in seconds
s = saturation flow rate for arterial street, in vehicles per second

4. Determine work zone queue at lane closure.

$$Q_{wz}^* = (s * G_s) - (c * G_s) \tag{5}$$

$$Q_{wz} = Q_{wz}^* + V * (g - G_s) - c * (g - G_s) \tag{6}$$

where:

c = lane closure capacity

Q_{wz}^* = queue at work zone during saturation flow rate
Q_{wz} = queue at work zone during unsaturated flow rate

5. Calculate number of cars that will form on minor street during red signal.

$$V_{RIGHT} = V_{ms} * \% \quad (7)$$

$$V_{LEFT} = V_{ms} * \% \quad (8)$$

$$Q_{ms} = (V_{RIGHT} + V_{LEFT}) * r \quad (9)$$

where:

V_{LEFT} = left-turn minor street volume
V_{RIGHT} = right-turn minor street volume
% = percentage of left or right turning movement
Q_{ms} = queued vehicles in minor street

6. Determine the saturation green time for minor street using the formula in step 3.
7. Calculate work zone queue at lane closure using formulas on step 4.
8. Use Q_{MAX} to calculate critical distance.

$$\text{Critical Distance} = Q_{MAX} * \text{vehicle length} \quad (10)$$

Spreadsheet Calculations

A spreadsheet as shown in Table 5 was then set up to calculate all these values automatically. This spreadsheet calculated a complete cycle length in five second intervals. Other information provided by spreadsheet table included input/output on major street, minor street, and lane closure.

TABLE 5. WORKING SPREADSHEET.

Number of lanes:	2		Number of closed lanes:	1	
Number of phases:	2		Time cycle:	90 sec	
Lane closure capacity:	720 vphpl	0.2vps	Cycle split:	67 %	
			Effective green time:	60 sec	
			Effective red time:	30 sec	
Arterial street volume:	300 vph	0.083 vps	Minor street volume:	500 vph	0.139 vps
Saturated flow:	1700 vphpl	0.944 vps	Saturated flow:	204 vphpl	0.113 vps
Saturated green:	2.90 sec		Saturated green:	25.00 sec	

Time	Elapsed Time	Arterial Street Ind.	Arterial Street Input	Arterial Street Output	Arterial Street Q	Minor Street Ind.	Minor Street Input	Minor Street Output	Minor Street Q	Lane Closure Input	Lane Closure Output	Lane Closure Q
0-5	5	R	0.08	0.00	0.42	G	0.0333	0.113	1.60	0.113	0.20	0.00
6-10	10	R	0.08	0.00	0.83	G	0.0333	0.113	1.20	0.113	0.20	0.00
11-15	15	R	0.08	0.00	1.25	G	0.0333	0.113	0.80	0.113	0.20	0.00
16-20	20	R	0.08	0.00	1.67	G	0.0333	0.113	0.40	0.113	0.20	0.00
21-24.99	24.99	R	0.08	0.00	2.08	G	0.0333	0.113	0.00	0.113	0.20	0.00
25	25	R	0.08	0.00	2.08	G	0.0333	0.113	0.00	0.113	0.20	0.00
26-30	30	R	0.08	0.00	2.50	G	0.0333	0.033	0.00	0.033	0.20	0.00
31-32.9	32.9	G	0.08	0.94	0.00	R	0.0333	0.000	0.10	0.944	0.20	2.16
33-35	35	G	0.08	0.08	0.00	R	0.0333	0.000	0.17	0.083	0.20	1.91
36-40	40	G	0.08	0.08	0.00	R	0.0333	0.000	0.33	0.083	0.20	1.33
41-45	45	G	0.08	0.08	0.00	R	0.0333	0.000	0.50	0.083	0.20	0.75
46-50	50	G	0.08	0.08	0.00	R	0.0333	0.000	0.67	0.083	0.20	0.16
51-55	55	G	0.08	0.08	0.00	R	0.0333	0.000	0.83	0.083	0.20	0.00
56-60	60	G	0.08	0.08	0.00	R	0.0333	0.000	1.00	0.083	0.20	0.00
61-65	65	G	0.08	0.08	0.00	R	0.0333	0.000	1.17	0.083	0.20	0.00
66-70	70	G	0.08	0.08	0.00	R	0.0333	0.000	1.33	0.083	0.20	0.00
71-75	75	G	0.08	0.08	0.00	R	0.0333	0.000	1.50	0.083	0.20	0.00
76-80	80	G	0.08	0.08	0.00	R	0.0333	0.000	1.67	0.083	0.20	0.00
81-85	85	G	0.08	0.08	0.00	R	0.0333	0.000	1.83	0.083	0.20	0.00
86-90	90	G	0.08	0.08	0.00	R	0.0333	0.000	2.00	0.083	0.20	0.00

RESULTS

Results for the critical distances are summarized in Tables 6, 7, and 8. The table set up is subdivided in two major categories: one indicating the major street volume, and the other, the minor street volume. The cycle length is indicated with the percentage amount of green time on arterial street, and the critical distance can easily be obtained by matching the different characteristics of the intersection.

The results of this research involved three different scenarios:

1. Two lanes in one direction with one lane closed.
2. Three lanes in one direction with one lane closed.
3. Three lanes in one direction with two lanes closed.

Each scenario result will be briefly described in the following paragraphs.

Scenario 1

Two lanes in one direction with one lane closed - The critical distances recorded in Table 6 indicate a correlation between the arterial street volumes and the critical distance. All critical distances turned out to be the same although minor street volumes were different. This indicates that the traffic volume in the minor street was probably pretty low and turning movements into the arterial did not considerably affect traffic flow through the lane closure. This also means that the major street volume is a critical factor in the calculation of the maximum number of vehicles that queued in the intersection. Volumes greater than 500 vph on a major street caused the intersection to oversaturate, thus data obtained was discarded. During oversaturation, the queue will never dissipate, and it will continue to get longer. In this particular scenario, oversaturation occurs in all 75 percent cycle splits with the minor street volume of 500 vph, and also on the 67 percent and 75 percent cycle splits with the minor street volume of 700 vph. These calculations were also discarded since in these cases saturated green time exceeded green time on the minor street.

Scenario 2

Three lanes in one direction with one lane closed - The same observations as in scenario 1 are found in this scenario with a few exceptions. Effective green time oversaturation only occurs on the 75 percent cycle split of the 700 vph minor street volume, and oversaturation on 700 vph major street volume does not occur.

Scenario 3

Three lanes in one direction with two lanes closed - As in Scenario 2, oversaturation of green time occurs in the 75 percent cycle split of the minor street volume of 700 vph. Oversaturation occurs in both minor street volumes of 300 and 700 vph with a major street volume of 500 vph. A notable difference is found in the critical distances of the 75 percent cycle split for all the different cycle lengths; this is primarily due to the fact that the maximum queue occurs on the minor street. Oversaturation occurs all through the 700 vph major street analysis, so they are not presented in the table. This indicates that volumes in a major street larger than 500 vph will create congestion in the intersection, and even a bottleneck if other signalized intersections are located in the nearby area.

CONCLUSIONS AND RECOMMENDATIONS

At this time, four major conclusions can be drawn from this research study:

1. All critical distances for a particular cycle split and cycle length turned out to be the same. This is primarily due to the assumption that the maximum queue takes place during major street volume flow through work zone. Apparently the amount of vehicles turning from the minor street into the arterial street was small enough that queue dissipated quickly, and did not conflict with vehicles moving on the major street. Still, some exceptions occurred where the maximum queue was found on the minor street. It can be noticed that this situation only happened during a specified cycle split and when major and minor street volumes were the same.

2. Maximum amount of queueing usually took place during saturation flow rate. From this conclusion, the maximum number of vehicles queueing at the work zone was easily estimated. With the spreadsheet table set at five second intervals, the time when the maximum queue occurred was determined, and also the time it took to dissipate.

3. Critical distances obtained in the 75 percent cycle split are approximately half of the critical distances found in the 50 percent cycle split. A relationship exists between these two values because the amount of effective red time on the arterial street in the 75 percent cycle split is half of the effective red time in the 50 percent cycle split on the arterial street. Therefore, the saturated green time in the 75 percent cycle split will also be half of the one in the 50 percent cycle split.

4. From data gathered in Arlington, Texas for the estimation of a lane closure capacity for a two-lane/one closed arterial, the result turned out to be approximately 54 percent of the value given in the "Highway Capacity Manual" for lane closures in freeways with the same conditions (4). For the

TABLE 6. CRITICAL DISTANCES IN FEET FOR LANE CLOSURES IN ARTERIAL STREETS TWO LANES IN ONE DIRECTION WITH ONE LANE CLOSED.

	MAJOR STREET VOLUME																	
	300 VPH Cycle Lengths									500 VPH Cycle Lengths								
	60 Sec			90 Sec			120 Sec			60 Sec			90 Sec			120 Sec		
Minor Street Volume	% of Green Time			% of Green Time			% of Green Time			% of Green Time			% of Green Time			% of Green Time		
	50%	67%	75%	50%	67%	75%	50%	67%	75%	50%	67%	75%	50%	67%	75%	50%	67%	75%
100 VPH	52	35	26	78	52	39	104	69	52	92	62	46	139	92	69	185	123	92
300 VPH	52	35	26	78	52	39	104	69	52	92	62	46	139	92	69	185	123	92
500 VPH	52	35	**	78	52	**	104	69	**	92	62	**	139	92	**	185	123	**
700 VPH	52	**	**	78	**	**	104	**	**	92	**	**	139	**	**	185	**	**

** Indicates effective green time oversaturation.

NOTE: If work zone needs to be placed a distance closer than that given in the table, lane closure needs to be relocated upstream of the intersection.

TABLE 7. CRITICAL DISTANCES IN FEET FOR LANE CLOSURES IN ARTERIAL STREETS THREE LANES IN ONE DIRECTION WITH ONE LANE CLOSED.

Minor Street Volume	MAJOR STREET VOLUME																										
	300 VPH Cycle Lengths									500 VPH Cycle Lengths									700 VPH Cycle Lengths								
	60 Sec			90 Sec			120 Sec			60 Sec			90 Sec			120 Sec			60 Sec			90 Sec			120 Sec		
	% of Green Time			% of Green Time			% of Green Time			% of Green Time			% of Green Time			% of Green Time			% of Green Time			% of Green Time			% of Green Time		
	50%	67%	75%	50%	67%	75%	50%	67%	75%	50%	67%	75%	50%	67%	75%	50%	67%	75%	50%	67%	75%	50%	67%	75%	50%	67%	75%
100 VPH	43	29	22	65	43	33	87	58	43	76	50	38	114	76	57	152	101	76	111	74	56	167	111	83	222	148	111
300 VPH	43	29	22	65	43	33	87	58	43	76	50	38	114	76	57	152	101	76	111	74	56	167	111	83	222	148	111
500 VPH	43	29	22	65	43	33	87	58	43	76	50	38	114	76	57	152	101	76	111	74	56	167	111	83	222	148	111
700 VPH	43	29	**	65	43	**	87	58	**	76	50	**	114	76	**	152	101	**	111	74	**	167	111	**	222	148	**

** Indicates effective green time oversaturation.
- - Indicates lane closure oversaturation.
NOTE: If work zone needs to be placed a distance closer than that given in the table, lane closure needs to be relocated upstream of the intersection.

TABLE 8. CRITICAL DISTANCES IN FEET FOR LANE CLOSURES IN ARTERIAL STREETS THREE LANES IN ONE DIRECTION WITH TWO LANES CLOSED.

	MAJOR STREET VOLUME																	
	300 VPH Cycle Lengths									500 VPH Cycle Lengths								
	60 Sec			90 Sec			120 Sec			60 Sec			90 Sec			120 Sec		
Minor Street Volume	% of Green Time			% of Green Time			% of Green Time			% of Green Time			% of Green Time			% of Green Time		
	50%	67%	75%	50%	67%	75%	50%	67%	75%	50%	67%	75%	50%	67%	75%	50%	67%	75%
100 VPH	56	37	28	84	56	42	112	74	56	97	65	48	146	97	73	194	130	97
300 VPH	56	37	36	84	56	54	112	74	73	--	--	--	--	--	--	--	--	--
500 VPH	56	37	28	84	56	42	112	74	56	97	65	48	146	97	73	194	130	97
700 VPH	56	37	**	84	56	**	112	74	**	--	--	--	--	--	--	--	--	--

** Indicates effective green time oversaturation.
-- Indicates lane closure oversaturation.
NOTE: If work zone needs to be placed a distance closer than that given in the table, lane closure needs to be relocated upstream of the intersection.

analysis of the other scenarios in this report, values obtained from the manual were multiplied by 54 percent. This indicates that lane closure capacity for work zones in arterial streets is considerably lower than the ones obtained for freeways. More data collection will be necessary to support this assumption.

Due to the short duration of this research study, much more research can be done with this topic. During this study, many variables were not taken into consideration such as travel time, lost time, parking around area, etc. due to time constraints, and the limited knowledge of the researcher on this subject. There is also a need for more documentation on the topic of work zones on arterial streets, because with this information, more detailed guidelines for the calculation of critical distances in arterial lane closures could be developed.

More information on lane closure capacities in arterial work zones is also needed. More data should be collected, and analyzed in the different scenarios to provide accurate estimates of lane closure capacities as those given for freeways.

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An Examination of the Indicators of Congestion Level

SHAWN M. TURNER

This research report examines the relationships between many possible indicators and congestion level in an attempt to identify and/or validate indicators for area-wide congestion measurement and mobility analysis purposes. The study estimates the congestion level for 50 large and medium United States urban areas with three congestion measures. The estimated congestion level is then compared to indicators composed of travel, facility supply, and urban area variables to determine direct relationships.

Two of the congestion measures used in this study, the roadway congestion index and the congestion severity index, produced results with a high correlation to each other. The lane-mile duration index suffered from unavailable data for several urban areas, and was consequently less comparable among the other two measures.

It was determined that two indicators relating to travel intensity -- daily vehicle-miles of travel per lane-mile and daily vehicle-miles of travel per square mile -- had lost the closest correlation to level of congestion. Average daily freeway traffic per hourly capacity, previously used as a facility measure, was also identified as having a close relationship to area-wide congestion level.

INTRODUCTION

Over the past decade, traffic congestion in urban and suburban areas has grown from a mere annoyance to a severe problem. Although traffic congestion is not a new problem for residents of the central city, it has spread and intensified to envelop the urban fringe and outlying suburban areas. Increasing traffic congestion and decreasing urban mobility has become a major concern of transportation professionals nationwide. Current predictions offer no relief either. Freeway delay has been projected to increase from between 300 percent to 500 percent by the year 2005.

There are several factors that have contributed to the rapid growth of traffic congestion in the United States in the past decade. The number of registered vehicles has increased disproportionately to population and household growth; consequently, vehicle travel spiraled to over two trillion vehicle-miles by the late 1980's. To compound the increase in travel, construction of new highway facilities has slowed considerably since the near completion of the Interstate System in the early 1970's. A higher percentage of commuters now drive as opposed to using public transit or walking because of increased access to the automobile and the suburban migration of both business and residential properties. This change in commuting patterns has, in turn, clogged local street networks and highway facilities.

Congestion Measures

The type of measure used to quantify the level of congestion on a transportation system should deliver comparable results for various systems with similar congestion levels. These measures should accurately reflect the quality of service for any type of system, whether it be a single facility or an entire urban area. A congestion measure should also be simple, well-defined, and easily understood and interpreted among various users and audiences.

Existing congestion measures use assorted variables in equation formats to describe the extent, severity and/or duration of congestion. One type of measure uses indicators, or variables related to the level of congestion, to quantify congestion. Examples of possible indicators would include travel, roadway supply, and the travel-to-roadway supply ratio. Indicators are generally related to the probable causes of congestion. Another type of measure uses variables which are descriptors of the effects of congestion. Vehicle delay, congestion duration, and average travel speed are all examples of variables which characterize the effects of congestion.

This report examines the relationships between many possible indicators and congestion level in an attempt to identify and/or validate indicators for congestion measurement and mobility analysis purposes. The indicators used in this study were those composed of travel, facility supply, urban area variables, or any combination thereof.

Background

There have been several efforts in recent years to improve the analysis of traffic congestion data. Many of the efforts concentrated on developing an accurate area-wide measure of congestion. However, there has been no clear consensus on which indicator, if any among those currently used, most directly reflects congestion level in urban areas. Additionally, many possible indicators have not been fully examined with regard to their relationship to congestion. The following paragraphs discuss previous research regarding indicators and measures of congestion.

The level-of-service (LOS) concept, as adopted by the "1985 Highway Capacity Manual," represents a range of operating conditions (1). The LOS, or quality of service, of a facility is determined by traffic characteristics like vehicle density and volume-to-capacity (v/c) ratio, depending on the facility type. Most congested traffic conditions fall into the LOS F range, a range "used to define forced or breakdown flow." It is in this LOS range that the demand of traffic exceeds the capacity of the roadway. Since the v/c ratio theoretically can not exceed 1 (volume can not exceed capacity), past a certain level of demand, this ratio and the LOS concept is of little use in distinguishing between levels of congested flow in LOS F. Forced flow conditions in LOS F may reduce traffic volumes and subsequently lower the v/c ratio, making the flow conditions appear to be less than capacity. While dense travel corridors now experience many hours of the daily "rush hour", the v/c ratio has been traditionally used to describe a single peak-hour condition.

In an attempt to better describe the duration dimension of congestion, the California Department of Transportation uses the number of hours of LOS F service (2). For example, LOS F2 represents two hours of LOS F service. This combination of the v/c ratio and the duration of congested operation enhances the LOS concept and accounts for the "peak spreading" common in many urban areas which extends the peak period over several hours. This improved measure is relatively easy to calculate, interpret and communicate, but has only been used in planning analyses by the California Department of Transportation.

An analysis technique developed by Lindley used Highway Performance Monitoring System (HPMS) data, traffic distribution patterns, and "Highway Capacity Manual" calculations to determine freeway travel delay (3). A congestion severity index was defined as the total freeway delay (vehicle-hours) per million vehicle-miles of travel. Urban area freeway systems were then ranked according to the congestion severity index. A methodology was developed to include the delay caused by incidents using an accident database of breakdown types and rates. Delay on principal arterial streets was not included in this analysis. In Lindley's calculations, the congestion threshold was defined at a v/c ratio of 0.77 (LOS D or lower) during one or more hours per day. This definition of the beginning of congestion is consistent with values reported to Congress on the status of national highways by the Department of Transportation and values recommended for urban freeway design standards by AASHTO (4, 5).

Early research by Lomax and Christiansen investigated the use of several variables as indicators of urban area mobility (6). Among those presented as possible indicators were traffic per lane, percentage of congested freeway, k-factor, and peak-hour travel distance. Trends in these possible indicators were calculated for 1975 to 1980 for five urban areas in Texas. The study concluded that VMT per lane was perhaps the most reliable indicator, and developed a "congestion standard" that combined weighted values for freeway and principal arterial street VMT per lane.

Subsequent research by Lomax, et. al. resulted in the development of a roadway congestion index (RCI) (7, 8, 9, 10). The indicator of daily vehicle-miles of travel (DVMT) per lane-mile for both freeways and principal arterial streets is weighted and normalized in the index's equation. Major U.S. urban areas are then ranked according to the RCI value. The threshold of congestion was chosen at a v/c ratio of 0.77 (LOS D or lower) and was correlated to ADT per lane values for freeways and principal arterial streets through basic assumptions about traffic characteristics.

In an as-of-yet unpublished report by Cottrell, a lane-mile duration index ($LMDI_F$) is presented as a measure of recurring freeway congestion in urbanized areas (11). The $LMDI_F$ represents a summation for an urban area of the congested freeway lane-miles multiplied by the respective duration of LOS F service (similar to Caltrans's reporting of LOS F1, F2, etc.). Traffic distribution patterns in an HPMS technical manual were used to relate the value of average daily traffic per hourly capacity (AADT/C) to congestion duration (12). Previous research by Lisco determined that peak-period delay occurred where the AADT/C value reached 8 to 10 (13). In his analysis, Cottrell chose LOS F as the congestion threshold and correlated that to an AADT/C value of 9. The analysis excluded arterial streets and did not consider the effects of incident delay.

As evidenced by the above, no clear consensus exists on where congestion begins or which indicator or measure most accurately reflects congestion level on an area-wide basis.

METHODOLOGY

This study attempted to identify congestion indicators for congestion measurement and mobility analysis purposes. An indicator is a variable directly related to the level of congestion. The study used several congestion measures to estimate the congestion level in 50 large and medium U.S. urban areas. Possible indicators of congestion were chosen, and the relationships between the indicators and congestion level were examined for the year 1989. The study methodology is described in detail on the following page.

Use of an Existing TTI Database

Significant research efforts at the Texas Transportation Institute (TTI) have compiled an extensive database of congestion-related statistics. The database currently contains annual summary statistics from 1982 through 1989 for 50 large and medium (population generally greater than 500,00) U.S. urban areas. These urban areas may be found in Table 1. The database statistics of interest to this study were those relating to travel, facility supply, and urban area characteristics. The majority of possible indicators to be examined were composed of one or a combination of two of these three basic types of variables related to congestion. These possible indicators and other data needed to calculate congestion measures were extracted from the existing database.

Estimation of the Level of Congestion

In order to examine the relationships between the various possible indicators and congestion level, a representation of relative congestion levels for each urban area had to be attained. This was accomplished by choosing several congestion measures currently in use. The congestion levels calculated from these measures were then compared to ensure similar results among the measures. The choice of several comparable measures also prevented bias towards any particular indicator. The congestion measures were chosen with consideration given to previous results, data availability, and ease of interpretation. Each of the three measures chosen for this study are described below.

Roadway Congestion Index - The roadway congestion index (RCI) was initially developed by Lomax and others at TTI to study mobility trends in major Texas cities. The RCI analysis was gradually expanded to 50 urban areas throughout the U.S. Urban areas in this analysis are consistent with the boundaries as defined by the U.S. Census Bureau. The major source of data for the calculation of the RCI comes from the Highway Performance Monitoring System (HPMS) database. This database is supplemented with information collected from local Metropolitan Planning Organizations (MPOs), state Departments of Transportation (DOTs) cities, counties, and other regional agencies for each area.

In calculation of the RCI it is presumed that delay, and consequently congestion, begins to occur at level-of-service (LOS) D, corresponding to a v/c ratio of 0.77 (1). This was determined to be approximately equivalent to 15,000 vehicles per lane per day on freeways, and 5,750 vehicles per lane per day on principal arterial streets. On an area-wide basis where averages over many facilities may be misleading, it was determined that lower values were more appropriate. The values of 13,000 vehicles per lane per day for freeways and 5,000 vehicles per lane per day for principal arterial streets were then used on an area basis for the congestion threshold. In the RCI equation, the daily vehicle-miles of travel (DVMT) per lane-mile for freeways and principal arterial streets are weighted by the respective amount of DVMT for each urban area. The congestion levels are then normalized (using the 13,000 value for

TABLE 1. STUDY CITIES - 50 LARGE AND MEDIUM U.S. URBAN AREAS.

Alburquerque, NM	Atlanta, GA	Austin, TX	Baltimore, MD
Boston, MA	Charlotte, NC	Chicago, IL	Cincinnati, OH
Cleveland, OH	Columbus, OH	Corpus Christi, TX	Dallas, TX
Denver, CO	Detroit, MI	El Paso, TX	Fort Worth, TX
Ft. Lauderdale, FL	Hartford, CT	Honolulu, HI	Houston, TX
Indianapolis, IN	Jacksonville, FL	Kansas City, MO	Los Angeles, CA
Louisville, KY	Memphis, TN	Miami, FL	Milwaukee, WI
Minn.-St. Paul, MN	Nashville, TN	New Orleans, LA	New York, NY
Norfolk, VA	Oklahoma City, OK	Orlando, FL	Philadelphia, PA
Pheonix, AZ	Pittsburgh, PA	Portland, OR	Sacramento, CA
Salt Lake City, UT	San Antonio, TX	San Bern.-Riv., CA	San Diego, CA
San Fran.-Oak., CA	San Jose, CA	Seattle-Everett, WA	St. Louis, MO
Tampa, FL	Washington, DC		

freeways and the 5,000 value for principal arterial streets) with an RCI greater than 1.0 representing undesirable area-wide congestion. The RCI equation for each urban area is illustrated below in Equation 1.

Congestion Severity Index - The congestion severity index (CSI) was originally developed by Lindley as a measure of freeway delay per million vehicle-miles of travel (3). The measure was modified for this study to include principal arterial street delay, as it was felt that this functional class makes substantial contributions to area mobility. Delay for both freeways and principal arterial streets was calculated by determining average travel speeds and comparing those to desirable travel speeds. The delay values used in this study were obtained from the TTI congestion database. In combining the delay for the two different functional classes, it was felt that delay on freeways and principal arterial streets was roughly equivalent; consequently, the delay values were not weighted with respect to functional class. The CSI equation for each urban area is illustrated below in Equation 2.

Lane-Mile Duration Index - The lane-mile duration index, LMDIF, was recently developed by Cottrell as a measure of recurring freeway congestion in urban areas. The analysis technique used the HPMS database to calculate an average daily traffic per hourly capacity (AADT/C) value for each urban area freeway segment. This AADT/C value was then related to a congested percentage of average daily traffic by utilizing traffic distribution patterns in an HPMS technical manual (12). The congestion duration is the product of the AADT/C value and the congested percentage of average daily traffic. The LMDIF for each urban area, then, is the summation of the product of congested lane-miles and congestion duration for all area freeway segments. Cottrell's methodology was used to calculate LMDIF values, and Equation 3 below applied for each urban area (11).

Choosing Possible Congestion Indicators

It has been generalized that congestion is related to three basic types of variables: travel (e.g. vehicle-miles of travel), supply (e.g. lane-miles of roadway), and urban area characteristics (e.g. population density) (14). The indicators chosen for this study were composed of one or a combination of two of these three basic types of variables. The indicators were chosen with consideration given to data availability, intuitive relation to the causes of congestion, and logical results. With the exception of one, all indicators were extracted or calculated with data from the existing TTI congestion database. The indicators that were examined in this study are as follows:

1. *Travel* - daily vehicle-miles of travel (DVMT); transit trips; passenger-miles of travel.
2. *Supply* - lane-miles; transit revenue miles.
3. *Urban Area Characteristics* - population size; population density; registered vehicles; registered vehicles per square mile; registered vehicles per capita.
4. *Travel-Supply* - DVMT per lane-mile; freeway average daily traffic per hourly capacity (AADT/C).
5. *Travel-Urban Area Characteristics* - DVMT per square mile; DVMT per registered vehicle; DVMT per capita; transit trips per capita; passenger-miles per capita.
6. *Supply-Urban Area Characteristics* - lane-miles per capita; lane-miles per square mile; registered vehicles per lane-mile; revenue miles per capita.

The transit indicators were totals for bus and heavy, light, and commuter rail. The other indicators, with the exception of freeway AADT/C, were calculated for both freeways and principal arterial streets.

Examination of Congestion Relationships

There were two basic steps in the examination of the congestion relationships. The first was a graphical comparison of all possible indicators to the three congestion measures. Each indicator for freeways, principal arterial streets,

$$\mathrm{RCI} = \frac{[\text{Freeway DVMT/Ln-Mi} * \text{Freeway DVMT}] + [\text{Prin. Art. DVMT/Ln-Mi} * \text{Prin. Art. DVMT}]}{[13{,}000 * \text{Freeway DVMT}] + [5{,}000 * \text{Prin. Art. DVMT}]} \quad (1)$$

$$\mathrm{CSI} = \frac{\text{Total Freeway Delay (veh-hrs)}}{\text{Freeway VMT (million)}} + \frac{\text{Total Prin. Art. Delay (veh-hrs)}}{\text{Prin. Art. VMT (million)}} \quad (2)$$

$$\mathrm{LMDI_F} = \sum_{i=1}^{m} \text{Congested Lane-Miles}_i * \text{Congestion Duration}_i \text{ (hours)} \quad (3)$$

and the total of both, were graphed against each congestion measure, with each urban area representing a data point on a "scatter-plot." Each graph was then inspected for variability of data points and ease of constructing a best-fit line, whether it be linear or exponential. This graphical comparison was by no means precise, but did give a sense of the relationship, if any, between the indicator and the estimated congestion level.

The second step in examination of the congestion relationships was a limited regression analysis. This analysis determined the coefficient of determination, R^2, a statistic which represents the proportion of variability that is accounted for in a relationship. In general, an R^2 value of 0.5 or greater was interpreted as a close relationship. It is recognized that an R^2 value is not a statistically complete treatment, but it was felt that the use of a graphical comparison combined with the R^2 value provided enough information to infer whether some sort of a relation existed.

RESULTS

Estimation of Congestion Level

The congestion level was estimated using the three measures whose equations were presented in the methodology section of this paper. The measures were calculated for the 50 urban areas for 1989, the most recent year for which data was available. The HPMS database composition prevented determination of LMDIF for nineteen urban areas in the following states: California, Connecticut, Florida, Hawaii, Michigan, Ohio (Cleveland only), Oregon, and Washington. In the past, HPMS reporting procedures did not require states to report traffic data for each urban area individually; consequently, the above states chose to submit traffic data with several urban areas "grouped" into one data set. The use of several transportation agencies' data by TTI in the development of their congestion database prevented similar deficiencies in calculation of the roadway congestion index and the congestion severity index. A summary of the measures and ranking for the 50 urban areas in this study for 1989 may be found in Table 2.

An important criterion for the measures used to estimate congestion level was comparability of results. Upon comparing the measures, it was discovered that the roadway congestion index and the congestion severity index were closely related, with an R^2 value of 0.72. Both of these measures include freeways and principal arterial streets, but the roadway congestion index uses a travel-to-supply ratio while the congestion severity index uses a delay-to-travel ratio. In a graphical comparison (see Figure 1), an RCI value of 1.0 was related to a CSI value of 24,000 (vehicle hours per million vehicle-miles of travel) by means of a calculated regression line. The relationship between these two measures and the lane-mile duration index was less distinguished but nonetheless comparable (R^2 values of 0.60 and 0.45 for the CSI and the RCI, respectively).

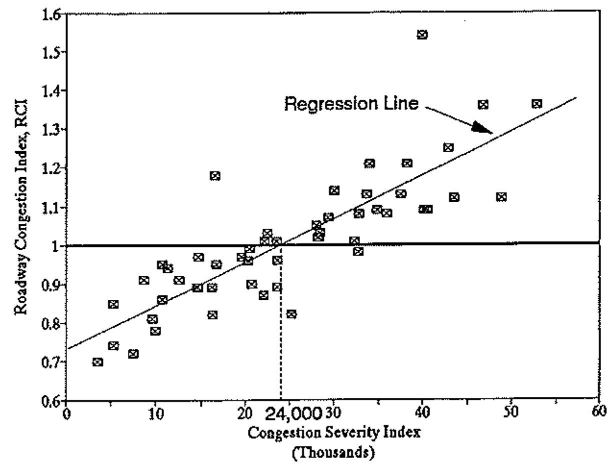

FIGURE 1. CORRELATION OF ROADWAY CONGESTION INDEX AND CONGESTION SEVERITY INDEX.

Examination of Congestion Relationships

As described earlier, there were two steps in examination of the congestion relationships: a graphical comparison and determination of R^2. The results of these two steps will be presented for the relationships with the highest correlation.

The indicator of daily vehicle-miles of travel (DVMT) per lane-mile, a travel-to-supply ratio, was found to have the highest correlation among all other indicators. Because the roadway congestion index uses DVMT per lane-mile in a weighted, normalized equation, it was excluded from this comparison. Also, because of the difficulty of combining this indicator for freeways and principal arterial streets without replicating the roadway congestion index, it was analyzed separately for the two different functional classes. A plot of freeway DVMT per lane-mile versus congestion level is presented in Figure 2. The R^2 values for this relationship were 0.68 for the CSI and 0.45 for the LMDI. The relationship between arterial DVMT per lane-mile and the three congestion measures was less pronounced, but did give R^2 values between 0.35 and 0.45.

The indicator with the next highest correlation to congestion level was daily vehicle-miles of travel (DVMT) per square mile of urban area. Travel (DVMT) was combined for both freeways and principal arterial streets in this examination. It was found that the roadway congestion index and the congestion severity index were most closely related to this indicator (Figure 3). The R^2 values for this relationship were 0.48 for the RCI, but only 0.27 for the CSI. It was noted throughout the examination that R^2 values were consistently higher for RCI-indicator relationships than for CSI- or LMDI-indicator relationships.

TABLE 2. SUMMARY OF CONGESTION MEASURES FOR 50 URBAN AREAS, 1989.

	Roadway Congestion Index		Congestion Severity Index		Lane-Mile Duration Index	
Urban Areas	Value	Rank	Value	Rank	Value	Rank
Los Angeles, CA	1.54	1	39,938	8	-	-
San Fran-Oak, CA	1.36	2	46,817	3	-	-
Washington, DC	1.36	2	52,882	1	2773	3
Miami, FL	1.25	4	42,930	5	-	-
Chicago, IL	1.21	5	34,015	13	1820	4
Seattle-Everett, WA	1.21	5	38,215	9	-	-
San Diego, CA	1.18	7	16,472	35	-	-
Atlanta, GA	1.14	8	30,044	17	1813	5
Houston, TX	1.13	9	37,612	10	3392	2
New Orleans, LA	1.13	9	33,616	14	95	21
New York , NY	1.12	11	48,924	2	4381	1
San Jose, CA	1.12	11	43,494	4	-	-
Boston, MA	1.09	13	40,551	6	999	6
Honolulu, HI	1.09	13	34,885	12	-	-
San Bernardino-Riv, CA	1.09	13	40,197	7	-	-
Detroit, MI	1.08	16	35,923	11	-	-
Norfolk, VA	1.08	16	32,788	15	469	9
Portland, OR	1.07	18	29,406	18	-	-
Philadelphia, PA	1.05	19	28,011	22	347	13
Phoenix, AZ	1.03	20	28,482	19	446	10
Tampa, FL	1.03	20	22,472	27	-	-
Charlotte, NC	1.02	22	28,151	21	-	-
Dallas, TX	1.02	22	28,234	20	887	7
Denver CO	1.01	24	23,546	26	212	15
Sacramento, CA	1.01	24	22,107	28	-	-
Baltimore, MD	0.99	26	20,442	31	545	8
Orlando, FL	0.98	27	32,731	16	-	-
Jacksonville, FL	0.97	28	19,538	33	-	-
Milwaukee, WI	0.97	28	14,694	38	125	19
Austin, TX	0.96	30	23,632	25	435	11
St. Louis, MO	0.96	30	20,298	32	2	28
Cleveland, OH	0.95	32	10,642	43	-	-
Nashville, TN	0.95	32	16,669	34	31	23
Cincinnati, OH	0.94	34	11,242	41	2	28
Alurquerque, NM	0.91	35	12,565	40	135	18
Memphis, TN	0.91	35	8,714	46	2	28
Minn-St. Paul, MN	0.90	37	20,710	30	393	12
Ft. Lauderdale, FL	0.89	38	23,679	24	-	-
Hartford, CT	0.89	38	14,600	39	-	-
San Antonio, TX	0.89	38	16,181	37	259	14
Ft. Worth, TX	0.87	41	22,037	29	-	-
Louisville, KY	0.86	42	10,768	42	86	22
Indianapolis, IN	0.85	43	5,267	48	29	24
Columbus, OH	0.82	44	16,237	36	96	20
Pittsburgh, PA	0.82	44	25,235	23	142	17
Salt Lake City, UT	0.81	46	9,624	45	29	24
Oklahoma City, OK	0.78	47	9,921	44	15	26
El Paso, TX	0.74	48	5,241	49	12	27
Kansas City, MO	0.72	49	7,494	47	159	16
Corpus Christi, TX	0.70	50	3,497	50	0	31

- Missing values due to "grouped" data in HPMS database.
Source: TTI Analysis

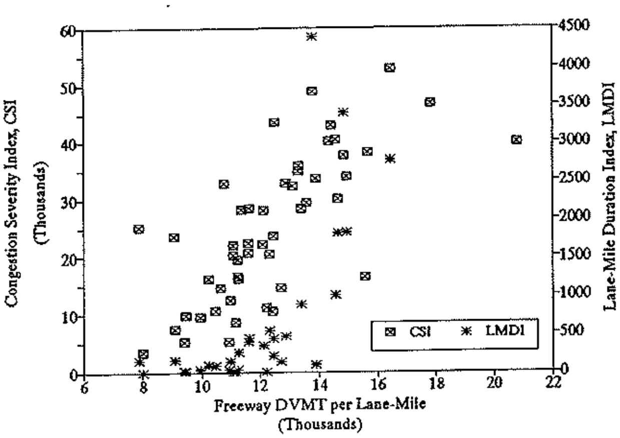

FIGURE 2. DAILY VEHICLE-MILES OF TRAVEL PER LANE-MILE VERSUS CONGESTION LEVEL.

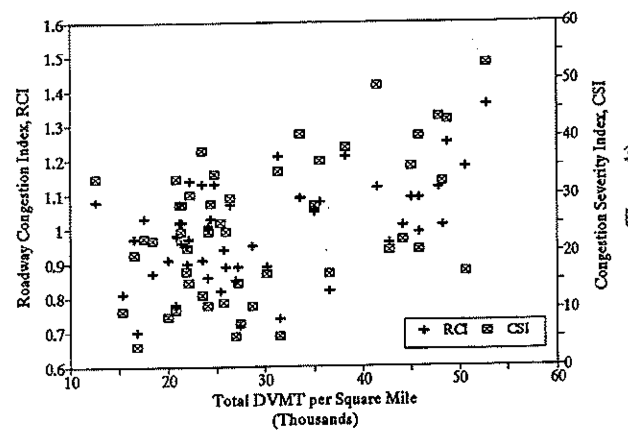

FIGURE 3. DAILY VEHICLE-MILES OF TRAVEL PER SQUARE MILE VERSUS CONGESTION LEVEL.

The third indicator that had a significant relationship to congestion level was average daily freeway traffic per hourly capacity (AADT/C). Because AADT/C is a facility measure, each HPMS freeway segment in an urban area was classified into a congestion range corresponding to the AADT/C value. Previous research by Cottrell assumed that congestion occurred on a facility when the AADT/C value was greater than 9 (11). It was beyond the scope of this study to determine the accuracy of this value; consequently, an AADT/C value greater than 9 represented congested conditions for the purposes of this study. The lane-miles for all HPMS freeway segments were totaled in the following AADT/C ranges: less than 9, 9 to 11, 11 to 13, 13 to 15, 15 to 17, and greater than 17. Very few urban areas had freeway segments with an AADT/C value higher than 17.

In order to make a comparison of this indicator to a congestion measure, 31 of the 50 urban areas (areas with "grouped" data were excluded) were grouped according to the RCI value. The RCI congestion ranges were 0.70 to 0.85, 0.85 to 0.95, 0.95 to 1.05, 1.05 to 1.20, and greater than 1.20. The comparison is illustrated in Figure 4. It can be seen that, as the percentage of freeway lane-miles in higher AADT/C ranges increases, the RCI congestion range also increases. For example, the percentage of lane-miles with AADT/C less than 9 (no congestion) is much less in the RCI range of greater than 1.20 (heavy area-wide congestion) than in the 0.70 to 0.85 range (none to low area-wide congestion). The implication of Figure 4 is that, as the distribution of freeway lane-miles shifts towards a higher AADT/C value, the congestion level increases. Due to the nature of this comparison, an R^2 value was not available. The congestion severity index and the lane-mile duration index were not included in this comparison due to the lack of a definition of congestion ranges.

The average AADT/C values for each RCI congestion range are displayed at the bottom of each bar for the respective range. This value was determined by weighting the percentage of lane-miles for each AADT/C range by the corresponding average AADT/C value for that AADT/C range. The average AADT/C value at the beginning of area-wide congestion (RCI range of 0.95 to 1.05) is 7.1. The initial premise of this particular examination was that congestion on a facility begins at an AADT/C value of 9. The discrepancy in these two numbers — 7.1 and 9 — may be partially attributed to the translation of AADT/C from a facility measure to an area-wide average value.

Several indicators had a moderate correlation to congestion level. These indicators had R^2 values between 0.35 and 0.45, and all were intuitively related to congestion. Those indicators are registered vehicles, daily vehicle-miles of travel, and population density. Most of the transit indicators fared poorly, having R^2 values below 0.1. Surprisingly, many of the supply-related indicators had low correlations to the level of congestion. For instance, freeway lane-miles per square mile (freeway density) is shown in Figure 5. Freeway density is an indicator used often by automobile clubs and other groups lobbying for construction of new freeway facilities because of congestion. Figure 5 shows that freeway density has a low correlation to congestion level, indicating that there may be several other variables which more strongly affect congestion level.

CONCLUSIONS

This report examined the relationships between possible indicators and congestion level as estimated by three congestion measures. The examination included 50 large and medium U.S. urban areas for the year 1989. The study

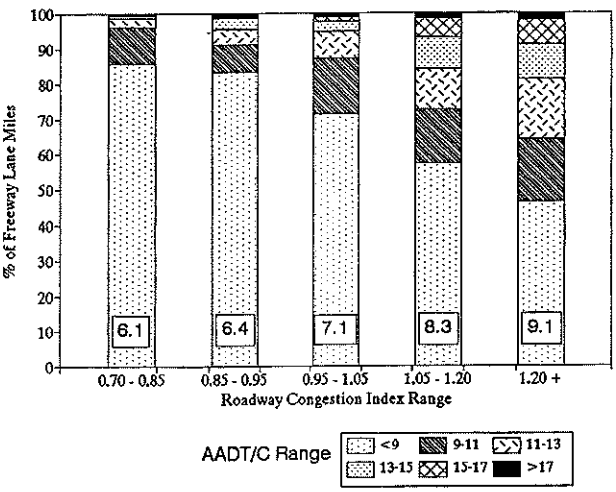

FIGURE 4. SHARE OF FREEWAY LANE-MILES BY AADT/C AND RCI RANGE.

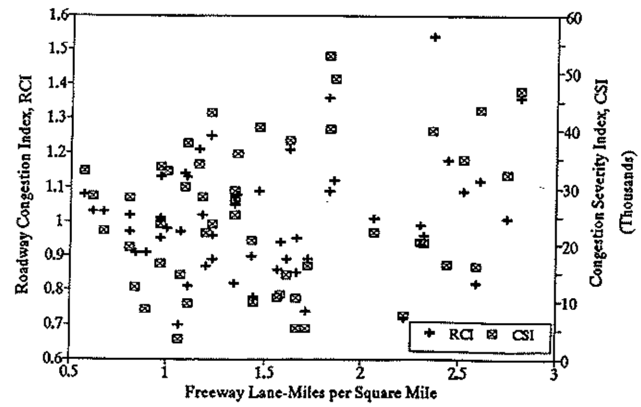

FIGURE 5. FREEWAY DENSITY VERSUS CONGESTION LEVEL.

gathered data from the TTI congestion database and the Highway Performance Monitoring System (HPMS) database.

Indicators of Congestion

This study identified three indicators with a high correlation to congestion level: daily vehicle-miles of travel (DVMT) per lane-mile, daily vehicle-miles of travel (DVMT) per square mile, and average daily freeway traffic per hourly capacity (AADT/C). The indicator of DVMT per lane-mile had R^2 values of 0.68 and 0.45 for the congestion severity index and the lane-mile duration index, respectively. The indicator of DVMT per square mile had the next highest correlation, with an R^2 value of 0.48 for the roadway congestion index. It was shown that freeway AADT/C had a clear relationship to the roadway congestion index, although an R^2 value was unobtainable for the type of comparison made.

It should be noted that each of these three indicators is a gauge of the travel intensity for a particular area. It is concluded, then, that travel intensity is most directly related to congestion level, and would be the most useful type of indicator for area-wide congestion measurement purposes. This is not to deny, however, the importance of the effects of roadway supply or demographic factors within an urban area on congestion level.

Congestion Measures

Three congestion measures were used to estimate the congestion level for the urban areas in this study. Two of the measures, the roadway congestion index and the congestion severity index, were found to be very comparable in the results they produced. A regression analysis was used to calculate a "best-fit" line (Figure 1) through the linearly related data ($R^2 = 0.72$). Because the roadway congestion index is normalized, an RCI value greater than 1.0 represents undesirable area congestion. This RCI value of 1.0 was related to an approximate CSI value of 24,000 using the calculated regression line. It is suggested, then, that a CSI value greater than 24,000 represents undesirable area congestion.

Ideally, congestion measures should provide an accurate representation of mobility for a transportation system. Freeways and principal arterial streets are both major providers of mobility in urban areas, and were included in two of the three measures in this study. Limited data in the HPMS database and lack of a sound analytical procedure prevented the inclusion of principal arterial streets in the lane-mile duration index. An illustration of the importance of principal arterial streets is presented in Table 3, where delay is compared between these two different functional classes.

It can be seen that, for several cities, delay (loss of mobility) occurs primarily on the freeway system. For several areas, however, equal or greater delay occurs on principal arterial streets. This illustrates the importance of principal arterial streets in determination of area-wide mobility.

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TABLE 3. DELAY BY FUNCTIONAL CLASSIFICATION FOR 10 SELECTED URBAN AREAS.

Urban Areas	Freeway Delay (1000 vehicle-hours)	Principal Arterial Delay (1000 vehicle-hours)
Atlanta, GA	133,113 (71%)	53,928 (29%)
Chicago, IL	246,637 (63%)	147,008 (37%)
Detroit, MI	165,885 (57%)	125,612 (43%)
Houston, TX	276,488 (88%)	38,741 (12%)
Los Angeles, CA	1,119,387 (77%)	325,949 (23%)
New York, NY	884,915 (72%)	351,790 (28%)
Oklahoma City, OK	6,116 (38%)	9,771 (62%)
Philadelphia, PA	66,317 (32%)	139,422 (68%)
Phoenix, AZ	32,843 (26%)	95,501 (74%)
Tampa, FL	10,998 (34%)	20,882 (66%)
Source: TTI Analysis		

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Evaluation of a High Volume Merge Operation

VORANIQUE V.WAGNER

There is very little data that exists on high volume merge operations. When the number of vehicles attempting to merge into freeway lanes is more than the freeway can comfortably accommodate, congestion can occur. As freeways reach capacity, there becomes a need to understand high volume merge operations. The purpose of this study was to observe a high volume merge site, evaluate the data collected from the site, and determine characteristics of travel when congestion begins and when freeway travel becomes unacceptable. Speed was the most important parameter used in this determination. A computer simulation was utilized to analyze the data, and data from the site was used to calibrate the simulation model. The results indicate that when 75-80 vehicles attempt to merge from the ramp in a five-minute period (900-960 vehicles per hour), the freeway speeds begin to become unacceptable. When the volume increases to an average of 105 vehicles in a five-minute period (1260 vph) the freeway segment is experiencing undesirable operating conditions.

INTRODUCTION

As a whole, most freeways and freeway systems have served the design year capacity in a satisfactory manner. Due to drastic increases in urban development and population growth, however, certain operational difficulties have resulted, mostly due to congestion in peak traffic periods. When these conditions occur, traffic flow entering from a ramp can cause turbulence in the freeway lanes which results in congestion or stoppage. Both through and entering traffic are forced to wait their turn to pass through the critical area. This critical area is defined as the point at which merging traffic creates highly undesirable operating conditions on the through freeway lanes.

High volume merges are good examples of the above mentioned operating conditions. High volume merge operations consist of high volumes of traffic flowing through the mainlanes while, simultaneously, high volumes of traffic are attempting to merge onto the mainlanes. The traffic flow, under these conditions, may eventually reach an unacceptable level of service.

The efficiency of traffic movement on the through lanes of an urban freeway is directly affected by the adequacy of the associated ramps. Efficient freeway operation is largely dependent on the facilities provided for vehicle ingress and egress. The proper design and placement of ramps on high-volume freeways is therefore imperative if those facilities are to afford fast, efficient, and safe operation. If efficient operation is to be obtained, the facilities must be designed so that traffic entering or leaving the freeway will have a minimum of influence on through freeway traffic. The development of such suitable designs depends to a large extent on the accurate determination of the capacity at the ramp junction, referred to as the merging capacity. The merging capacity is the maximum number of vehicles that can be accommodated on the freeway with a continual backlog or queue of ramp vehicles.

OBJECTIVES

The primary objective of this report is to observe and analyze a high volume merge operation, and determine the volume level at which uniform flow can not be maintained. Data collected from a high volume merge site and compiled from a computer simulation model were used to accomplish the objective.

In order to evaluate the high volume merge operations and obtain a volume level at which freeway travel becomes congested and unacceptable, the following research activities were conducted:

1. Determine the speed and volume values when merging and through vehicles begin traveling in forced flow conditions.
2. Determine the speed and volume values when merging operations become unacceptable.
   a. Determine characteristics of travel at unacceptable speeds.
   b. Compare field speeds to unacceptable speed and determine when the flow becomes unacceptable.

LITERATURE REVIEW

Speed-volume relationships have been used extensively to express operating conditions and capacity, and as a measure of the efficiency of traffic facilities (1). It has been difficult to determine the limits of efficiency desired on freeways. Speed, however, is a very valuable parameter in the evaluation of congestion.

In the 1956 Proceedings of the Institute of Traffic Engineers, it is recommended that the design hourly volume (dhv) be the hourly volume such that the average speed during the highest 15 minutes will not be less than 45 miles per hour (mph) (2). At an average speed of 45 mph, operation is usually very smooth with gaps available for lane changing and no undue strain for urban driving conditions. It is believed that volumes which result in an average freeway speed of 45 mph should be considered as the upper limit of practical capacity. At an average speed of 45 mph, the traffic in the shoulder lane will average about 40 mph and traffic in the median lane will average near 50 mph.

The proceeding also stated that undesirable congestion is usually experienced when traffic volumes increase to the point where operating speeds are reduced to 35 mph or less. At this speed, traffic is practically bumper to bumper; there are very few acceptable gaps available for lane changing, and there is noticeable driving tension even for short rides. Also, at this speed, stoppages can occur quickly even from a single driver's faulty maneuver or hesitation.

METHODOLOGY

Study Site

In the early stages of this study, a site was selected having characteristics of operational difficulty necessary for this research. One section of US 290, the westbound Tidwell entrance ramp, in Houston, TX was chosen as the study site. This site was selected because the characteristics of a tapered acceleration lane 1060 feet in length were consistent with the parameters for analysis of the primary objective. During peak periods, high volumes of traffic flowed in both the mainlanes as well as the ramp. Figure 1 shows a layout of the study site. The following list of features summarizes the geometric features of the study site:

1. Three 11-foot through lanes in each direction.

2. A High Occupancy Vehicle (HOV) lane, located in the center of the freeway, enclosed by Concrete Median Barriers on each side.

3. One-way frontage roads on each side of freeway.

4. A tapered acceleration lane with the taper running approximately 1060 feet long and 21 feet wide at the beginning of the taper.

5. A 14-foot wide entrance ramp.

6. A 10-foot wide shoulder.

7. The entrance ramp is part of an X-interchange configuration.

8. An exit ramp (Fairbanks N. Houston) located approximately 2000 feet downstream of the entrance ramp.

Field Data

Data Collection. Two methods of obtaining data for this study were selected. The primary method of securing data was by utilizing a video camera, and the secondary method consisted of loop detectors, an electronic data collection technique. Data was collected on Monday, July 1, 1991. The data collection began at 3:00 p.m. with the loop detectors and at 4:00 p.m. with the video camera. The stopping point for both data collection devices was around 5:45 p.m. at which time it began to rain and all equipment was packed up.

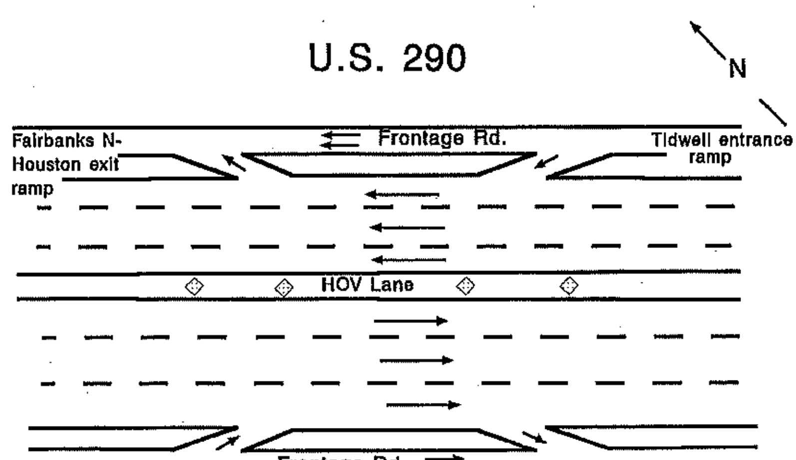

FIGURE 1. SCHEMATIC LAYOUT OF STUDY SITE LOCATION.

An 8-mm video camera was used to record the merging and through vehicles at the site. The video camera was placed on a tripod which was positioned on the Fairbanks N. Houston exit sign located off the shoulder in between the Tidwell and Fairbanks N. Houston entrance and exit ramps. The camera was stationed approximately 25 feet high and was positioned to videotape vehicles entering the freeway from the Tidwell entrance ramp as well as all three lanes of through traffic. Figure 2 shows the approximate location and field of view of the video camera.

As displayed in Figure 3, six permanent inductive loop detectors, two in each of three lanes, were placed approximately 350 feet downstream of the end of the taper from the ramp. The detectors were able to collect data during free flow and congested conditions. The placement of detectors on the freeway mainlanes before the ramp and on the ramp itself, however, would have enabled a more accurate analysis. The loop detectors were used to collect speed, headway, and volume data.

Data Reduction. The main source used for data reduction was the 8-mm video. The information was transferred to VHS video cassettes and viewed to count vehicles and analyze the characteristics of travel. While observing the VHS video, fifteen minute volumes were measured from 4:00 p.m. to 5:30 p.m. for the ramp and all three mainlanes. The volumes are shown in Table 1. The 15-minute volume data, however, were too coarse to allow one to perceive what was actually taking place within the ramp operations. At many high-volume merges, the vehicles on the ramp did not approach at uniform flow rates as they do on the freeway. Vehicles arrived in platoons due to signal control on the surface streets which made the instantaneous merging rate much higher than the 15-minute volumes indicated. Therefore, to increase accuracy, the entrance ramp volumes were measured in five minute increments so as to more precisely predict the point at which normal traffic flow began to break down. These values are listed in Table 2.

Average speeds of the vehicles every minute from 3:00-5:30 p.m. were obtained from the loop detectors. Statistical Analysis Software (SAS) was utilized to summarize the speed data. The plotted data from SAS is shown in Figure 4. This data was used in the calibration of the computer simulation model.

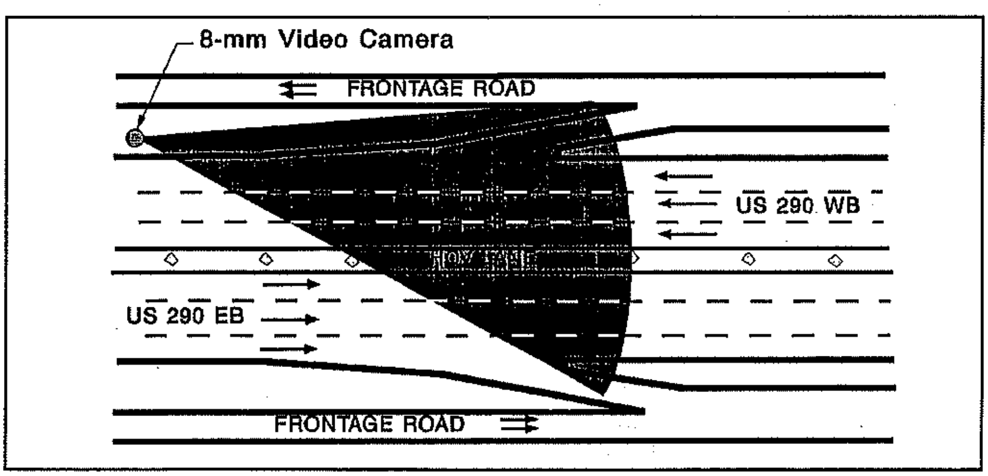

FIGURE 2. VIDEO CAMERA LOCATION AND FIELD OF VIEW. (US 290)

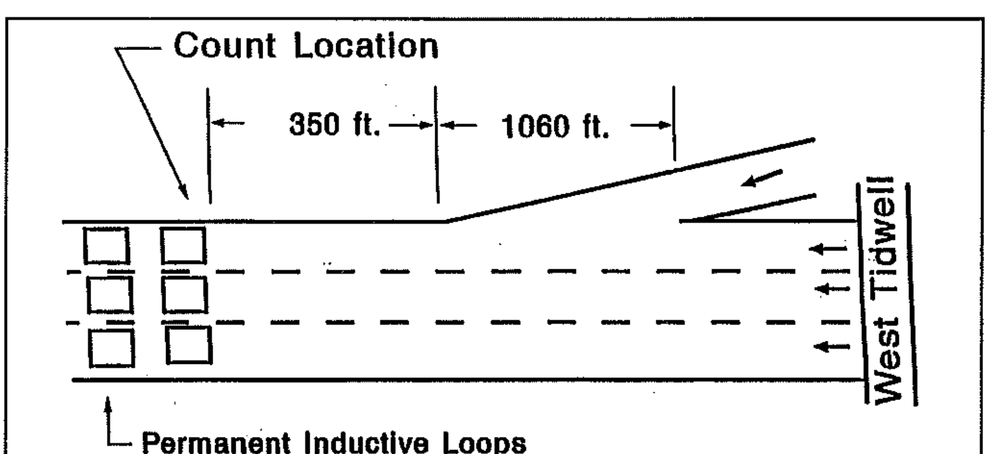

FIGURE 3. SITE DIMENSIONS AND LOOP DETECTOR LAYOUT. (US 290)

TABLE 1. VOLUME DATA REDUCED FROM VIDEO CAMERA.

Time (pm)	15-Minute Volumes Ramp	Lane 1	Lane 2	Lane 3	Freeway
4:00 - 4:15	175	363	441	450	1254
4:15 - 4:30	159	389	476	496	1361
4:30 - 4:45	214	354	497	534	1385
4:45 - 5:00	209	416	516	574	1506
5:00 - 5:15	271	318	575	554	1447
5:15 - 5:30	244	300	548	542	1390

TABLE 2. FIVE-MINUTE RAMP VOLUMES.

Time (pm)	Volume
4:00 - 4:05	51
4:05 - 4:10	65
4:10 - 4:15	57
4:15 - 4:20	52
4:20 - 4:25	57
4:25 - 4:30	52
4:30 - 4:35	58
4:35 - 4:40	89
4:40 - 4:45	67
4:45 - 4:50	69
4:50 - 4:55	64
4:55 - 5:00	77
5:00 - 5:05	109
5:05 - 5:10	117
5:10 - 5:15	89
5:15 - 5:20	91
5:20 - 5:25	94
5:25 - 5:30	71

Computer Simulation

Description of Model. The simulation model used in the study was FRESIM (FREeway SIMulation), a microscopic freeway simulation model capable of simulating the traffic behavior on freeway only networks (3). In a microscopic simulation model, each vehicle is modeled as a separate entity. The FRESIM model provides the highest available level of detail that can be achieved in simulating traffic behavior on a freeway at the present time. FRESIM was chosen as the method of simulation because its higher levels of detail are essential in such instances as merging and congested freeway conditions. FRESIM also has the ability to adequately represent the geometrics and the operational characteristics of the test-site.

Calibration of Model. A conceptual model was developed to represent the study site as shown in Figure 5. Link A represents the mainlanes before the ramp, Link B the mainlanes after the ramp, and Link C the ramp itself. The loop detectors were located on link B. Modifying the free flow speeds on link B was necessary to achieve calibration. When the simulation ran, and the speeds for link B closely matched the speeds obtained from the loop detectors, the model was considered calibrated and could, therefore, be used for further analysis.

Output Parameters. Variables used as measures of congestion are speed, volume, and density. The computer simulation provided values for these factors for every 15-minute period, however, the model gives cumulative statistics. For example, an average speed was computed for the period from 4:00 p.m. to 4:15 p.m. The succeeding average speed, from 4:15 p.m. to 4:30 p.m., was not just the average speed from that interval, but was the average speed from 4:00 p.m. to 4:30 p.m. The average speed, density, and volume for each 15-minute period were calculated and are tabulated in Table 3.

FINDINGS

In examining the video and regarding the ramp volumes, it was observed that the volume at which congestion began to occur (speed is below 35 mph) was when approximately 75-80 vehicles attempted to merge into the freeway lanes within a five minute period (900-960 vph). Looking back at Table 2, that volume was first exceeded in the time interval from 4:30-4:45 p.m. From 5:00-5:15 p.m., when an average of 105 vehicles attempted to merge in a 5-minute period (1260 vph), the conditions of flow became unacceptable.

The congestion that occurred was mostly observed in Link A. Figure 6 shows the relationship of speed and time for each link. The speeds on Link A were definitely the lowest. On each section of the freeway, speeds steadily decreased until after the traffic peak period, beginning at

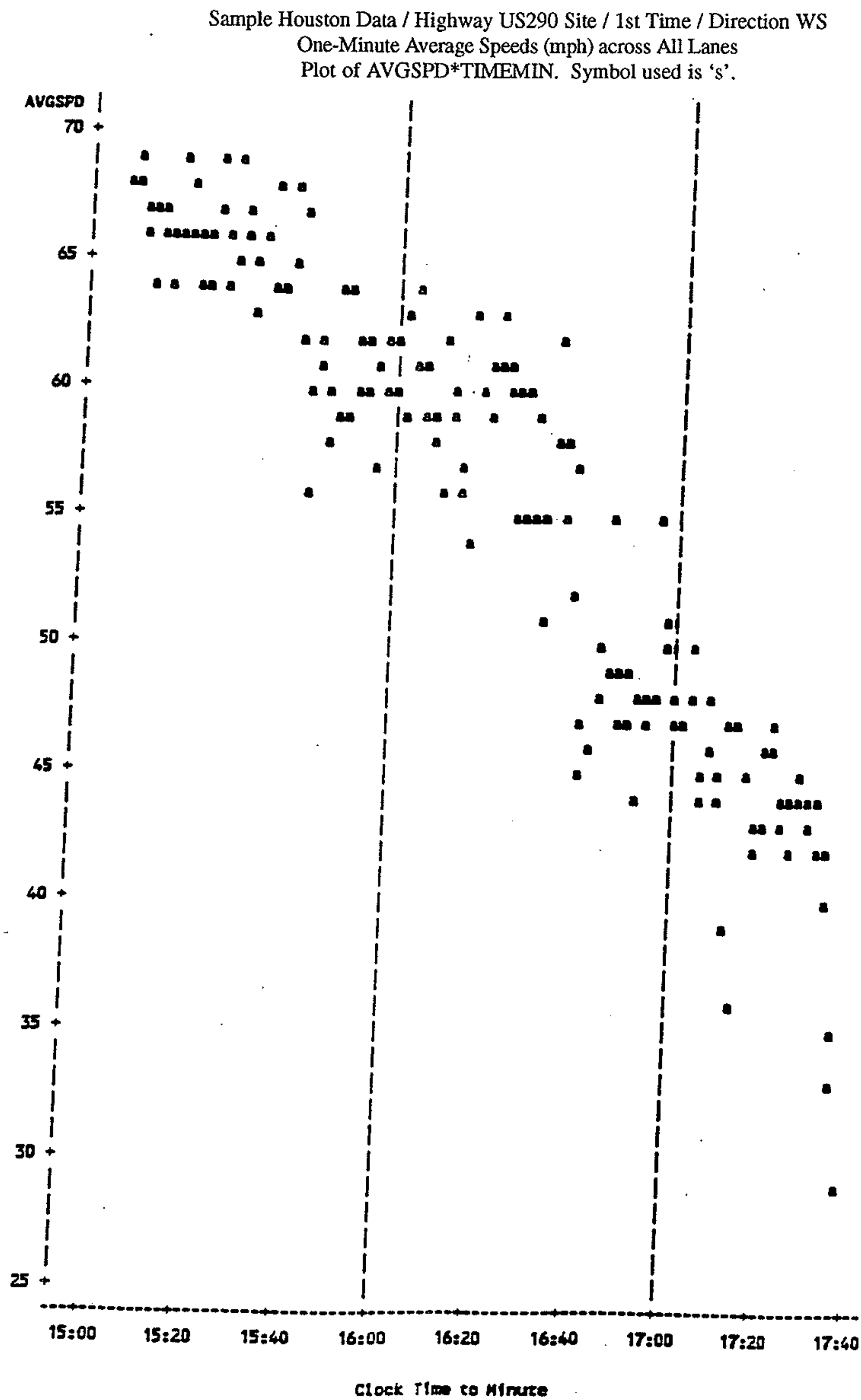

FIGURE 4. PLOT OF AVERAGE SPEED VS. TIME OBTAINED FROM LOOP DETECTORS.

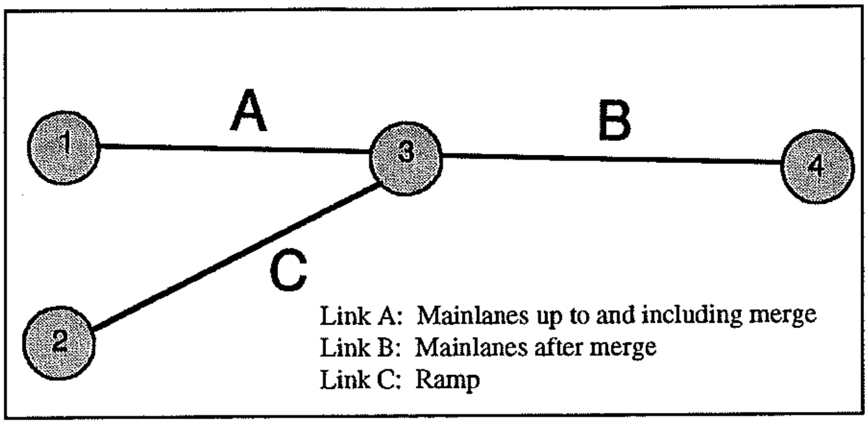

FIGURE 5. REPRESENTATION OF LINK MODEL USED IN COMPUTER SIMULATION.

TABLE 3. SIMULATION DATA FOR EVERY FIFTEEN-MINUTE PERIOD.

Link	Vehicle Minutes	Vehicle Miles	Average Content	Density (v/ln-mi)	Speed (mph)	Volume (v/l/h)
Time Period 1 4:00 - 4:15						
A	428.5	477.8	28.6	25.2	66.9	1685.9
B	705.5	676.5	47.0	33.1	57.5	1903.3
C	22.7	17.0	1.5	15.8	44.9	709.4
Time Period 2 4:15 - 4:30						
A	1081.6	988.4	43.5	38.3	46.9	1796.3
B	1496.5	1375.8	52.7	37.1	53.0	1966.3
C	43.6	32.4	1.4	14.7	44.2	649.7
Time Period 3 4:30 - 4:45						
A	2091.4	1514.4	67.3	59.2	31.3	1853.0
B	2384.0	2122.3	59.2	41.7	50.5	2105.9
C	71.5	53.0	1.9	19.6	44.3	868.3
Time Period 4 4:45 - 5:00						
A	3201.3	2067.7	74.0	65.1	29.9	1946.5
B	3313.8	2907.6	62.0	43.6	50.7	2210.5
C	99.1	73.1	1.8	19.4	43.7	847.8
Time Period 5 5:00 - 5:15						
A	4387.5	2605.4	79.1	69.6	27.2	1893.1
B	4272.5	3696.2	63.9	45.0	49.4	2223.0
C	134.7	99.2	2.4	25.1	44.0	1104.4
Time Period 6 5:15 - 5:30						
A	5532.8	3153.7	76.4	67.2	28.7	1928.6
B	5227.2	4487.0	63.6	44.8	49.7	2226.6
C	166.8	122.7	2.1	22.6	43.9	992.1

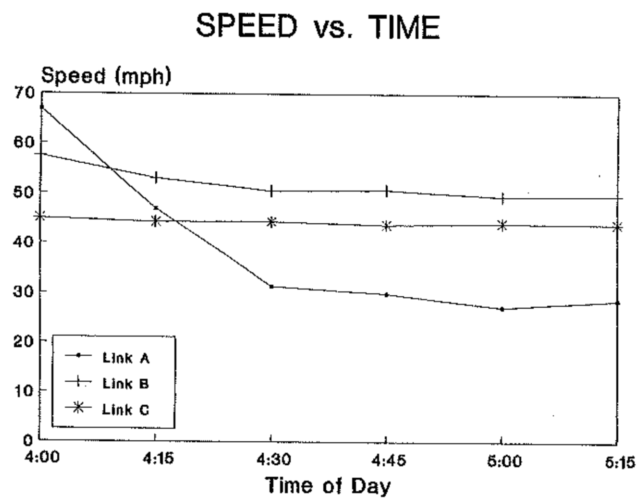

FIGURE 6. PLOT OF SPEED VS. TIME.

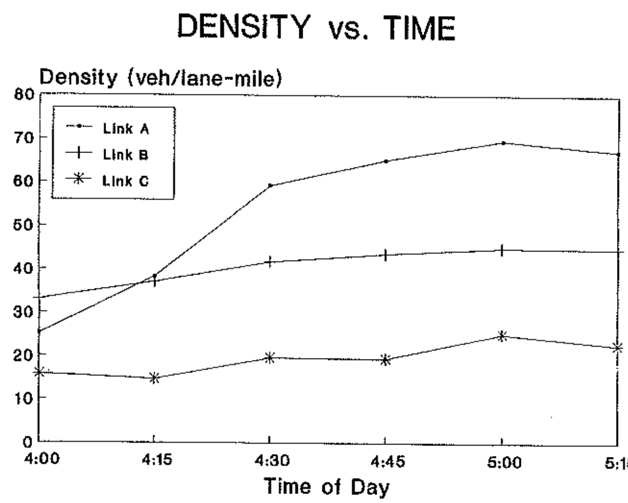

FIGURE 7. PLOT OF DENSITY VS. TIME.

5:00 p.m., at which time the speeds picked up. The speed on Link A during the peak was 27.2 mph. The speeds of the vehicles on the ramp, however, remain fairly steady at about 44 mph. Once the vehicles pass the merging point the speeds tend to increase, that is, the speeds on Link B are greater than those on Link A. The conditions of travel improve once the vehicles pass the point at which vehicles merge.

A similar relationship in densities as shown for speeds is displayed in Figure 7. The densities steadily increase until the traffic peak period, at which point they began to decrease. The highest densities are noticed on Link A. At the peak period, the densities reach a high of 69.6 v/ln-mi. For a basic freeway section, densities greater than 67 v/ln-mi are considered Level of Service F and signify undesirable conditions (4).

RESULTS

In analyzing the data, several conclusions were drawn based on the relationship of speed and densities with the time of day, and position of the vehicle on the freeway. From the analysis it was determined that, because the vehicles were traveling at the slowest rate on the mainlanes prior to the ramp, the point after which the vehicles merged was a discharge from a queue. The congestion occurred on the mainlanes before the ramp and includes the point at which vehicles were merging. This is illustrated in Figures 6 and 7.

Congestion became increasingly noticeable when an average of 75-80 vehicles attempted to merge in a 5-minute period (900-960 vph) with an average of about 6400 vph flowing in the mainlanes. Merges at these volumes tended to cause speeds to reduce to an undesirable level of 31.3 mph. Operating speeds were as low as 27.2 mph when the average volume in a 5-minute period increased to 105 vehicles (1260 vph).

CONCLUSIONS

The results of this study indicate that the ramp volumes should remain below 75-80 vehicles in a five minute period (900-960 vph) if freeway speeds are to be acceptable. When that volume is exceeded congestion is noticeable. Undesirable operating conditions occur when the merging vehicle volumes increase to an average of 105 vehicles in a five minute period (1260 vph). The congestion occurs on the mainlanes prior to the ramp. Beyond that point conditions on the freeway improve, which indicate that the mainlanes after the merging point experienced a discharge from the queue located upstream.

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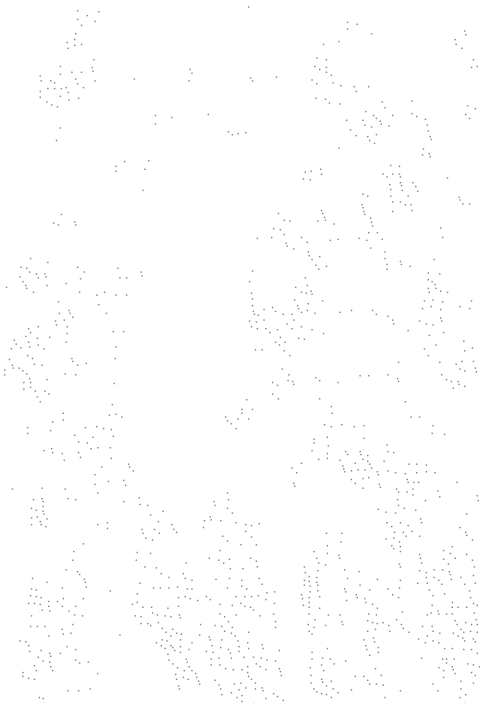
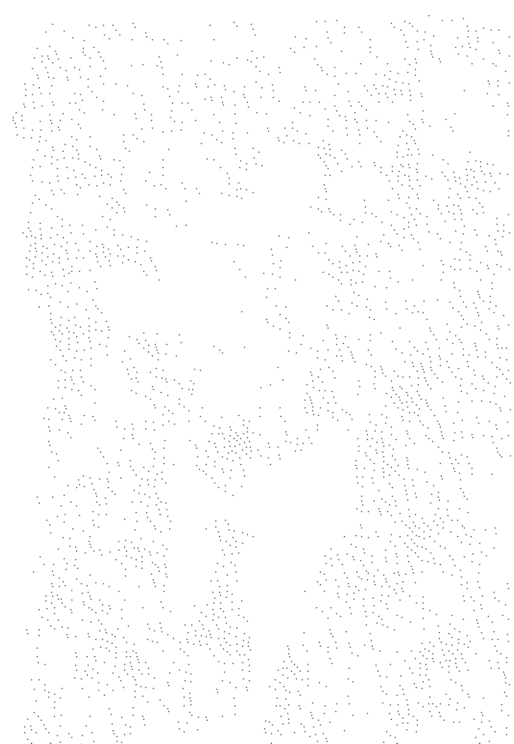


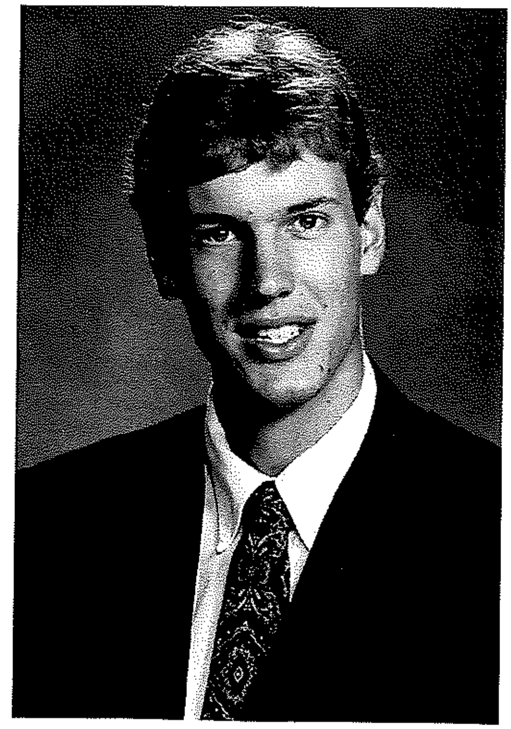
Biographical Data



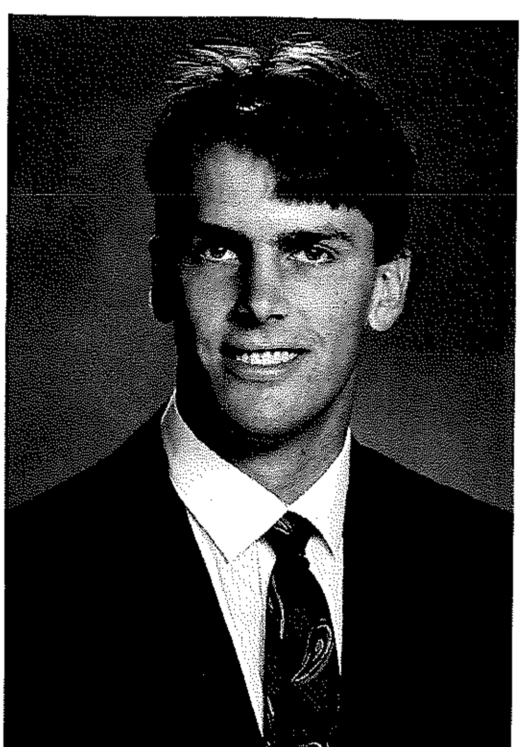
Rick Bartoskewitz was born in Bryan, Texas on March 7, 1970. He is the son of Richard and Polly Bartoskewitz of Bryan. Rick attended school in his hometown, graduating from Bryan High School in 1988. He subsequently enrolled at Texas A&M University to pursue a Bachelor's Degree in Civil Engineering, which he will receive in December 1992.

Rick has been employed by the Texas Transportation Institute since December 1988, originally as a student worker and most recently as a fellow in the Undergraduate Transportation Engineering Fellows Program. At the conclusion of his fellowship, he will continue at TTI as a student worker.

Rick is a member of the Texas A&M Student Chapter of the Institute of Transportation Engineers. He serves as the student chapter's Corresponding Secretary. Rick is also a member of Chi Epsilon. His plans are to pursue a Master's Degree in Transportation Engineering.



RICHARD BARTOSKEWITZ



PHILLIP SCOTT BEASLEY

Phillip Scott Beasley was born in Blacksburg, Virginia on June 9, 1969. Scott grew up in Lynchburg and attended primary and secondary schools in the Campbell County School System, graduating from Brookville High School in June, 1987. In August of 1987 he entered the College of Engineering at Virginia Polytechnic Institute and State University and will receive his Bachelor's Degree in Civil Engineering in May, 1992.

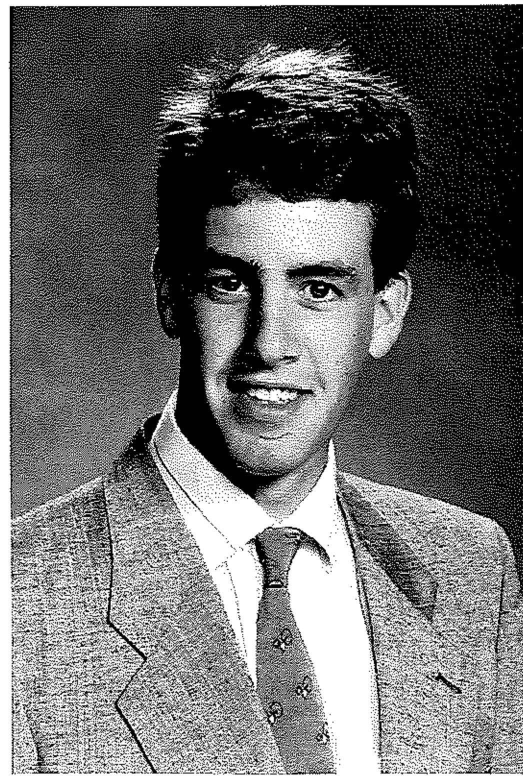
In May, 1988 Scott entered the cooperative education program at VPI & SU. He was employed by the Virginia Department of Transportation and worked at the Lynchburg District and the Christiansburg Residency offices. Scott received an undergraduate transportation engineering fellowship at Texas A & M University during the summers of 1990 and 1991 to work as a research assistant with the Texas Transportation Institute.

Scott is a member of Chi Epsilon, the civil engineering honor society, Kappa Theta Epsilon, the cooperative education honor society and was recently inducted into Gamma Beta Phi, an honor and service organization. He is also a member of the American Society of Civil Engineers, and Circle K, a national service organization. After graduation, Mr. Beasley is planning to pursue a Master's Degree in Transportation Engineering.

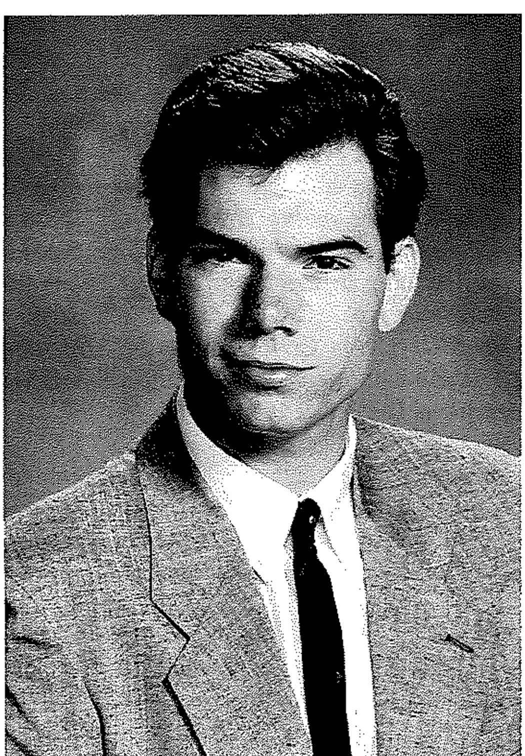
Steven Ross Blanchard was born and raised in the Middle Atlantic region. He has resided in the state of Delaware since the age of six and was raised in Christian roots by a family of two working parents. Hence, he considers himself a rather benevolent and self-sufficient individual.

Upon graduation from St. Mark's High School, Ross chose to enter the University of Pittsburgh with an interest in studying computer science. After completing one year, he decided to regain his roots and transfer back home to the University of Delaware, where he is currently pursuing a Bachelor of Science in Civil Engineering. It was during his study this past semester that he developed interest in the field of transportation. His acceptance in the TTI fellows program stimulated this interest further. Ross aspires greatly towards a career in public interest, however, he considers the pursuit of a graduate degree his primary concern.

One of the most influential elements in Ross' life has been his participation in competitive sports, particularly swimming, as he is a member of the University of Delaware men's swim team. He is active in the American Society of Civil Engineers and has served as an undergraduate teaching assistant in the Civil Engineering Department.



STEVEN ROSS BLANCHARD



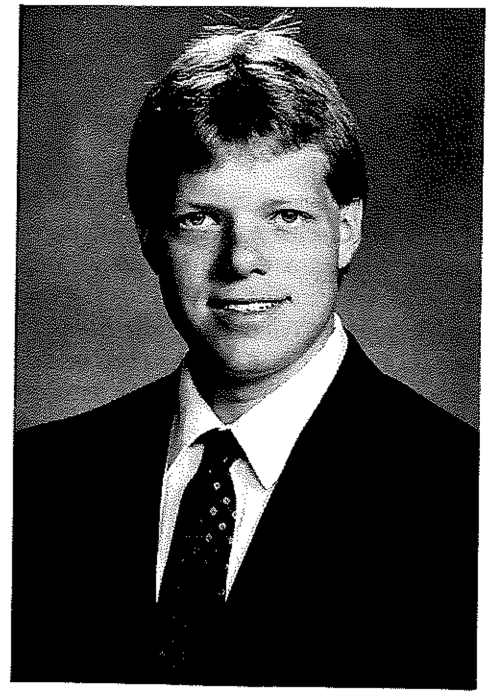
KENNETH LEE FINK

Kenneth Lee Fink attends The Pennsylvania State University where he is majoring in Civil Engineering. He will graduate in May 1992 with a Bachelor of Science in Civil Engineering. Mr. Fink is a member of the American Society of Civil Engineers, Tau Beta Pi, and Chi Epsilon. Upon graduation Mr. Fink plans to pursue a Master's Degree in Transportation Engineering.

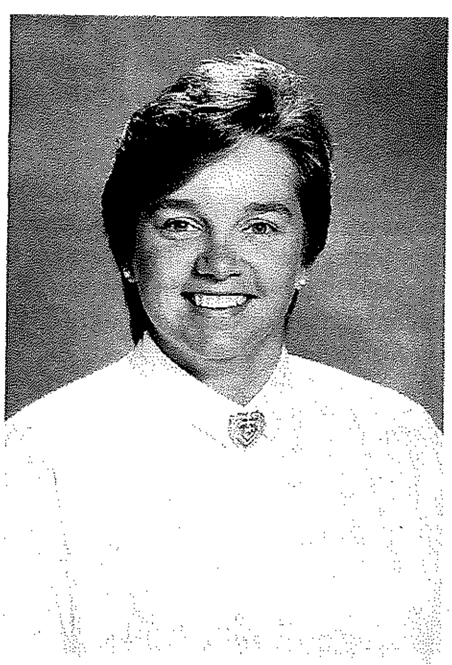
For the past three years, Mr. Fink has been employed on a part-time basis for KCI Technologies, formerly Kidde Consultants Incorporated, a civil engineering consulting firm in Maryland. Prior to this, Mr. Fink was employed by Loiederman Associates Incorporated, also a civil engineering firm in Maryland.

Robert Alan Hamm was born in Somerville, New Jersey on October 27, 1969. In 1979 he moved to Beaumont, Texas where he attended primary and secondary schools in the Beaumont Independent School District. Robert graduated from West Brook Senior High School in May 1988.

During the summer of 1990, Robert was employed by the Texas State Department of Highways and Public Transportation. Robert is currently a senior at Texas A&M University and will receive his Bachelor's Degree in Civil Engineering in August 1992. He is a member of Chi Epsilon, the American Society of Civil Engineers and the National Society of Professional Engineers. Robert is considering attending graduate school following graduation.



ROBERT ALAN HAMM



JANIS LYNN PIPER

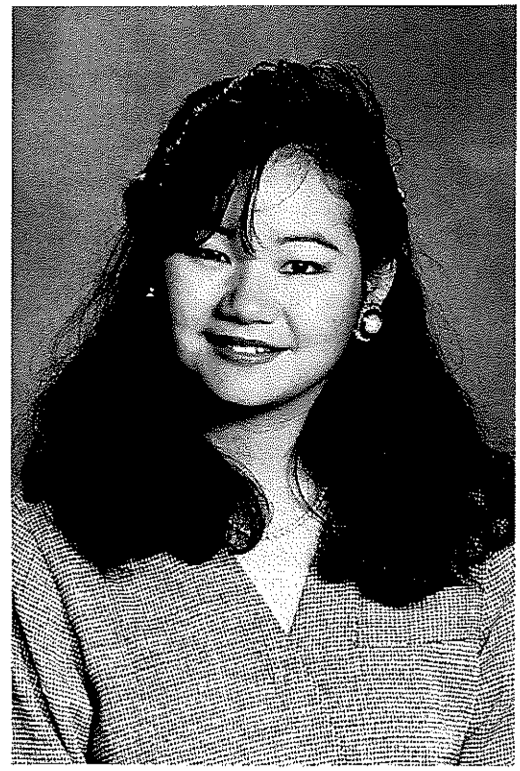
Janis Lynn Piper was born in Bangor, Maine on September 16, 1966. She grew up in Brewer, Maine and graduated from Brewer High School in June 1984. In September of 1984, she entered the University of Maine. After spending three years at the University, she chose to leave the program temporarily to work in the construction industry, returning in the fall of 1989 to complete her undergraduate degree in Civil Engineering. She received an Associate's Degree in Civil Engineering Technology in May 1991 and will receive her Bachelor's Degree in Civil Engineering in December 1991.

Janis received an undergraduate transportation engineering fellowship at Texas A&M University for the summer of 1991. During her fellowship, she was employed to do research with the Texas Transportation Institute.

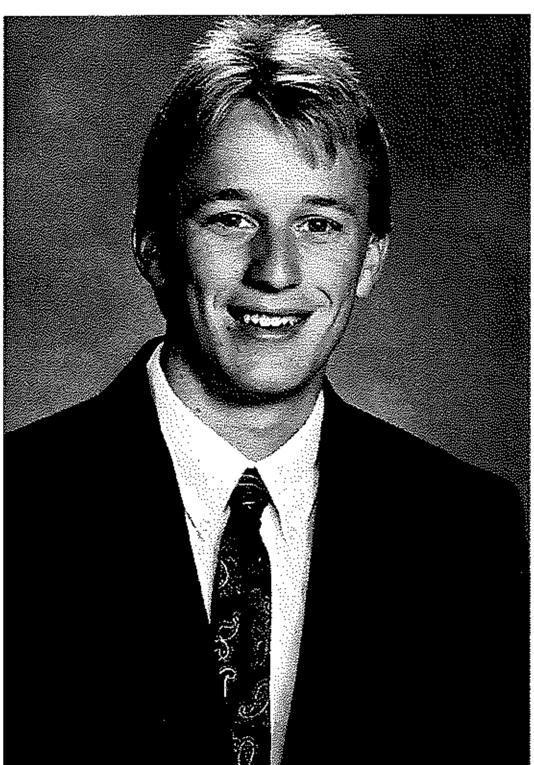
Janis is a member of Pi Mu Epsilon National Mathematics Honor Society, the American Society of Civil Engineers, and is currently the president of Chi Epsilon National Civil Engineering Honor Society. After graduation, Ms. Piper is seriously considering pursuing a Master's Degree at Texas A&M University.

Isabel S. Siu was born in Panama City, Republic of Panama on April 10, 1970. She graduated from high school in 1987, and went to study pre-engineering in Angelo State University in San Angelo, Texas until she transferred to Texas A & M University in the fall of 1989. Isabel was awarded a Distinguished Student Award for the fall of 1990, and received an Undergraduate Transportation Fellowship at Texas A & M University for the summer of 1991. She will receive her Bachelor of Science in Civil Engineering in August 1992, and plans to pursue a Master's Degree in Civil Engineering.

Isabel is a member of the American Society of Civil Engineers (ASCE), the Panamanian Students Association, and the International Students Association (ISA). She practices aerobics, volleyball, badminton, and is also an avid bowler. She has participated in various bowling tournaments and was also a member of the Texas A & M travelling team. She likes to work with international students, and has participated in numerous activities sponsored by ISA including New International Student Orientation, Mini-Olympics, and International Week.



ISABEL S. SIU



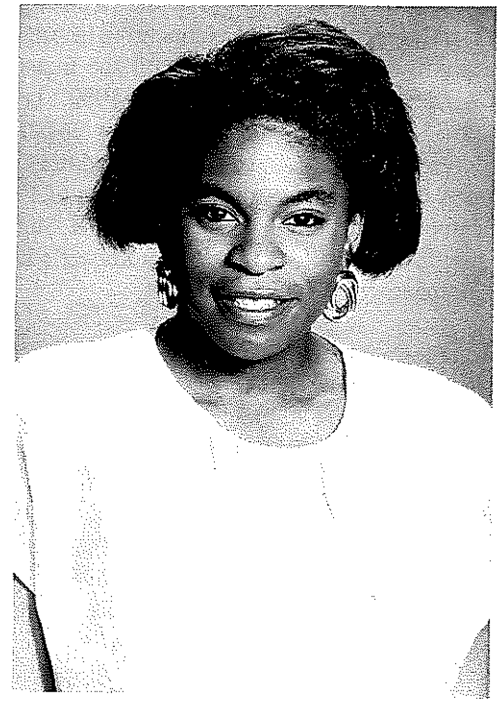
SHAWN M. TURNER

Shawn M. Turner was born in Washington, D.C., on the 15th of July, 1969. While enrolled at Southern Fulton High School in Pennsylvania, Shawn was exposed to the engineering profession through conversations with several cousins who were practicing engineers. Upon graduation from Southern Fulton in 1987, Shawn was admitted to the College of Engineering at The Pennsylvania State University.

About halfway through his college education, Shawn participated in a cooperative education program with the Eastern Federal Lands Highway Division of the Federal Highway Administration in Sterling, Virginia. Upon returning to Penn State after completing the co-op program, he assisted in research on Strategic Highway Research Project C-101, "Assessment of the Physical Condition of Concrete Bridge Components," for the next year. Before participating in the Undergraduate Transportation Engineering Fellows Program at Texas A&M University, Shawn had started working at the Pennsylvania Transportation Institute.

At Penn State, Shawn is a member of the University Scholars Program, the student chapter of ITE, and was once a member of the Cycling Team. Shawn will graduate with a B.S. in Civil Engineering with honors in January, 1992, and is planning to enter into a master's program at an undetermined graduate school.

Voranique V'Yarn Wagner was born in Monroe, Louisiana on August 24, 1970. She spent the first sixteen years of her life traveling because her father was in the military. In 1986 she moved to Killeen, Texas and graduated from Killeen High School in 1988. She then began her college education. She entered the College of Engineering at Texas A&M University, and will receive her degree in Civil Engineering in May 1993. In an Introduction to Civil Engineering course taken her freshman year, Voranique gained an interest in Transportation Engineering.



VORANIQUE V'YARN WAGNER

Voranique received a fellowship at Texas A&M University for the summer of 1991. During her fellowship, she was employed as a Research Assistant with the Texas Transportation Institute.

Voranique is a student member of the Texas A&M Chapter of Chi Epsilon. After graduation she plans to pursue a master's degree, however, she has not decided her field.