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Program Coordinator: Daniel B. Fambro

Mentors: David F. Pearson, Donald L. Woods, Dallas N. Little, Daniel B. Fambro, Russell Henk, Kay Fitzpatrick, and Timothy J. Lomax

16. Abstract

The engineering research reports in this document resulted from the third year of the Undergraduate Transportation Engineering Fellows Program during the summer of 1993. 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 reports document planning research in the investigation of life-cycle variables in estimating trip rates; traffic operations research addressing the effects of lead length on inductive loop detectors and the investigation of the heights of vehicle fleet taillights and headlights as parameters to be incorporated in roadway design; materials research on the effects of fiber and polymer modifications on the fatigue life of asphalt pavement; multi-modal research into the travel time reliability of high-occupancy vehicle (HOV) lanes and the air quality and fuel consumption benefits that can be realized by implementing HOV facilities; safety investigation of limited sight distance at railroad-highway grade crossings; and, maintenance research directed toward the development of cost-effective means of determining the functional condition of signs and markings along highways in Texas.

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Transportation Engineering Research Reports

1993 Undergraduate Transportation Engineering Fellows

SPONSORED BY THE

Advanced Institute in Transportation Systems Operation and Management

IN COOPERATION WITH

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Contents

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, established the Undergraduate Transportation Engineering Fellows Program in 1990. 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, eight students and faculty/staff mentors participated in the program:

STUDENTS	MENTORS
Allison C. Cherry Texas A&M University	Mr. David F. Pearson TTI Assoc. Research Engineer
Juene K. Franklin Texas A&M University	Dr. Donald L. Woods Professor of Civil Engineering
Bridgette D. Keller Clemson University	Dr. Dallas N. Little Professor of Civil Engineering
Joshua E. Martin Cornell University	Dr. Donald L. Woods Professor of Civil Engineering
Jennifer Messick University of Delaware	Dr. Daniel B. Fambro Assoc. Professor of Civil Engineering
Charles J. Naples III Temple University	Mr. Russell H. Henk TTI Asst. Research Engineer
C. Brian Shamburger Texas A&M University	Dr. Kay Fitzpatrick TTI Asst. Research Engineer
Edward D. Sulak, Jr. Texas A&M University	Dr. Timothy J. Lomax TTI Research Engineer

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.
Associate Professor of Civil Engineering
Undergraduate Fellows Program Coordinator

Analysis of Life-Cycle Variables For Texas Travel Surveys

ALLISON C. CHERRY

Trip generation models use trip rates calculated from data collected in travel surveys to predict future travel demand. Currently in Texas, households in these surveys are stratified by income and household size. This study has examined the life-cycle variable, age of the head of household, to determine if any improvement in the accuracy of trip rates over the currently used variables could be achieved.

The variables, age of the head of household, income, and household size, were examined independently and then stratified with each other in two-way stratifications and a three-way stratification. The statistical analyses of the mean trip rates calculated for each cell consisted of calculating standard deviations of the mean and the sample, running a z-test for each cell within the stratifications, and calculating the coefficient of variation and estimated error for each trip rate.

Results obtained were that of the one-way stratifications; household size had the least amount of variation in the trip rates, a low estimated error and, therefore, the most reliable trip rates. Of the two-way stratifications, age of the head of household and household size had the lowest variation of trips within each of the cells and the lowest estimated errors. The use of age of the head of household may offer some improvement over the currently used variables, household size and income. The three-way stratification showed high variations of trip rates and high estimated errors. This may have been due to a lack of observations in some of the cells. There were not enough households to have a statistically accurate sample for the three-way stratification in many of the cells.

INTRODUCTION

As stricter air quality regulations come into effect, the need for more accurate travel demand forecasting is becoming increasingly important. The use of trip generation models which utilize trip rates calculated from travel surveys is an important part of travel demand forecasting. This study examines the use of one life-cycle variable in calculating these trip rates to determine if an improvement could be offered over currently used variables.

Travel Surveys

Travel surveys are an effective tool in transportation planning. They give planners an idea of what is available now and can also be used to monitor changes in travel patterns and demand. By studying the travel patterns of specific groups of people, these groups and their travel can be forecast to determine future travel demand.

Data collected in travel surveys are used in trip generation models to predict the number of trips that will be produced and attracted by certain areas. This is done by calculating trips per household. To improve forecasting abilities, households have historically been stratified by categories such as income of the household, number of people in the household, and number of vehicles available to the household. Currently, trip rates used in Texas are stratified by income and household size (1). During this study, life-cycle will be examined as a new variable to categorize the households from the survey data.

Life-Cycle as a Variable

The term "life-cycle" is used to describe the physiological and psychological periods of an individual's life. Life-cycle is useful in identifying significant stages in the human life and in understanding how a person at one stage is different from or similar to a person at another stage. Some of these differences are clearly biological while others are social. The life-cycle concept also helps to explain behaviors that occur during certain stages of life that may be associated with specific travel patterns (2).

There are many aspects of the life-cycle that were considered for evaluation during this study including the number of children and/or adults within the household, the number of licensed drivers in the household, the number of persons employed in the household, and the age of the head of household. Due to time constraints, only age of the head of household would be examined. Age of head of household was also chosen because these data were available from the 1990 census.

Peoples' activities and their households will be affected by lifestyle changes and changes in responsibility as the household progresses through various stages of life. Most marriages and families are started during a specific life-stage. Children grow up, begin driving, and eventually leave home. This not only changes the characteristics of the original household but also creates a new household. The lifestyle of the household also changes as household members reach retirement age. Because each stage of human life is unique to certain lifestyles and activities, the travel characteristics of a household may be influenced by age.

Objective

As the age and characteristics of a household change, changing activities may influence the travel patterns of the household. During this study, it will be determined if households in different life-cycle groups have significantly different trip rates. The objective of this study is to determine if life-cycle should be used as a variable in developing trip rates for use in planning models. Use of these trip rates may improve the prediction of an area's travel needs.

Background

Travel surveys funded by the Texas Department of Transportation (TxDOT) were conducted in 1990-1991 in San Antonio, Amarillo, Brownsville, Tyler, and Sherman-Denison. Several types of surveys were conducted including household surveys, workplace surveys, special generator surveys, external station surveys, and truck surveys. The information needed for this study was obtained exclusively from the household surveys that were collected.

Information acquired through the household surveys include characteristics of the household such as the number of people in the household, the age and relation of the people, income, number of licensed drivers, the number of persons employed, and the travel during that day for each household member over age 5. This study has analyzed the survey data to estimate trip rates by age of the head of household and has compared and stratified these with income and household size.

Portland, Oregon, is currently using life-cycle as a variable in their trip generation models. This was the only study found using similar variables. A model was created in 1988 in Portland to better simulate transit ridership. The primary data for the household surveys that were conducted for that study were households by size, income, and age of head of household. Other variables collected during that survey and used in the model were number of workers in the household and number of cars available to the household. Trip purposes and travel mode choices were examined in the estimation of future transit ridership (3).

METHODOLOGY

Each of the variables being investigated in this study, income, household size, and life-cycle, were examined independently and then stratified with each other in two-way stratifications. All three variables were also analyzed together in a three-way stratification. In total, seven combinations were statistically examined to determine which would give the most accurate mean trip rates.

Trip Rates

Trip rates are used in trip generation models to estimate the number of trips produced by households within specific income and household size categories. Trip rates are the average number of trips made by each household in a category and are calculated by dividing the number of trips made by households in a category by the total number of households in the category. Person trip rates are calculated using the total number of trips made by each person in a household category. Auto-driver trip rates are calculated by using only the number of trips made by persons who were driving.

Trip rates are calculated for three types of trip purposes. Home based work (HBW) trips are trips made from home to work or from work to home. Home based non-work (HBNW) trips are trips that begin or end at home but are made for a purpose other than work. Non-home based (NHB) trips begin and end somewhere other than home.

Determination of Age Groups

It was decided that the age of the head of household would be used as a variable in classifying households into age groups. Trip rates are calculated by trips per household which requires that each household to be placed into a single age category. To examine age of the head of the household as a variable, specific age groups had to be determined.

Age groups were determined by conducting a literature review on life-cycle and examining characteristics of different age groups. The age groups chosen were 15 to 24, 25 to 44, 45 to 64, and 65+. It was felt that households in the 15-24 age group would have mostly adults living in the household and some of the adults would be attending college rather than working a regular full-time job. Households with children would be seen more in the 25 to 44 age group with at least one adult in the household working. Households in the 45 to 64 age group would have fewer children because the children would be older and leaving home, but the households would still have working adults. The 65+ age group would consist mainly of adults and fewer work trips would be made because of retirement.

The groupings chosen for household size and income are consistent with those currently used for these variables in Texas. Household size is broken into categories of 1-person, 2-person, 3-person, 4-person, and 5+ person households. Income is broken into ranges are $0 to $4,999, $5,000 to $9,999, $10,000 to $19,999, $20,000 to $34,999, and $35,000+.

Utilization of Databases

This study utilized data collected from the TxDOT surveys done in 1990 and 1991. Data for the individual persons of the households were used to classify each household into an age group. These data contained the age, relation, sex, and other information on each member of the household over 5 years of age. Trip information for each of the households along with household size and income was available in a database from the Texas Transportation Institute (TTI). A SAS program was written that placed each of the households into an age group and then merged this database with the age group in order to calculate the trip rates for each of the stratifications.

Stratification of Variables

Households may be stratified by more than one variable to make trip rates more sensitive to changes in the urban area. A two-way stratification of household size and income is presently being used to develop trip rates. Age of the head of household was stratified separately with each of the presently used variables, household size and income, and then together in a three-way stratification.

Examination of Stratifications

These stratifications were analyzed using a spreadsheet. Figure 1 shows an example of the two-way stratification. In this report, the term "cell" will be used to describe an individual rate, such as hh size=1, age=15-24. "Level" will be used to describe one level of the stratification, such as hh=1 or age=15-24. Mean trip rates were calculated for each cell in the seven stratifications. Statistical analyses were performed for each cell to determine which of the stratifications gave the overall least variable trip rates.

Standard Deviation

Both the standard deviation of the sample and the standard deviation of the mean were calculated. The standard deviation of the sample shows to what extent the trips within a cell differ, and the standard deviation of the mean shows how much variation there is in the mean. The standard deviation of the sample was calculated by the same SAS program that was used to calculate the mean trip rates. The standard deviation of the mean was calculated by the
equation:

$$s_{\bar{x}} = \frac{s}{\sqrt{n}} \quad (1)$$

where:

$s_{\bar{x}}$ = standard deviation of the mean
s = standard deviation of the sample
n = number of observations (4)

Z-test

A z-test was performed to measure the statistical difference between the stratifications. A 95 percent confidence level was used to determine if there was enough difference between the levels to offer some improvement. If the z-test resulted in a value of less than -1.96 or greater than 1.96, the trip rates were considered statistically different. The equation used in performing the z-tests was

$$z = \frac{(\bar{x}_1 - \bar{x}_2)}{\sqrt{\frac{s_1^2}{n_1} + \frac{s_2^2}{n_2}}} \quad (2)$$

where:

z = test statistic
$\bar{x}_{1,2}$ = mean trip rate
$s_{1,2}$ = standard deviation of the sample
$n_{1,2}$ = number of households in category (5)

Coefficient of Variation

The coefficient of variation for the sample and the coefficient of variation for the mean were calculated to show relative variation. This was necessary in comparing trip rates because the trip rates were different for each cell, and the standard deviation would vary depending on the magnitude of the trip rates. The coefficient of variation measures the percentage of the mean that the standard deviation represents. The less variation that there is in the trip rates within a cell, the more confidence there will be in the calculated trip rate. Coefficient of variation was calculated using this equation:

$$C.V.=\frac{s}{\bar{x}} \quad (3)$$

where:

$C.V.$ = coefficient of variation
s = standard deviation
$\bar{x}$ = mean trip rate (6)

Required Sample Size

The number of households to be sampled in each category for a statistically accurate sample was calculated using this equation:

$$n=\left(\frac{t*s}{\epsilon}\right)^2 \quad (4)$$

where:
n = required sample size
s = standard deviation
ϵ = user-specified allowable error
t = coefficient of the standard error of the mean that represents user-specified probability level (4)

The allowable error used in the calculation of required sample size was 10 percent. The coefficient of the standard error of the mean used was 1.96 because the confidence level used was 95 percent. The required sample size was calculated to determine if the sample sizes used in this study were large enough to give a reliable estimate of trip rates.

Error

The error for each cell of the stratifications was calculated using the equation:

$$\epsilon=t*\bar{x}*C.V. \quad (5)$$

where:
ϵ = error of the mean
t = coefficient of the standard deviation of the mean that represents user-specified probability level
$\bar{x}$ = mean trip rate for the cell
$C.V.$ = coefficient of variation of the mean (4)

Because a 95 percent confidence level was used, the t-value used during this calculation was 1.96. The error in each cell of the stratifications was multiplied by the percentage of households in those cells. These were summed for the entire stratification to yield an estimate of the weighted average error in the trip rate.

Determination of Best Variables

The coefficient of variation and the percent error were used in determining which stratification gave the most accurate mean trip rates. The stratification that produced the lowest coefficients of variation and the lowest percent errors was considered the best.

STUDY RESULTS

The following are the results of the various statistical tests performed.

Trip Rates

Trip rates were calculated for each of the stratifications. Figures 1-3 show the trip rates for the one-way stratifications for each urban area and the average trip rate for each grouping. In stratifying households by age of the head of household, person trip rates per household are highest for the 25 to 44 age group; they decline for the 45 to 64 age group and for the 65+ age group. In stratifying households by household size, trip rates increase as the size of the household increases. This is also true for the income classifications; person trips per household increase as the income of the household increases.

The trip rates for the two-way stratifications showed the same trends as the one-way stratifications. For the stratification of age of the head of household versus household size, trip rates are highest for households in the 25 to 44 age group and increase as household size increases. A similar pattern was seen for the stratification of age of the head of household versus income. For household size versus income, trips per household increase with an increase in household size and income and are the highest for the category of a household size of 5+ and an income of greater than $35,000.

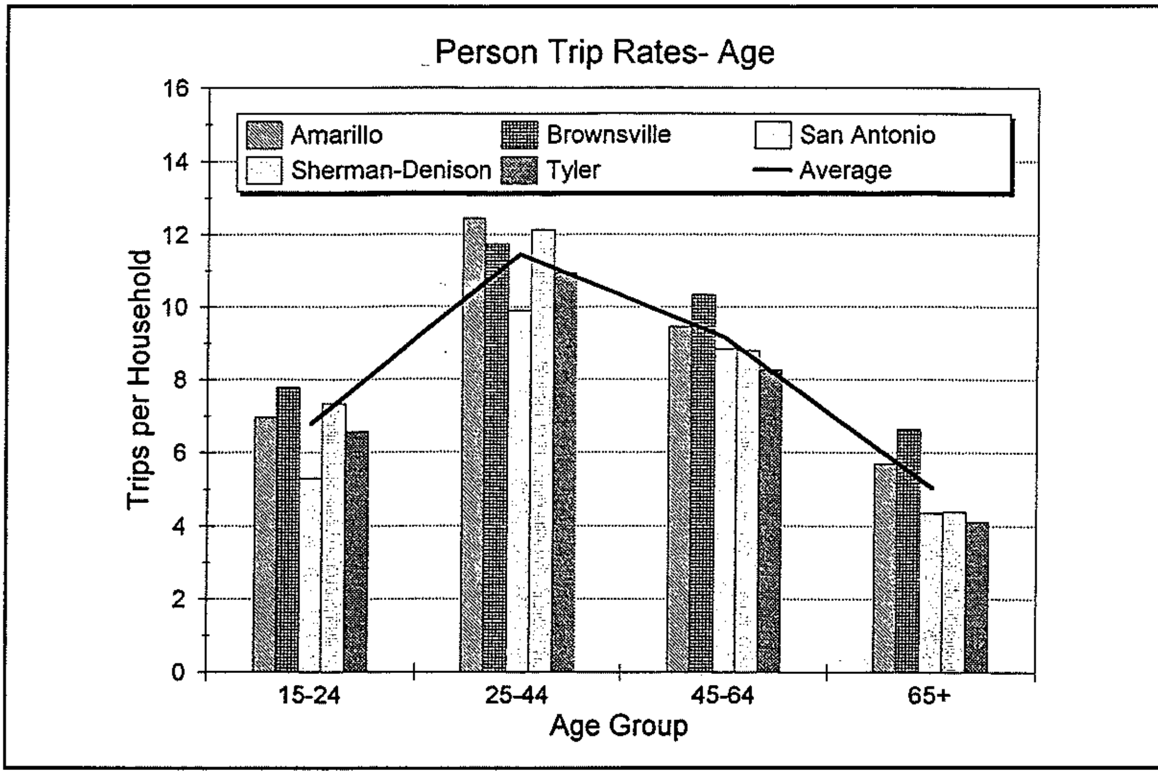

FIGURE 1. TRIP RATES FOR FIVE URBAN AREAS STRATIFIED BY AGE OF THE HEAD OF HOUSEHOLD.

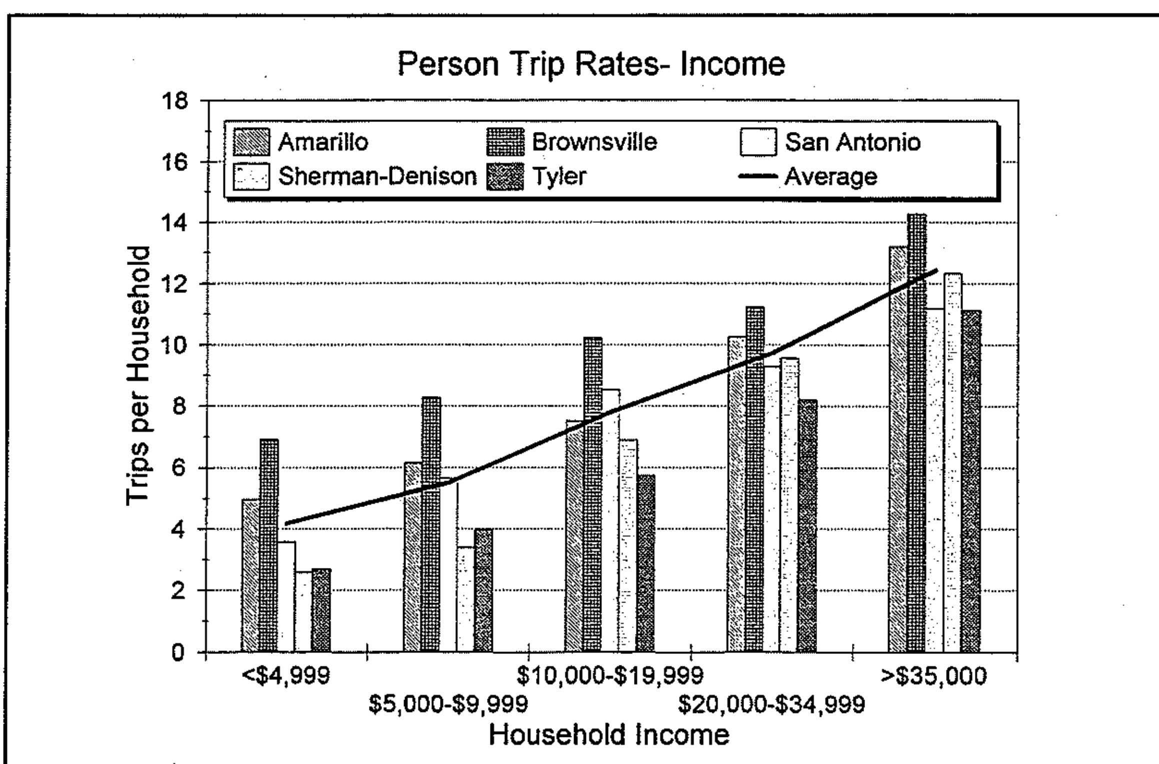

FIGURE 2. TRIP RATES FOR FIVE URBAN AREAS STRATIFIED BY HOUSEHOLD INCOME.

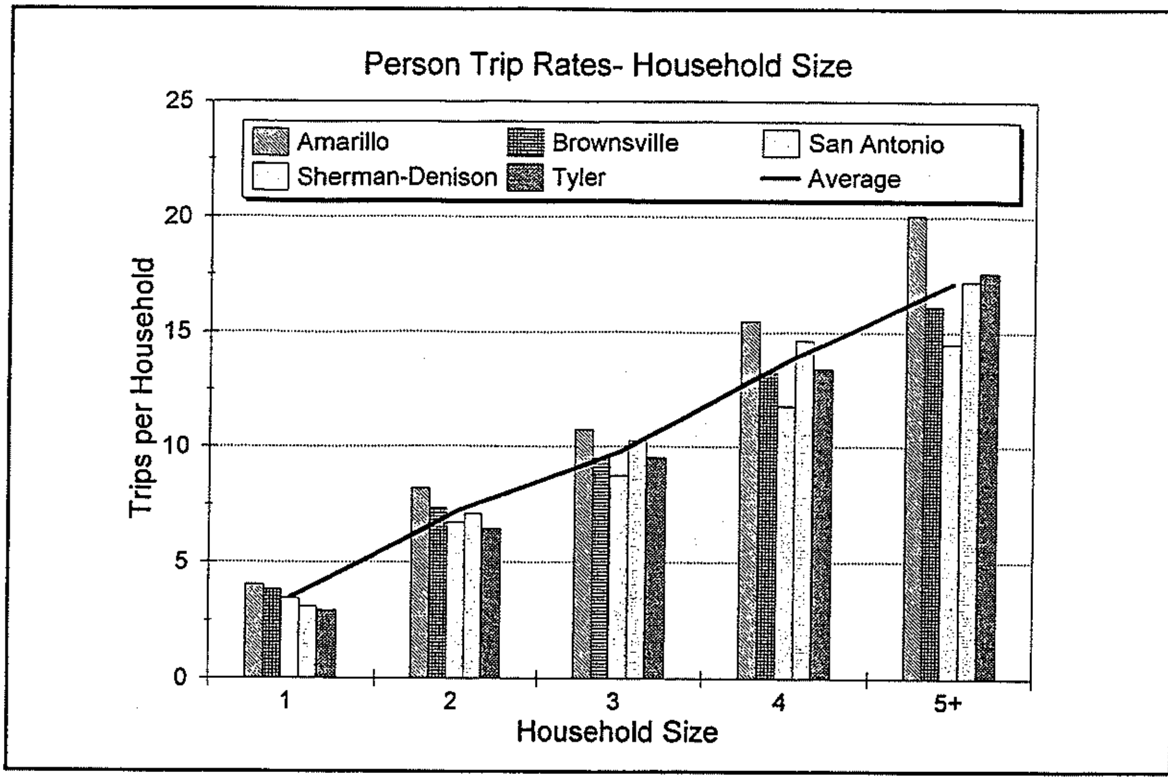

FIGURE 3. TRIP RATES FOR FIVE URBAN AREAS STRATIFIED BY HOUSEHOLD SIZE.

Z-test

Performing the z-test on all seven stratifications showed there was a significant difference for each level of the one-way and two-way stratifications. If the calculated z-value was less than -1.96 or greater than 1.96, the difference in the levels was considered significant. There were a few cells in the three-way stratification that did not show a significant difference. These were mostly between the 25 to 44 and the 45 to 64 age groups and the $20,000 to $34,999 and the $35,000+ income levels. Other levels of household size were not found to be significantly different because there were not enough observations at the lowest and highest ends of the age groups and income groups. This was due to the fact that the travel surveys were conducted to have statistically accurate samples using a two-way stratification; and when these households were spread out through three variables, some of cells of the three-way stratification did not have enough households to have a statistically accurate sample.

Coefficients of Variation

The coefficient of variation for each trip rate was calculated to determine how much variation was present between the households that fell within the same category. Figure 4 shows that between the one-way stratifications, household size had the least variation within the trip rates for each level. Out of the two-way stratifications, age of the head of household versus household size had the largest percentage of low coefficients of variation, and therefore, had the least variation of the two-way stratifications (Figure 5). The three-way stratification had a large number of high coefficients of variation.

Percent Error

The percent error of the mean trip rates for each stratification was also calculated. This value shows the range in which a trip rate would fall 95 percent of the time if the study were to be repeated. This number represents the reliability of the mean trip rate that was calculated. Table 1 shows the high and low percent errors for each person trip purpose and each auto-driver trip purpose in the urban area. Of the one-way stratifications, age of the head of household had the lowest percent errors. Age of the head of household versus household size generally had the same range of percent errors as age of the head of household versus income. The percent errors were higher for household size versus income. The percent errors were high in the three-way stratification due to a lack of observations within the cells.

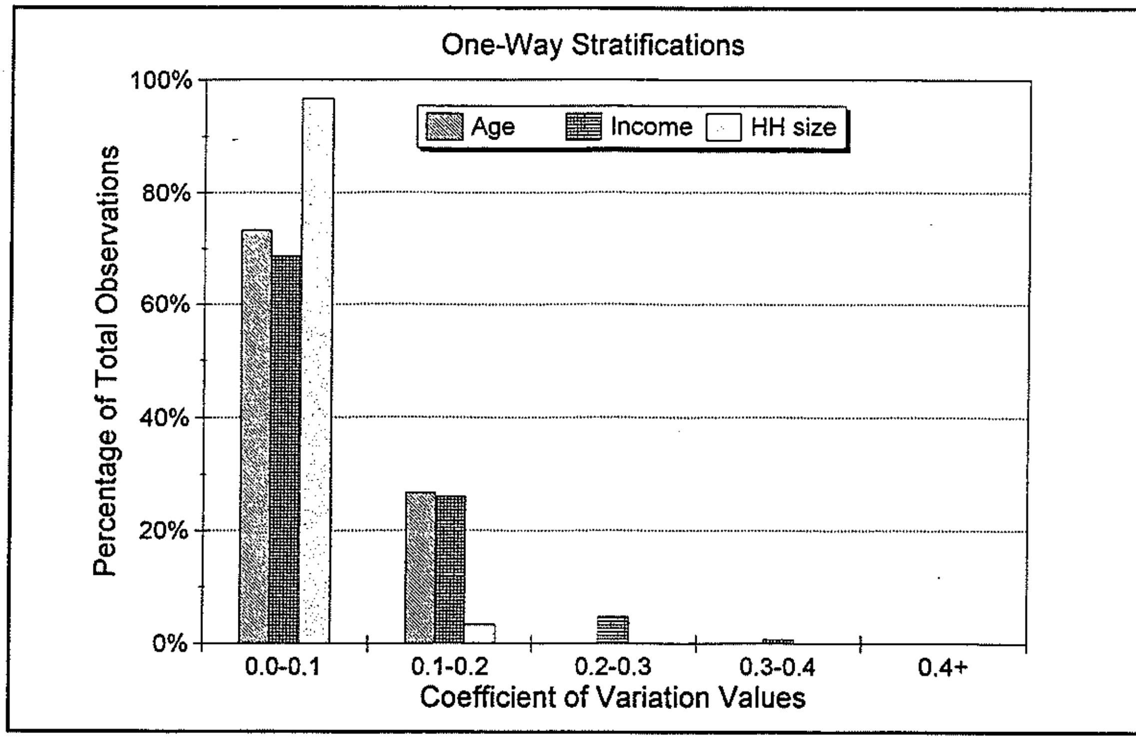

FIGURE 4. DISTRIBUTION OF COEFFICIENTS OF VARIATION FOR ALL ONE-WAY STRATIFICATIONS.

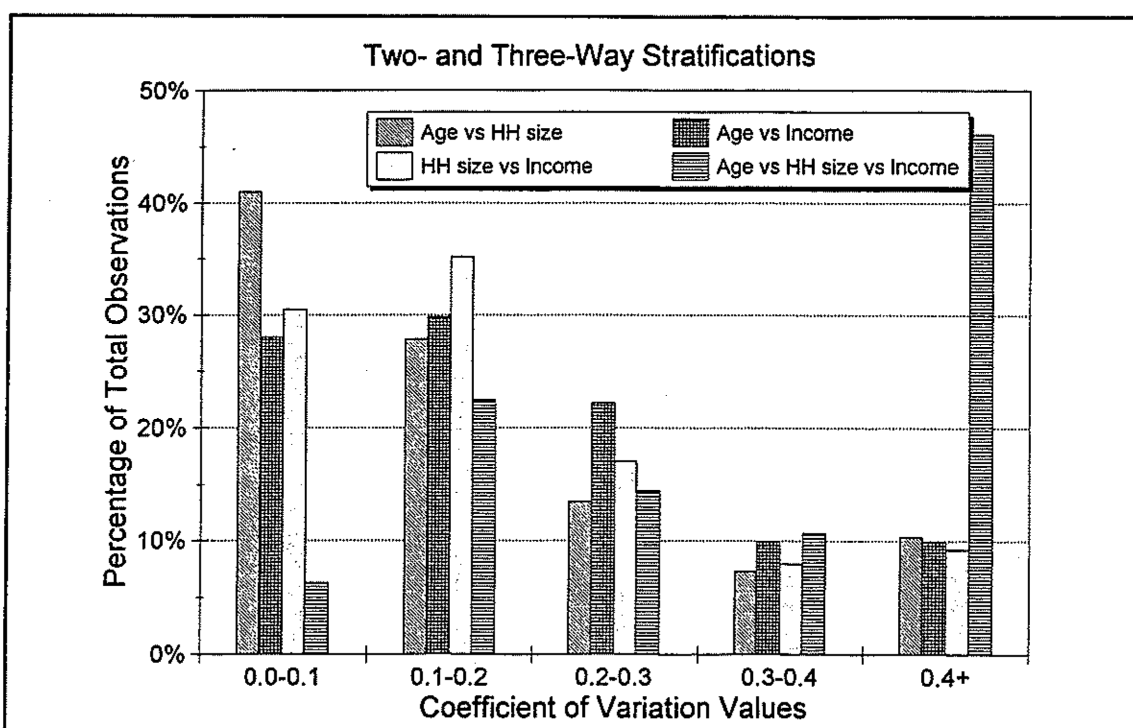

FIGURE 5. DISTRIBUTION OF COEFFICIENTS OF VARIATION FOR ALL TWO- AND THREE-WAY STRATIFICATIONS.

TABLE 1. RANGE OF PERCENT ERRORS FOR PERSON TRIP RATES.

	HBW		HBNW		NHB	
Stratification	Low	High	Low	High	Low	High
Age	6.3	8.7	7.8	11.2	6.5	8.8
Income	7.2	11.0	6.9	12.8	7.5	13.6
HH size	9.3	13.5	1.3	11.6	9.5	15.9
Age vs. HH size	11.1	15.1	13.0	19.3	13.9	19.7
Age vs. Income	12.5	22.1	11.8	21.9	14.0	26.5
HH size vs. Income	16.0	27.8	15.1	26.1	18.1	30.1

TABLE 2. RANGE OF PERCENT ERRORS FOR AUTO-DRIVER TRIP RATES.

	HBW		HBNW		NHB	
Stratification	Low	High	Low	High	Low	High
Age	6.3	9.8	6.8	11.8	4.1	11.9
Income	9.1	13.2	8.1	14.7	9.2	15.1
HH size	7.2	9.9	7.5	12.1	7.5	11.3
Age vs. HH size	11.9	19.5	11.6	20.4	13.8	23.9
Age vs. Income	15.5	26.2	15.0	26.7	17.9	29.7
HH size vs. Income	12.9	19.6	13.4	21.5	15.3	24.7

CONCLUSIONS

Because household size had the lowest variations in trip rates and low error, household size should be used if trip rates are to be stratified one-way. Stratifying trip rates by more than one variable is commonly used, however, to increase the sensitivity of the model to change within the urban area. Therefore, it is better to use at least a two-way stratification of variables for determining trip rates.

This study has shown that using one life-cycle variable may have some benefits in more accurate trip rates for use in planning models when stratified with other variables such as income and household size. The stratification of age of the head of household with household size showed lower variation in the trips within each category and a low percent error range. Because trip rates can be more accurately determined when stratified in this manner, some improvements may be seen if this variable is used.

The three-way stratification of age of the head of household and income and household size examined in this study showed that there was a large amount of variation in household trips for most of the cells and that there was a high error range for the calculated mean trip rates. This does not necessarily mean that it would not be beneficial to use a three-way stratification of these variables for estimating trip rates but that the surveys used in this study were not large enough to provide each cell of the three-way stratification with a statistically accurate sample. If a larger sample for each cell was provided, the variation and error might drop to a level where it would be beneficial to use this stratification.

RECOMMENDATIONS

Though there was not a dramatic improvement in trip rate calculations when using a life-cycle variable, marginal improvement may be obtained. However, to use a life-cycle variable it must be projected into the future. Because it is not known at this time how to predict households by age of the head of household, this would be the next step in determining if life-cycle should be used as a variable for calculating trip rates for trip generation models.

When travel surveys are conducted again, enough households should be surveyed to have a statistically accurate sample for each cell of the three-way stratification: age of the head of household, income, and household size. The other aspects of the life-cycle variable such as number of children and/or adults in the household, number of licensed drivers and the number of persons employed in the household should also be examined to determine if one or more of these variables could improve the calculation of trip rates for use in trip generation models.

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An Analysis of the Effects of Lead Length On Inductive Loop Detectors

JUENE K. FRANKLIN

This report presents the results of a group of tests on the accuracy of inductive loop detectors (ILD) in speed determination. The effect of lead length on the detection capability of the ILD's is also explored.

The accuracy of the ILD's in speed determination at low velocities (20 mph and 40 mph) were reasonable during our test runs. The accuracy of the ILD's in speed determination was fairly low for a velocity of 60 mph. The type of error found for each test run at a particular speed was systematic. A correction factor could be created to eliminate the inaccuracies found in speed measurements for each speed. These correction factors could be set up in graphical form for quick and easy use.

An inductor (black) box is used for the discovery of the optimal amount of lead that would allow detection. The optimal lead length is found to vary with each detector type in this study. The limiting range was from 2950' to nearly 9000'.

INTRODUCTION

Purpose

Inductive loop detectors have been a significant factor in vehicular detection for about 30 years. Loop detectors are used in traffic-actuated signal control, warning signs, and other monitoring systems. ILD's can effectively determine speed for vehicles and improvement of flow on highway systems, if the inaccuracies of the measurements are minimized.

ILD's are the most popular form of detection device in use today. In the Summer of 1992, experimental data showed that ILD's are less reliable at speeds of 50 mph and above. A method of reducing the measurement errors in velocity calculations at higher speeds is necessary for the ILD's to be effective in highway demand measurement and speed calculations.

The effect that the length of lead wire from the detector to the area of sensitivity (optimal lead length) is also explored. The limiting lead length is important because the ILD will not detect vehicles consistently in the area of sensitivity if the lead length is excessive.

Objectives

1. Determine if the error in speed determination using two loop detectors is a constant or if it has a random component. If a random component exists, what is the magnitude of the random component.

2. Determine the speed for a design vehicle using three different trap lengths and compare that value with the radar estimated speed.

3. Evaluate the influence of lead length on the detection capability of several detectors.

BACKGROUND

How Detection Occurs

An Inductive Loop Detector system includes a detector unit, lead-in cable and the loop wire. The detector unit sends a current through the loop wire when it is connected to a power source. The current flows through the wire at a certain frequency causing a magnetic field to form in the area of sensitivity. A vehicle with any closed electrical circuit crossing the area of sensitivity, draws energy from the loop, thus changing the frequency of the loop, if the flux lines pass through the closed electrical circuit of the vehicles. This frequency change, if sufficiently large, causes the detector unit to activate. An example of a typical loop configuration is shown in Figure 1.

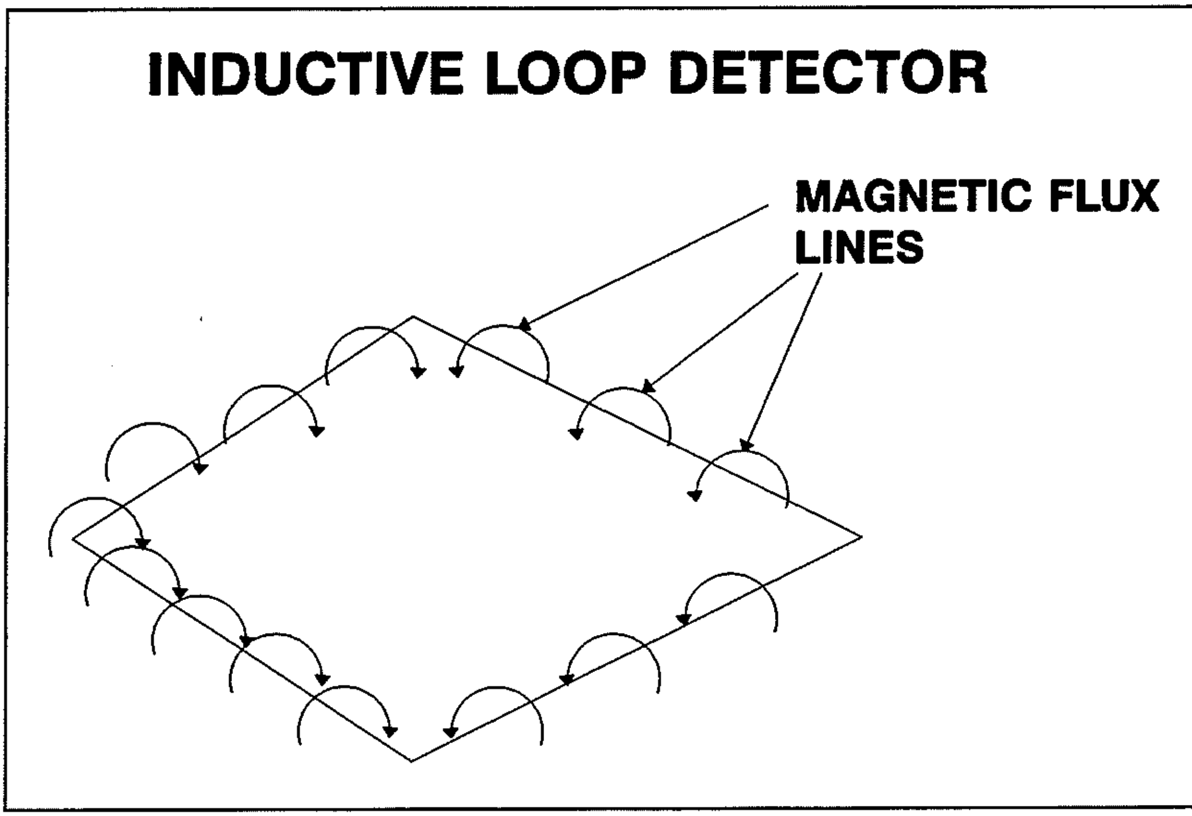

FIGURE 1. TYPICAL LOOP DETECTOR FLUX PATTERNS.

Measurement of Optimal Lead Length

The optimal lead length is the maximum amount of lead-wire that a detector can have and still detect the presence of a vehicle in the area of sensitivity. An inductor box in series with the loop detection system simulates the lead-wire. A given inductance has an equivalent length associated with it. Twenty-three microhenries is added for each 100 ft. (30.48 m) of lead-wire. Table 1 shows the limiting inductance values for the detector units used in the experiment and their equivalent lead lengths. Tests with different, actual lead-wire are scheduled some time this fall.

It can be seen from Table 1 that the amount of lead wire can vary with the type of detector unit in use. The variation for the detector tested is between 2957-8696+ft (900-2650+ m).

Speed Determination

Speed determination using inductive loop detectors is performed by forming a speed trap. Three different trap distances were used: 10 feet, 50 feet, 80 feet. A strip chart recorder is linked into the detector system, so there is a trace of the loop occupancy. This can be used to estimate the time of the vehicle passing through the speed trap. The time is used to estimate speed in the speed trap. These speed estimates are compared to the speeds measured by the radar gun. The error found in the comparison is examined to determine whether it is a systematic error or a random error. Figures 2 and 3 show the basic layout of the speed trap.

Experimental Equipment and Procedure

The lead length is explored in the first experiment using the inductive loop detectors. A loop is chosen and wired in series with an inductor box. The reason for the use of the inductor box is to give an accurate determination of equivalent wire length. Once the inductor box is hooked into the circuit, the detector unit itself is plugged into the 110 volt power source. A frequency tester is used to measure the frequency of the tuned RF circuit, so that one can know if the detector is working properly. It is important to measure the frequency, because the detector will not detect if the circuit is open.

The loop is setup and the detector unit reset. It is important to reset the detector unit because erratic signals can be emitted through it because of lead movement or

TABLE 1. EQUIVALENT MAXIMUM LEAD WIRE FEET AND METERS

Detector Unit	Inductance (μh)	Equivalent Length ft(m)
Sarasota 515TX	2000	8696 (2650)
Sarasota 535B	1000	4348 (1325)
Detector Systems 102S	1100	4783 (1460)
Detector Systems 103S	680	2957 (900)

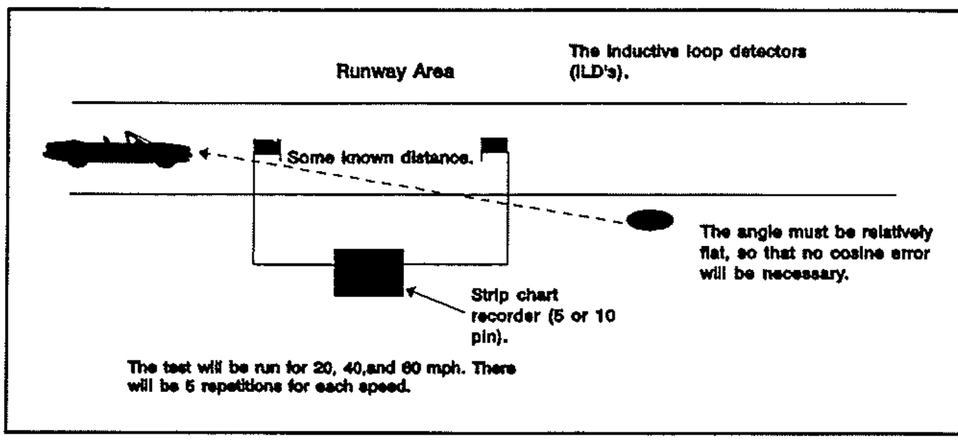

FIGURE 2. SPEED TRAP LAYOUT.

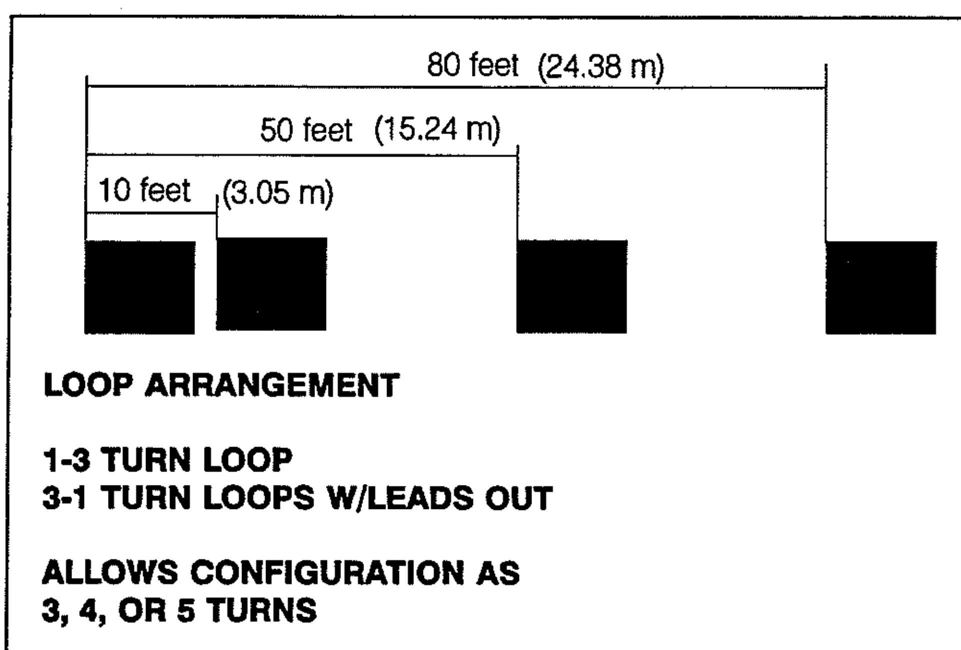

FIGURE 3. THE THREE TRAP LENGTHS USED IN RESEARCH.

unsoldered splices. This results in a detector error. If the detector unit does not reset properly, it can be another indication that there is an open circuit.

Once the loop system is properly setup, a Plymouth Reliant makes a run across the loop with a given inductance in the system. After five runs each of 20, 40, and 60 mph, the inductance is increased and the Reliant is run through the area of sensitivity the same number of times for each speed after the detector unit is reset. This procedure is repeated until the detector unit can no longer be reset or the detector unit fails to sense the vehicle as it passes through the loop. The inductance is recorded and the equivalent length is calculated for each detector unit (See Table 1 inductances and equivalent lengths).

In the second experiment, an investigation of the capability of inductive loop detectors to accurately determine speed is performed. The loops in this speed trap are setup at known distances apart, so that the time it takes for the vehicle to travel from one loop to another can be used for speed estimation. The loop wire is linked directly to a device called a strip chart recorder, while the detectors for both loops are in place. The strip chart recorder paper has lines that represent increments of time (for this experiment the increments represented 1/20 of a second). The speeds that are calculated from this experiment are compared to the true speeds (radar speed), so that an error, if any exists, can be determined.

In both experiments, it is important to note that the radar gun was used to acquire true speeds for comparison, as well as, documentation.

Experimental Shortcomings of Equipment and Data

The accuracy of this experiment was influenced by certain variables that could not be controlled. In the speed trap, the Sarasota loop detectors would not give a stable signal when the car passed through the sensitivity zone. At times, the wind would blow moving the lead wire and causing detector activation. The Detector Systems loop detectors gave the most stable signal during our testing. The limited amount of testing against actual lead wire runs that were performed did show that the inductor box was reasonably accurate. A device called a strip chart recorder was used to record the detector "on" and "off" data. There was a problem with some of the strip chart recorder readings, in that, the signals received were not as crisp as desirable. There was some cross-talk occurring between the loops in the ten foot speed trap.

The radar gun used to calculate some of the velocities needs a correction factor (.9855) multiplied into its results to attain 'true speeds.' Any radar gun used will be accurate only to $\pm$ 1 mph.

RESULTS

Speed Determination

The information gathered from the test runs are very interesting. Based on past research information, it was known that an increase in speed would cause inaccuracies in the speed estimations to increase. The distance between the inductive loop detectors in the speed trap was found to influence the accuracy of the speed measurements. It was discovered that the greater the distance between the loops in a speed trap the greater the accuracy of the speed measurement. Figures 4 and 5 are two sets of confidence intervals obtained for each speed in a 50' (15.24 m) and a 80' (24.38 m) speed trap.

It is apparent that the confidence intervals shown on the chart with the 80' (24.38 m) speed trap was not as inaccurate as the confidence interval of the 50' (15.24 m) speed trap. Yet, the information obtained in these graphs show that there is a high level of error that is present in both speed trap calculations. Although the information had error, the values were consistent. With the level of consistency present being so high, it is apparent that the error experienced is systematic. With systematic error it is possible to set a correction factor that will compensate for the lack of accuracy.

Lead Length Study

It was assumed in the preliminary report that the lead length could cause problems with the detectors sensing objects in the area of sensitivity. The results gathered in the experiment proved this assumption to be true. According to the information that was found in Table 1 of this report, it can be seen that the lead length of a loop detection system can vary from one detector to another. The results show that the lead length could be anywhere between 2900-8700+ ft. for the four detectors tested. Speed did not seem to play a crucial role in whether or not the detectors sensed the presence of a vehicle. The detectors sensed the test vehicle passing through the area of detection consistently at speeds of 20 mph to 60 mph until the limiting condition was reached.

Another important discovery concerned the ratio of the loop inductance with the system inductance. It had been logically assumed that this ratio had to be .35 or higher for the detector to function well. According to the information listed below this was not necessarily true:

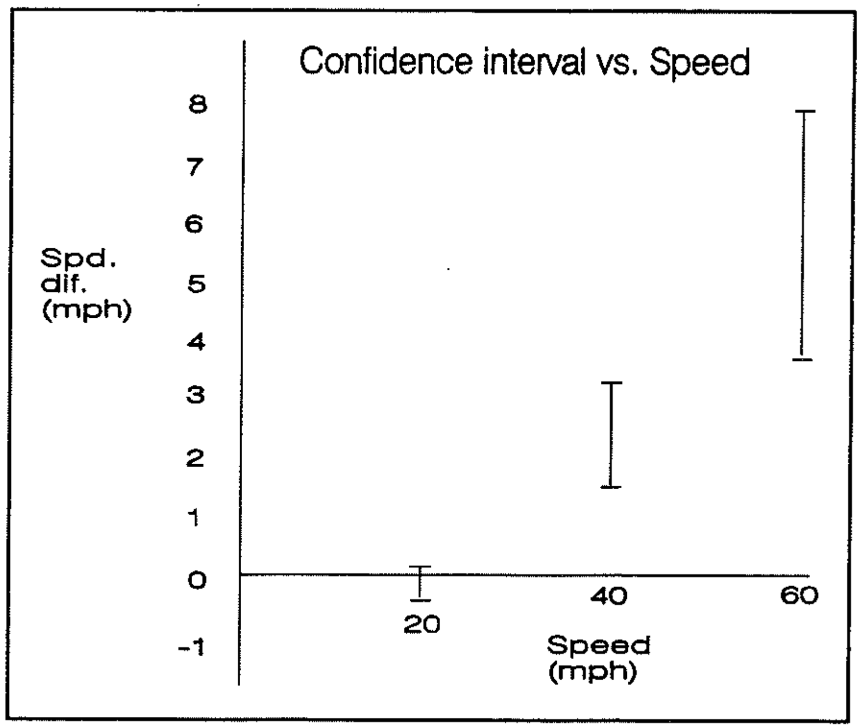

FIGURE 4. 50' (15.24 m) SPEED TRAP.

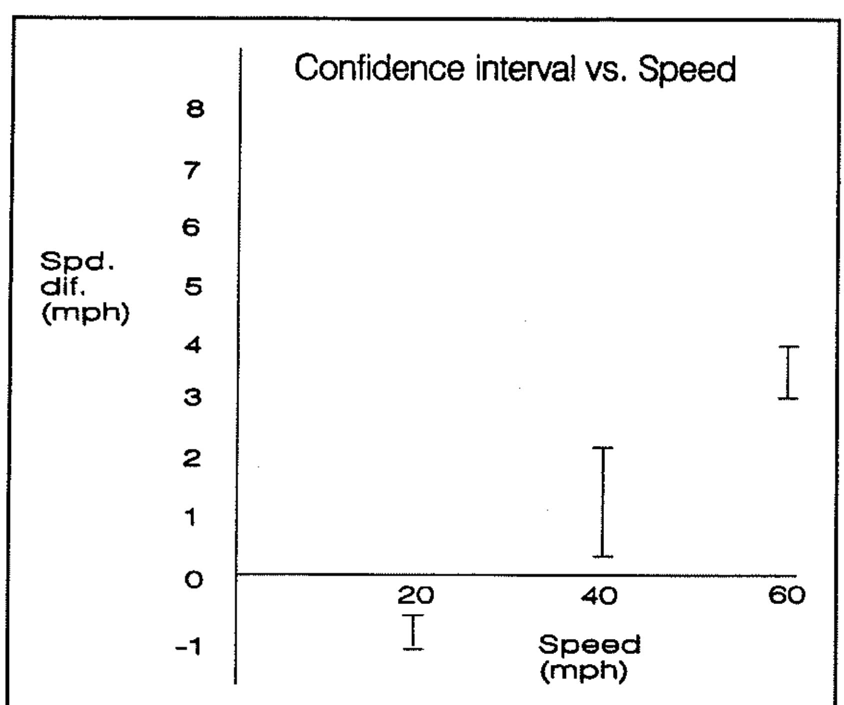

FIGURE 5. 80' (24.38 m) SPEED TRAP.

MEASURED LOOP/SYSTEM RATIO

SARASOTA 515TX	.065
SARASOTA 535B	.122
DETECTOR SYSTEMS	.112
DETECTOR SYSTEMS	.169

These results were quite shocking because it is unbelievable to imagine that the detectors would be capable of detection with ratios this small.

SUMMARY OF FINDINGS /RECOMMENDATIONS

Speed Determination

The results obtained through this research proves that the error found is systematic. For this reason, a correction factor is recommended to alleviate any error that may be encountered because of trap length or speed. Figure 6 is an example of a graph of correction factors that should minimize any error incurred from trap length or speed.

The graph shown in Figure 6 can be used to plot values to be added or subtracted from the calculated. This graph can be expanded to accommodate higher speeds or longer trap lengths. The mean values were taken from the confidence intervals and used to construct this graph. The values obtained from this graph of correction factors should bring the values calculated from the inductive loop detectors in the speed trap to within ± 1 mph of the "true" speed.

Lead Length Study

The information obtained from this particular study show that more in-depth research is necessary to confirm the findings mentioned in the results sections of this report. Instead of using an inductor box, the actual lead wire should be implemented to strengthen our equivalent length findings. The assumption of a 0.35 ratio of loop inductance to system inductance was shown to have questionable validity. Again, more research should be performed, so that a more viable inductance ratio can be discovered, if necessary.

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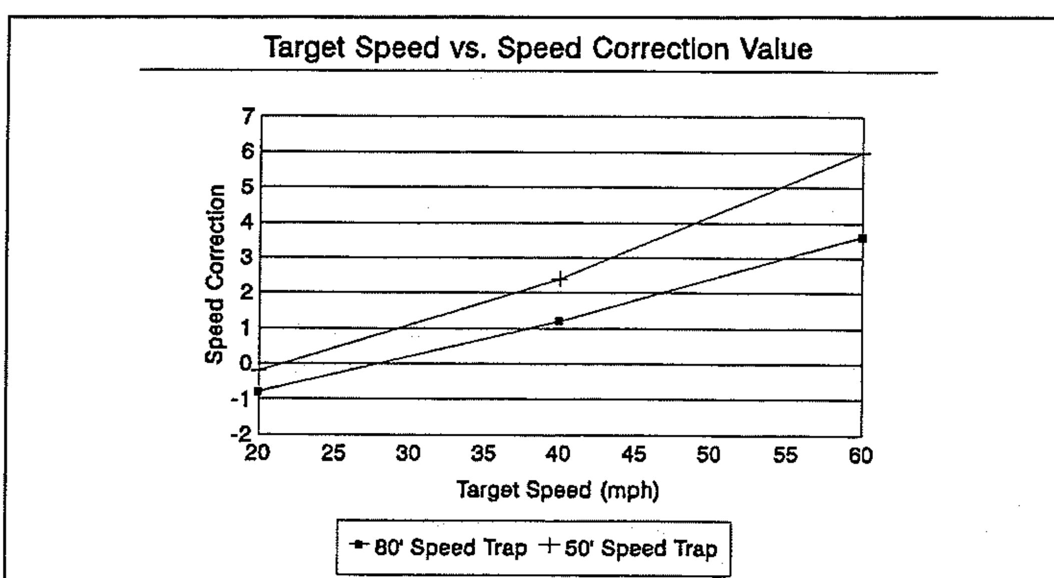

FIGURE 6. SPEED CORRECTION FACTORS

The Effects of Fiber and Polymer Modifications On An Asphalt Pavement's Tensile Fatigue Life

BRIDGETTE D. KELLER

With an ever growing number of road users, efforts must be made to develop a type of asphalt pavement which has a high resistance to permanent deformation and plastic flow. A Stone Mastic Asphalt (SMA) mixture has been developed that creates a stone to stone matrix that carries the load of an automobile wheel most effectively. This grain to grain contact achieved in the SMA helps the pavement resist rutting and lends itself to excellent stability. However, in these SMA mixtures, the asphalt cement drains away from the aggregate requiring an additive to help prevent this. Therefore, fibers and polymers have been added to the SMA type mixtures.

While laboratory and field testing has been conducted on polymer modified stone mastic asphalt type mixtures, no documented data exists regarding the effects fiber modification has on an asphalt pavement's resistance to permanent deformation and plastic flow. This paper presents the results of laboratory testing which investigates how these fibers (cellulose, mineral, and synthetic) and a recycled polymer affect the tensile fatigue life of a stone mastic type pavement. This research includes comparisons of healing abilities, energy necessary for crack propagation, relaxation moduli, and the number of cycles required to reach failure in high strain testing. The tests applied to the modified stone mastic closely mimic the loadings to which surface pavement such as this would be exposed.

INTRODUCTION

In a society so dependent upon transportation, especially by way of roadways, efforts must be made to continuously upgrade the conditions of the various modes of transportation. In the pavement materials division of transportation, innovative materials as well as innovative methods are being incorporated with traditional methods in order to lengthen the life of roadways. One such idea is Stone Mastic Asphalt (SMA).

Stone Mastic Asphalt mixtures were first used in Europe twenty years ago to combat the wearing caused by studded snow tires. However, in this type of mixture, the asphalt cement drains away from the aggregate and requires an additive to help bind the asphalt to the aggregate. For this reason, fibers were introduced to the asphalt mixture. The fibers most commonly used are mineral and cellulose, and some polyester fibers have been recently used. The cellulose fibers generally originate from recycled newspapers, while the mineral fibers are obtained from glass products (1).

It is a fact that the fibers help hold the asphalt to the aggregate and allow the use of higher asphalt contents in mixtures which improves mixture durability. However, there is no documented evidence of the effect the fibers have on the pavement's resistance to permanent deformation or fatigue damage. For an asphalt to have a long life with a low cost maintenance history, it must meet certain requirements. There must be a high resistance to plastic flow and consolidation over the life of the pavement so that the pavement does not rut prematurely due to heavy traffic. Along with this resistance to rutting, there must also be a resistance to fracture and abrasion to maintain a smooth riding surface. Exposure to moisture can also create problems in asphalt pavements; therefore, the pavement should be manufactured to retain a high strength after repeated exposure to environmental factors including aging due to ultra-violet radiation and atmospheric oxygen. A high quality asphalt pavement should also have a low abrasion number with very little loss of the fine aggregates used in its manufacturing. The surface should also combat the damages caused by low temperature cracking as well as have a resistance to oil and fuel spills. Along with these other factors, an asphalt pavement must also be well blended so that it is a homogeneous material (2).

PROBLEM STATEMENT

The reliability of roadways falls at an exponential rate as the pavement is exposed to high volumes of traffic. Engineers are continuously looking for ways to lengthen the life of roadways with little increase in construction cost. With the constant growth in population and continued increase in the number of registered vehicles, transportation agencies are forced to update techniques to meet the demands of the road users through improvements in highway materials such that the life of the road way is lengthened.

It has been proven that an SMA mixture containing fiber modified asphalt performs admirably with low rutting potential due to the stone matrix developed by the gap gradation and the fracture resistance and durability due to the ability to carry high asphalt contents. However, there has previously been no documented data on the effect these fiber additives have on the pavement's resistance to permanent deformation or its performance. On the other hand, mixtures with low density polyethylene (LDPE), or polymer, modification have demonstrated excellent performance based on both field and lab data. Fibers carry the asphalt cement most effectively; so much so that they create a demand for more asphalt. This demand creates an unknown effect in the asphalt pavement which this research explored.

OBJECTIVE

Both the fibers and polymers that are added to the asphalt mixtures create a demand for higher asphalt cement contents in the mixture. Because of this demand for higher asphalt contents, it a question was brought forward as to the role the fibers play in an asphalt mixture. Do they, in fact, act as a unit to help hold the asphalt to the aggregate or do they simply act as an odd shaped aggregate requiring their own asphaltic coating? The objective of this research was to determine the effects the introduction of fibers to an SMA type asphalt mixture has on the mixture's resistance to permanent deformation and plastic flow and to compare these results to the same type data obtained from mixtures containing polymer modified asphalt cement. More specifically, this research evaluated how the fiber and polymer modifications affect the tensile fatigue and shear deformation life of an SMA type mixture.

BACKGROUND

In 1990, nearly 370 tons of the cellulose fibers were used in fiber-modified mixtures. This is the equivalent of 100,000 tons of modified asphalt mixtures. The asphalt mixtures which comprised this group were stone mastic asphalt concrete, gap-graded asphalt concrete (GAC), and the traditional asphalt concrete (AC). GAC and SMA mixtures contain gap-graded aggregates. This means that there is a large amount of mineral filler and aggregate retained above the #10 sieve used in the mixture. The gradation of a gap graded aggregate compared to that of a traditional aggregate can be seen in Figure 1. As seen in this gradation curve, the gap graded aggregate consists of very little sand which passed the #10 sieve and was retained above the #200 sieve. On the other hand, the traditional gradation is comprised of a uniform distribution of the various sizes of aggregate. In this project, SMA was the specified asphalt mixture. The objective of the SMA mixture is to develop a stone to stone skeleton to transfer the load from the wheel of an automobile throughout the pavement. The large stones should interlock leaving the mastic (asphalt cement and fines) to fill the voids that are created by this interlocking. An important consideration in the production of an SMA is that as much mastic as possible needs to be pumped in between the larger stones while maintaining the proper percentage of air voids (3% to 4%). Fibers or polymers are usually added to the mixture to aid in holding the asphalt cement to the mineral filler and coarse aggregate (especially during lay down) while not interfering with the desired stone to stone contact (3).

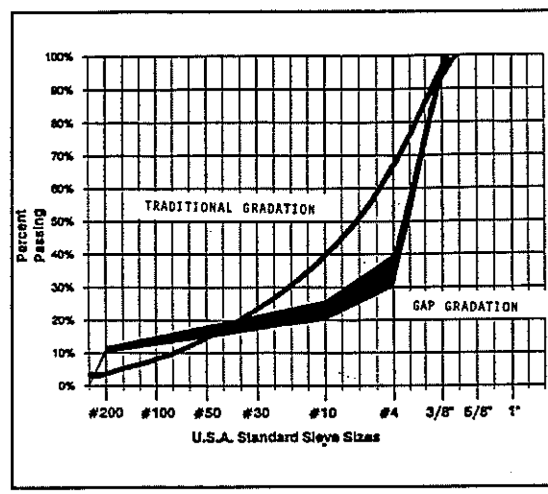

FIGURE 1. GRADATION COMPARISON.

Stone Mastic Asphalt

In this research project, fibers were added to stone mastic asphalt mixtures. An SMA is a mixture that was developed in Europe approximately twenty years ago to combat the wearing effect of studded tires. It was this need for high performance wearing courses that led to the development of an asphalt pavement which has great load carrying capability through the stone matrix. The high internal friction of an SMA mixture means excellent stability and a high resistance to wearing (3).

The SMA mixtures require a stone to stone skeleton with binder (mastic) holding these stones together. This mastic is generally composed of asphalt cement, sand and aggregate fines. Generally, a traditional aggregate gradation along with a traditional quantity of asphalt will not result in a desirable pavement. Therefore, fibers are added to the mix to reinforce the bond between the aggregate and the asphalt cement, thereby becoming a part of the mastic.

During production of the fiber-modified mixtures, the mix time increases approximately 5 to 10 seconds after the introduction of the asphalt (5). As with any asphalt mixture, the SMA mixtures should only be made with high quality materials.

Fibers

Generally, the fibers are added either in separate bags in proportions of 0.3% to 0.5% or as granules in proportions of 0.6% to 0.8% by weight of the asphaltic mixture. More granules are required because they are pellets of fibers held together by asphalt cement. In this project, the fibers will be added in separate bags in the proportions that follow based on the total weight of the asphalt mixture:

1) Cellulose 0.3% by weight of the mixture,
2) Mineral 0.5% by weight of the mixture, and
3) Polyester 0.5% by weight of the mixture.

It may be noted that of the fiber-modified mixtures, cellulose fibers require a lower proportion than mineral or polyester fibers. This is due to the fact that the surface area of the cellulose fibers is the largest of the three. Also, the cellulose fibers immerse asphalt cement most effectively (4). In terms of the addition of the fibers compared to the volume of the aggregate, the addition is still rather low at about 0.6% of the volume of the aggregate (3).

Polymer Modified Asphalt

Polymer modified asphalt was developed for typical highway use. It is produced by blending low density polyethylene (LDPE) in a hot (310°F), high shear process with asphalt cement to create a new product. The sheared polyethylene then creates 'bundles' within the asphalt cement that act to stiffen the asphalt, increasing its viscosity, thereby increasing its resistance to plastic flow. These bundles are extremely small; therefore, they fit well into the thin film of asphalt cement that covers the aggregate face during mixing. The hot mix production, installation, and compaction of the polymer modified pavements are carried out using standard construction equipment. The enriched asphalt cement is added to a specified gradation of aggregate just as a traditional asphalt cement would be. However, this type of asphalt does require extra blending time to insure that the polyethylene bundles have been evenly distributed throughout the mixture, and the polymer modified mixture may have to be placed and compacted at higher temperatures than a traditional mix due to its aforementioned higher viscosity (2).

RESEARCH APPROACH

This research involved both visual inspection and mechanical testing. A scanning electron microscope (SEM) was used to examine a fracture face of a fiber modified specimen to determine how the fibers were distributed within the mixture. The SEM also helped to identify possible modes of failure. Based on the inspections performed with the SEM and a working knowledge of SMA type mixtures, a hypothesis of distress was developed. This hypothesis was verified through mechanical testing, specifically with an overlay test.

Test Procedures

Sample Preparation

Before testing could begin, guidelines had to be set to determine the proper mix design for the asphalt specimen to insure all things would be held constant with the exception of the addition of fibers or polymers to the mixture. Therefore, a gradation curve for the mixture had to be established to allow for continuity between the specimen. This gradation was chosen based on previous laboratory testing and can be seen in Table 1. Second, the asphalt content had to be determined based on the percentage of air voids in the mixture. The air void content was evaluated using the Bulk Specific Gravity and Rice Specific Gravity tests of samples made with varying asphalt contents. The ideal air void content is between 3% and 4%. A plot was generated to interpolate the asphalt contents at which this occurred. While air void content is important, attention was also given to the sample's stability. Therefore, the samples were tested for their Marshall Stability. The same procedure was followed in determining the amount of asphalt that could be used in both the fiber modified and polymer modified samples in order to maintain the same desired qualities present in the traditional asphalt sample (i.e. proper air void content and high stability). A different amount of asphalt will be required in the modified specimen because the fibers and polymers create a demand for more asphalt cement.

Overlay Test

The SMA type mixture is used primarily as a surface pavement. Therefore, the most likely type of distress it will be exposed to is reflective cracking in which the crack in the layer directly below the surface is

Table 1. Applied Gradation

Gradation of a Stone Mastic Asphalt		
Sieve No.	% passing	% retained
1/2	100	0
3/8	95	5
4	35	60
10	24	11
40	17	7
80	14	3
200	10	4
-200	0	10

propagated through the surface pavement. The overlay tester was chosen as the testing apparatus because it mimics this type of action.

Three specimen of each type of modification (polymer, cellulose, polyester, mineral, and a control) were tested on the overlay machine. The first specimen, a 2" x 3" x 13" beam, was exposed to a constant strain (1.5 in/in millistrain) over a period of time. During this time period, the load required to maintain this strain was recorded by means of computer software. After this stage of testing, the stress necessary to achieve the specified strain was plotted against the elapsed time to evaluate the relaxation modulus of each type of specimen.

The second and third specimen of each type were notched to simulate the beginning of a crack in the pavement and tested in the following manners. First of all, they were exposed to a low strain rate (1.5 in/in millistrain) in a cyclic loading. Here, the specimen were attached to platens which were, in turn, fastened to the overlay machine. The overlay tester then pulled the specimen, opening the crack, exposing the specimen to the specified strain rate, then compressed it back to its original position. After 50 cycles, the specimen was allowed to rest for 20 minutes. At the end of this rest period, the 50 cycles were repeated. Again the load required to obtain the specified strain was recorded. The stress was then plotted against the strain, and the area under this curve was calculated to determine the energy necessary to propagate the crack through the specimen. These two samples were then exposed to high strain (3 in/in millistrain) and cycled to failure. Plots of stress versus strain were generated again to determine the energy required to propagate the crack. The number of cycles required for the specimen to reach failure was also recorded. Failure was defined as the cycle in which the stress required to reach the maximum strain was equal to one half of the stress required to reach the maximum strain in the second cycle. The first cycle was not used as a reference due to the additional load that had to be applied to overcome the static friction between the platens and testing surface.

Hypothesis of Fiber Modification

Based on work performed with the scanning electron microscope, a hypothesis of distress was developed. When viewing the fibers interacting with the asphalt in an asphalt sample, it was determined that the fibers require their own coating of asphalt cement. This led to the belief that the fibers would act as an odd shaped aggregate. However, the fibers and the asphalt cement have different engineering properties making it difficult to achieve a homogeneous mixture. This lack of complete blending leads to clumping of the fibers within the mixture which, in turn, causes zones of fatigue making the fibers deleterious to the mixture.

Hypothesis of Polymer Modification

The polymer modified asphalt should display different qualities than the fiber modified asphalt. This is primarily due to the fact that the polymers can be evenly distributed throughout the mixture. The low density polyethylene (LDPE) used in the mixture forms bundles within the asphalt cement making it stiffer and more resistant to flow at high temperatures. Therefore, an asphalt concrete mixture containing this polymer modified asphalt cement was thought to have more resistance to permanent deformation and plastic flow.

FINDINGS

Cycles to Failure: High Strain Testing

In the high strain testing procedure, the specimen were cycled to failure, where failure was defined as the cycle in which the stress required to obtain the specified strain was equal to one half of the stress required to obtain the same strain in the second cycle. The first cycle was not used because an initial stress had to be applied to over come the static friction between the platens and testing surface. As seen in Figure 2, all of the modified specimen out performed the control specimen with the exception of one synthetic fiber sample. However, there was no statistical difference discernible among the mixtures that were tested. This would suggest that there is no apparent benefit or detriment to an asphalt mixture due to the introduction of a fiber or polymer modification.

Energy and Healing

Figure 3 is a representative plot of the stress/strain curves obtained from the low strain tests. The area under the curve and above the x-axis represents the energy necessary to propagate the crack through the specimen. The amount of energy necessary for crack growth decreases between cycle 2 and cycle 50 but increases between cycle 50 and 52 (Figure 4). This is due to the introduction of a rest period after cycle 50. This recovery type phenomenon is called healing. After the energies were calculated for cycles 50 and 52 for each of the low strain test samples, a healing index was calculated by taking the difference in energy between the two cycles and dividing that number by the energy in cycle 50. A comparison of the healing abilities of the various types of modifiers is located in Figure 5. Though there was no statistical superior "healer," the polymer modified specimen did have the highest healing index of the asphalt mixtures tested.

Relaxation Modulus

A specimen's relaxation modulus is indicative of its ability to withstand deformation after being exposed to a loading over a period of time. As seen in Figure 6, all of the specimen act the same when exposed to a load for a short period of time, approximately 5 seconds. When the load is applied for a slightly longer time, such as a truck stopped at an intersection, the polymer modified sample is able to withstand deformation associated with this load better than the other modified samples.

CONCLUSIONS

As suggested in the results obtained from both the calculation of the healing indices and the plotting of the relaxation moduli, the polymer modified asphalt has a mixture rheology more favorable to resisting high temperature permanent deformation than the other samples; however, the difference is not so great from a traditional mixture so as to induce fracture susceptibility. As expected, the introduction of a rest period into the test procedure resulted in significant healing. Because no statistical difference was discernible among the mixtures tested in the uniaxial tension procedure and because healing is so important in a pavement's fatigue life, no further testing should be conducted in this area unless the testing incorporates a rest period. Based on the number of cycles required for failure in the tension test, fiber modification poses no apparent benefit or detriment to an asphalt pavement. High temperature repeated load testing should continue on all mixtures previously tested to evaluate the resistance of the stone mastic asphalt mixtures to shear induced deformation.

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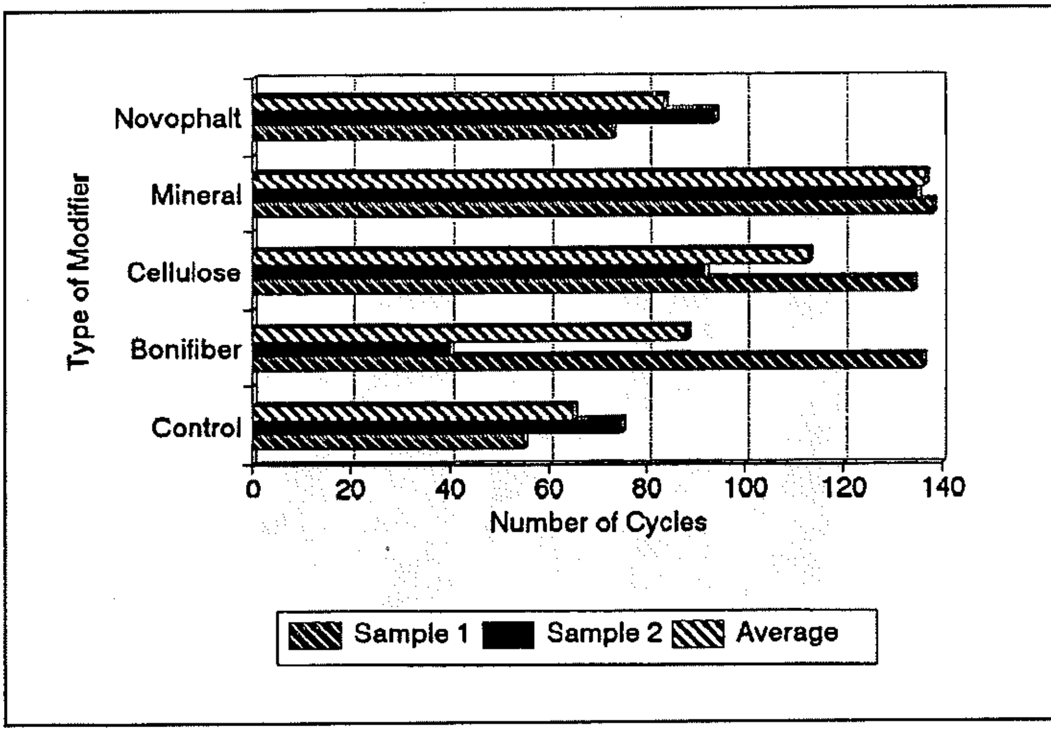


FIGURE 2. CYCLES OF FAILURE.

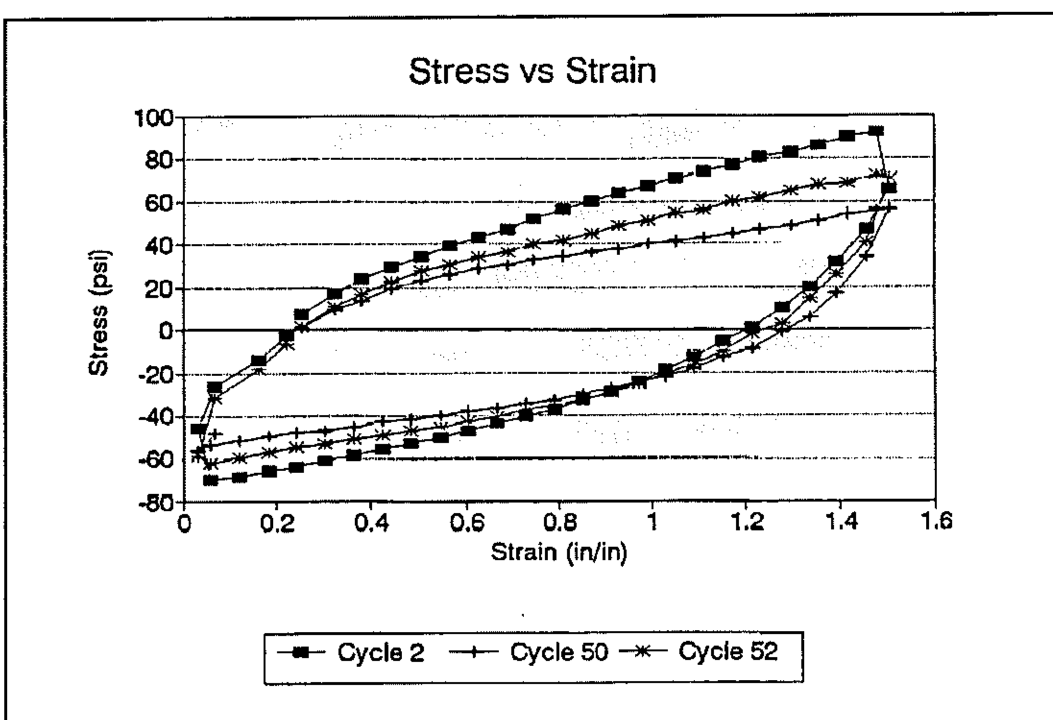


FIGURE 3. REPRESENTATIVE STRESS/STRAIN CURVE.

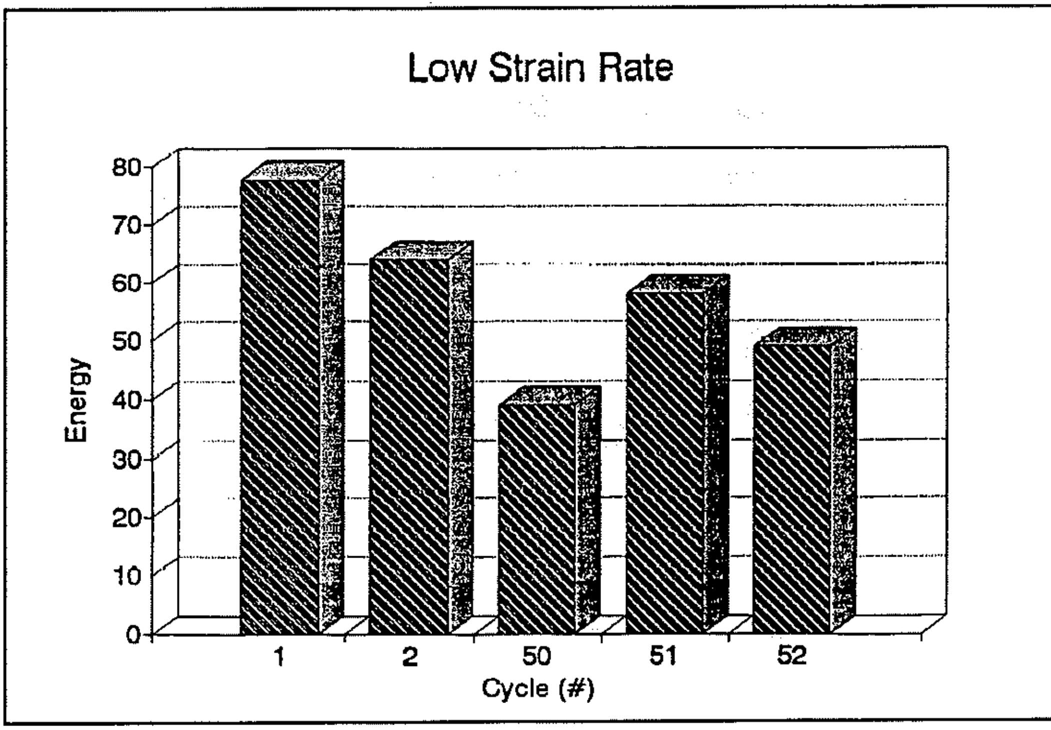


FIGURE 4. COMPARISON OF CYCLE ENERGIES.

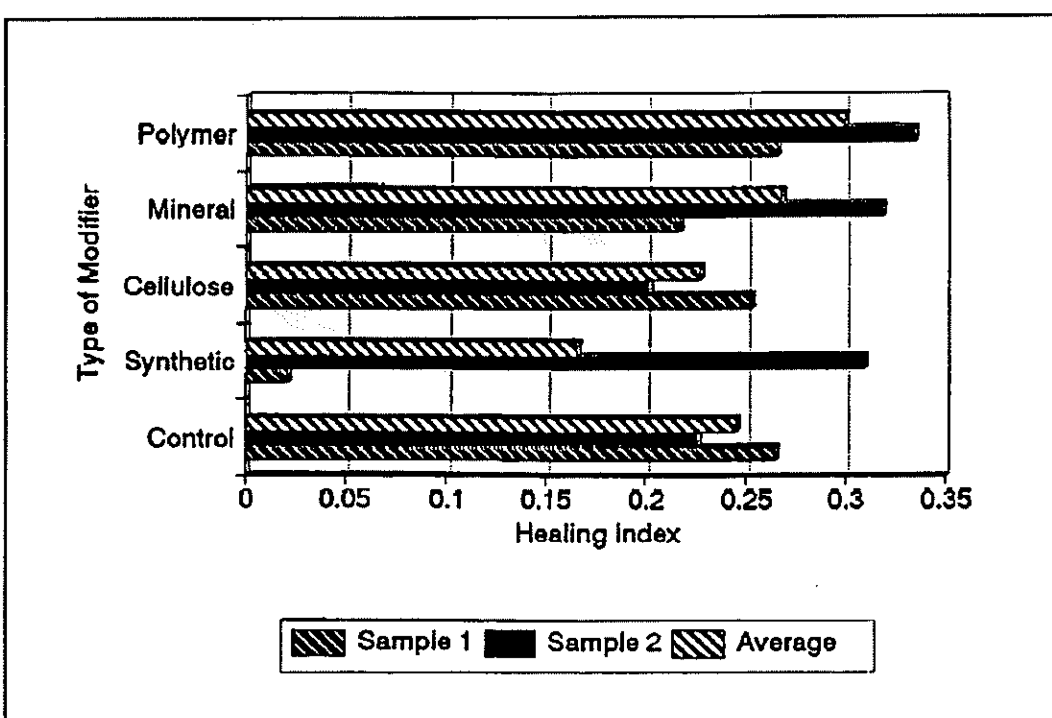


FIGURE 5. HEALING ABILITY.

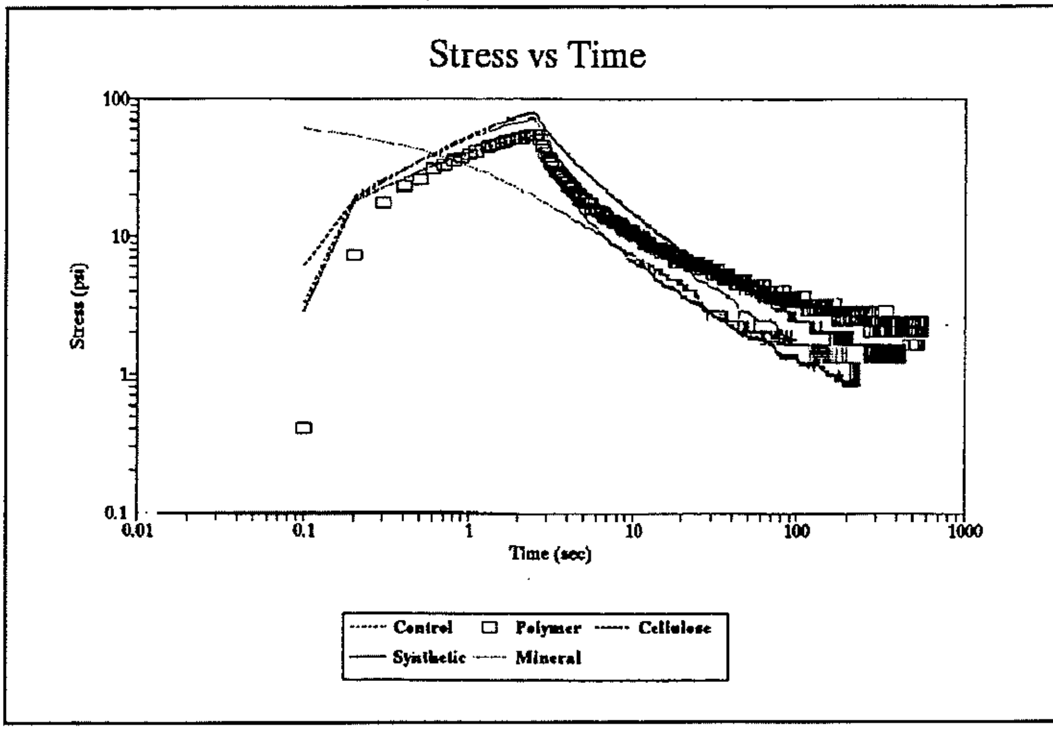


FIGURE 6. RELAXATION MODULUS.

Measuring and Evaluating Levels-of-Service of the Texas Highway System

JOSHUA E. MARTIN

A basis for evaluating the maintenance level-of-service for Texas highways is developed. A manual is produced to provide consistency in evaluating the maintenance levels-of-service. The manual covers guard fencing, illumination, traffic signals, signs, delineators, pavement markings, pavement markers, painted lines, and mail boxes. The project objective is to design a process which will allow the evaluation of the maintenance of the Texas highway system and keep it at an acceptable level with minimum funding. The intent of the data is to quantify the maintenance level-of-service of the Texas highway system. The evaluation would be conducted realistically on an annual basis. The evaluation process includes two lane undivided, four lane undivided, and four lane divided roadways, probably on a sampling basis.

In order to provide a quick and cost-effective survey, a window check process is suggested and is the only process discussed herein. The concept behind the window check survey is to allow an evaluator to determine the level-of-service of the roadway without having to exit the vehicle. It is desirable that the evaluator drive at the posted speed and evaluate the roadway. With a minimum of personnel, the Automatic Road Analyzer (ARAN) is examined for its ability to videotape the roadway at or near regular posted speeds. A manual consisting of the different level for each item anchored to color photographs and a verbal description is developed. Anyone can use it to evaluate the roadway, with minimal training.

Objective

The objective is to create a manual which will contain levels-of-service for the items assigned to this particular segment of the project. The levels of service for each item are determined and photographs representing the level made. The levels and photographs are combined to produce a manual which can be used to evaluate the roadway. The Automatic Road Analyzer (ARAN) system is studied to determine if its capabilities would be an asset to the evaluation process.

Background

Scope

The purpose of this project is to provide a manual which will create a consistent procedure for road maintenance surveys. The manual will allow a quick check of the level of maintenance of Texas roads. The project considers two lane undivided, four lane undivided, and four lane divided. The standard for Texas roads will be set using this manual. A three level system covers desirable, tolerable, and intolerable. Two levels are used where convenient. Videotaping of the roadway and necessary items is also covered. The videotape enables a person to examine the roadway in the office. Inventory of the items is not included.

Why A Manual Is Needed

Guidelines and manuals exist which govern the maintenance of Texas highways. At the present time there exists the 1980 Texas Manual on Uniform Traffic Control Devices (Tx MUTCD) for Streets and Highways. This manual covers some of the items being evaluated on the roadway, but in very detailed depth. Maintenance personnel have neither the time nor money to spend on evaluating each item on the roadway according to the depth in the Tx MUTCD. A simpler manual is needed which covers all of the items on the roadway and also accurately describes the level of service. The manual produced contains descriptions of each level-of-service and a photograph adjoining each description.

Current practice of maintaining the highway system is divided mainly into two parts. First, maintenance is performed when a certain number of people lodge complaints about that item on the roadway. Second, maintenance is performed when an accident occurs and there is damage to an item. Due to the vastness of Texas and the many miles of roads, these two steps have been the only feasible options. But with a simple manual, sections of roadway can be quickly evaluated and used to represent sections of Texas.

A manual with photographs in addition to the word descriptors was created in order to narrow down the mental image that is created using words only. If looking at the very same sign and the very same set of guidelines in words, two different people will draw up in their own mind what the picture should look like. This is a problem which has to be resolved in order to provide a uniform evaluation of all the roadways in Texas. If a certain district perceives that they are in an intolerable condition whereas they are actually at an acceptable level, they will receive funding which is not needed. A uniform evaluation is necessary to appropriately allocate the Department's resources. The photographs will narrow the mental image produced in the minds of different individuals. If an evaluator is trained to correctly decide which item is at which level, Texas will be able to proportionally divide its resources.

Effects Of Poor Quality Operations' Items

Items such as illumination, traffic signals, signs, delineators, pavement markings, pavement markers, painted lines, and mail boxes must be maintained in order to meet national legislation requirements and to provide a safe roadway for drivers. Legislation requires that items be maintained at a certain acceptable or tolerable level. It is up to each state and each maintenance division to develop manuals and maintenance procedures.

Items that are at a level less than acceptable can cause a hazard on the road. Pavement markers, pavement markings, and painted lines provide information to the driver using the road surface. If these items are below an acceptable level, a driver may become confused or misguided especially in inclement weather. Illumination below an acceptable level does not provide proper lighting of the road surface, which can be dangerous on unfamiliar roads. Signs at an intolerable level do not provide proper information to the driver. Drivers may miss a sign and get lost or try to improperly maneuver their vehicle after missing a sign.

Aesthetically, items below the acceptable level create an unsightly appearance. Signs are faded and unclear giving them a wornout look. Pavement markings are spotted and faded giving the pavement a haphazard look.

Developing The Divisions

The project is built off of the D-18 document 5-92. This document lists all of the items to be evaluated on the road system. These include pavement maintenance, roadside maintenance, operations, bridge maintenance, and ferry maintenance. The topic assigned to this research is operations. The initial topics listed for operations in the D-18 document are: safety appurtenances, illumination, traffic signals, sign and delineators, and pavement markings. These were refined to consist of guard fences, illumination, traffic signals, signs, delineators, pavement markings, pavement markers, painted lines, object markers, and mail boxes.

An elaborate five level scale is proposed in D-18, 5-92 to accurately cover any and all necessary items. Meetings with maintenance personnel revealed the general feeling that a five level scale would be too elaborate and not necessary to evaluate the necessary items. A simpler scale was developed consisting of two and three levels which creates a basic yet adequate scale for measuring the maintenance levels-of-service.

The survey was initially broken into a stop and check survey and a window check survey. The stop and check survey consisted of stopping the vehicle, exiting the vehicle, and checking the necessary items such as the torque on bolts on breakaway signs, rust on poles, etc. This process could be done, but at a large expense. Therefore, the window check process is the only concern of this project. The window check process is performed by driving along the road and evaluating the items without ever exiting or stopping the vehicle. Obviously the torque on bolts can not be measured in this manner, but due to the limited funds, these detailed items are not being considered at this time.

The next step is to determine the items to look at during the daytime evaluations. A similar list was also developed for the nighttime evaluations. Two daytime surveys were created so that too much information was not being gathered at once, thus overwhelming the evaluator. This division may not be necessary if the simpler scales are used for evaluation or if the ARAN vehicle is found to be a feasible option.

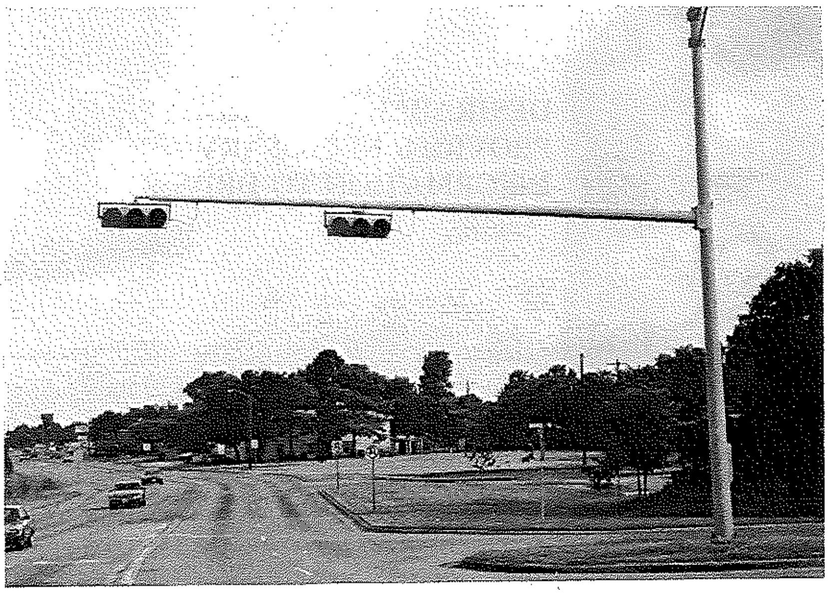
A nighttime listing is included which looks at items such as retroreflectivity and night visibility. The ARAN system has not been evaluated in its ability to analyze items during a nighttime survey. Discussions have indicated that it may not be possible to evaluate the items using technical equipment because of oncoming headlights which bloomout the video camera.

ROADSIDE SIGNS

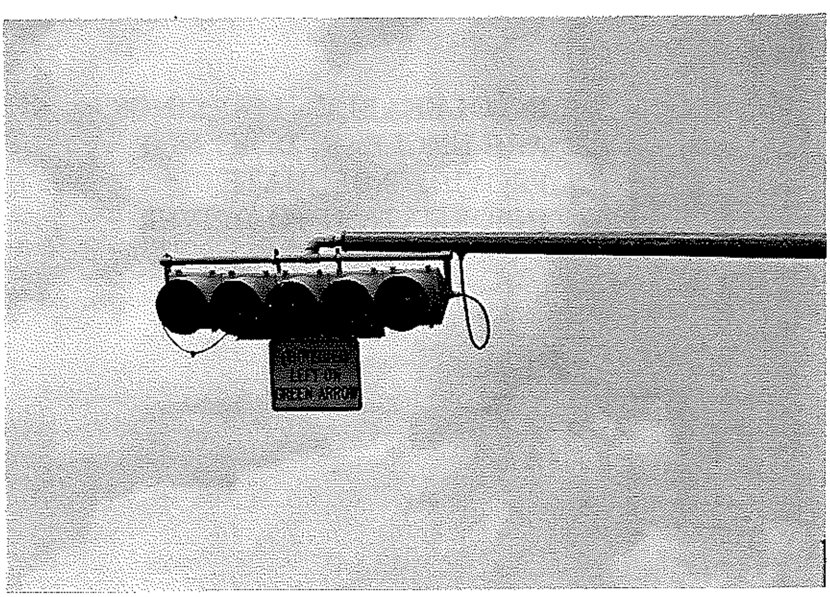
DESIRABLE - ORIGINAL COLOR, NO DAMAGED OR MISSING SIGNS, POSTS STRAIGHT

TOLERABLE - SIGNS CAN BE SEEN AT THE DISTANCE SPECIFIED IN THE MUTCD

TRAFFIC SIGNALS



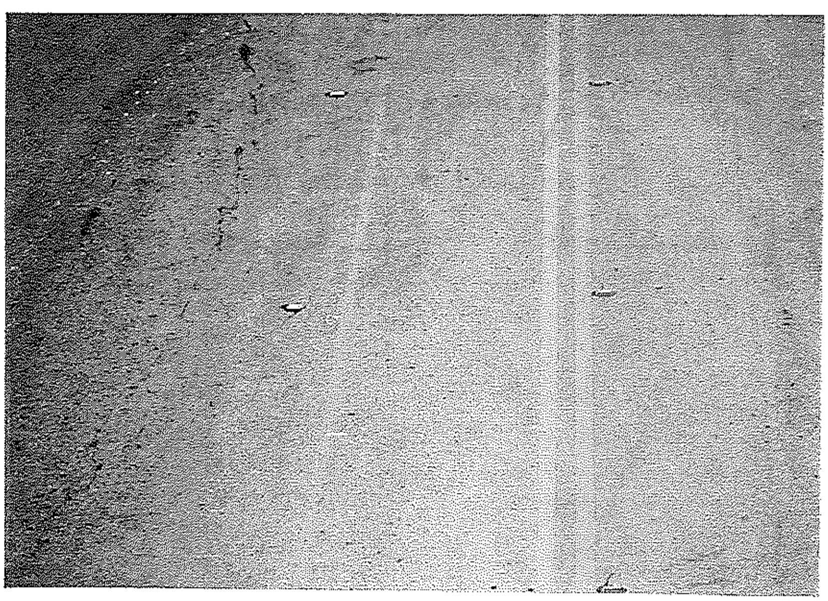
DESIRABLE - ALL BULBS WORKING



TOLERABLE - MALFUNCTION, BULB OUT, OR TREE LIMB BLOCKING DISPLAY

PAINTED LINES

DESIRABLE - CLEARLY PROVIDES NECESSARY INFORMATION TO DRIVER



INTOLERABLE - MARKINGS ARE UNCLEAR, DOES NOT PROVIDE ADEQUATE INFORMATION TO DRIVER

PAVEMENT MARKINGS

DESIRABLE - CORRECTLY CONVEYS DESIRED MESSAGE TO DRIVER

INTOLERABLE - MARKINGS ARE UNCLEAR AND DO NOT PROVIDE SUFFICIENT INFORMATION TO DRIVER

GUARD FENCES

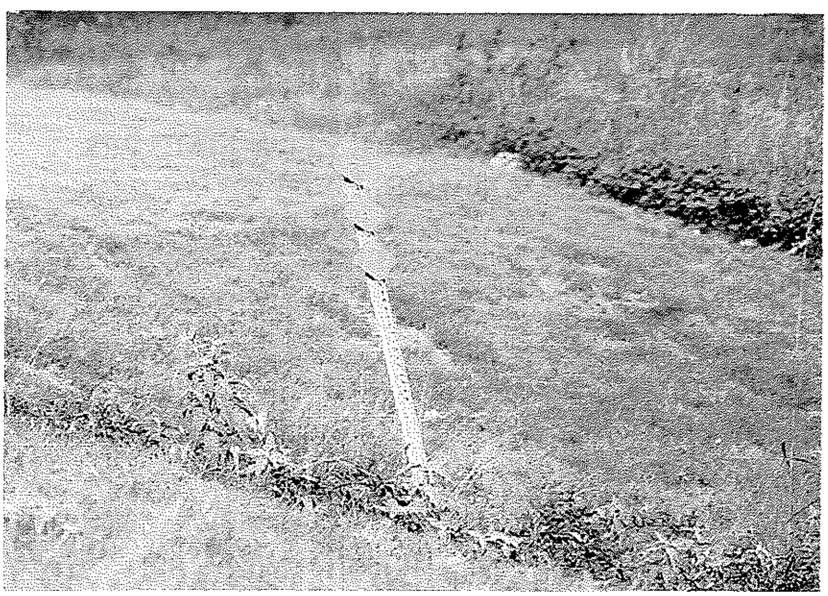
DESIRABLE - NO APPARENT GUARD FENCE DAMAGE

TOLERABLE - NO END TREATMENTS DAMAGED, BUT SOME MINOR DAMAGE IN THE RUN

INTOLERABLE - END TREATMENT DAMAGE OR MORE THAN 10% OF THE RUN DAMAGED

OBJECT MARKERS

DESIRABLE - OBJECT MARKER CORRECT AND CLEARLY VISIBLE



TOLERABLE - OBJECT MARKER DAMAGED BUT VISIBLE

INTOLERABLE - OBJECT MARKER SEVERELY DAMAGED, NOT VISIBLE, OR MISSING

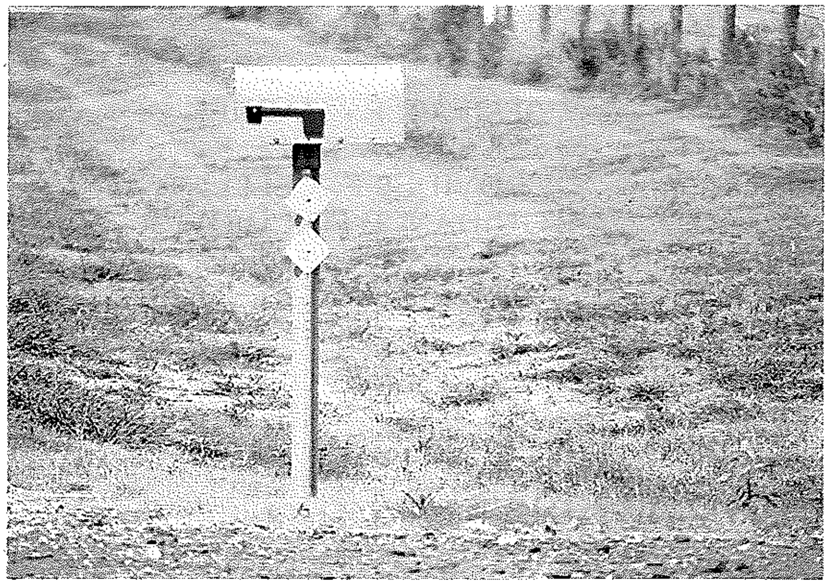
DELINEATORS

DESIRABLE - MARKERS CLEARLY INDICATE PATH OF ROADWAY

TOLERABLE - FEW MARKERS DAMAGED, PATH OF ROADWAY STILL DISCERNABLE

INTOLERABLE - MARKERS DAMAGED OR MISSING TO EXTENT THAT PATH IS NOT VISIBLE

MAIL BOXES

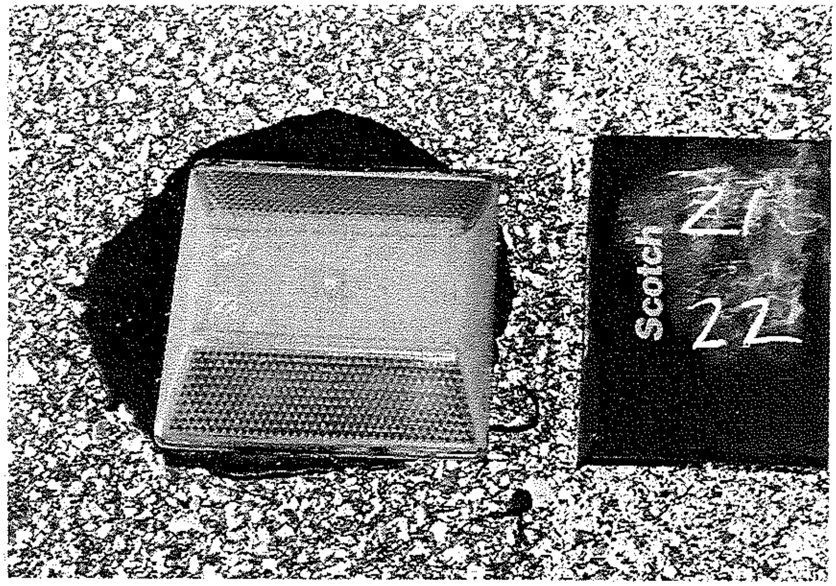


DESIRABLE - APPROVED POST WITH REFLECTOR

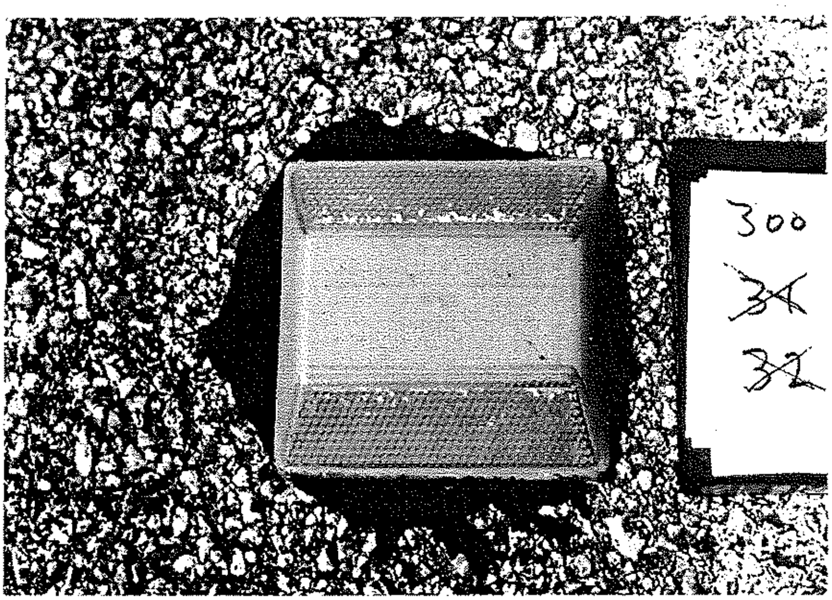
PHOTO NOT AVAILABLE

INTOLERABLE - UNAPPROVED POST OR NO REFLECTOR

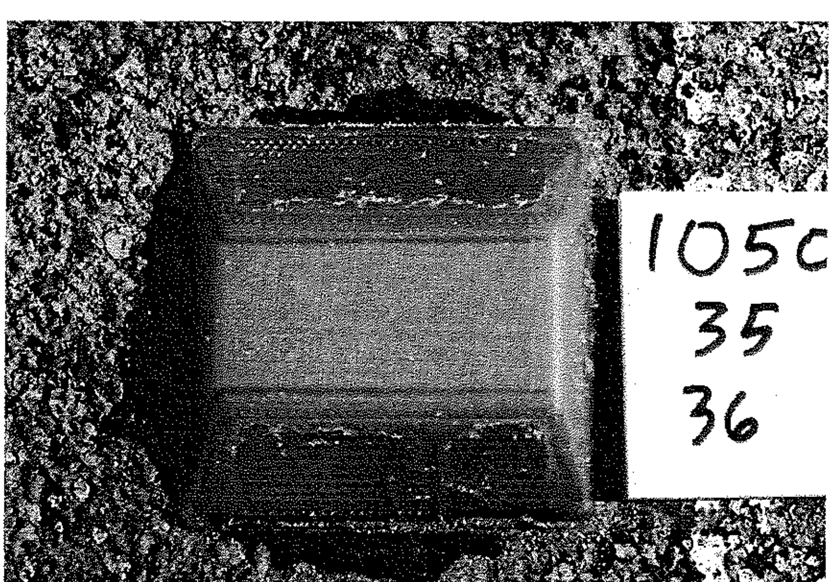
PAVEMENT MARKERS



DESIRABLE - MARKER CLEARLY VISIBLE, PROVIDES EXCELLENT GUIDANCE FOR DRIVER



TOLERABLE - MARKER VISIBLE, PROVIDES SUFFICIENT GUIDANCE FOR DRIVER



INTOLERABLE - MARKER BARELY VISIBLE OR MISSING, PATH OF ROADWAY UNCLEAR

Nighttime Evaluation

<u>Roadside Signs</u>

Desirable: excellent visibility
Tolerable: the signs can be seen at the distance specified in the MUTCD

<u>Delineators</u>

Desirable: clear indication of roadway alignment
Intolerable: does not provide proper guidance for vehicles

<u>Object Markers</u>

Desirable: reflectors properly indicate position of object
Intolerable: no reflectors indicating object, object marker missing

<u>Pavement Markers</u>

Desirable: marker clearly visible, clearly indicates path of roadway
Tolerable: marker visible, path of roadway visible
Intolerable: marker barely visible or significant amount missing

<u>Pavement Markings</u>

Desirable: correctly conveys desired message to driver
Intolerable: markings are unclear and do not provide sufficient information to driver

<u>Painted Lines</u>

Desirable: clearly provides necessary information and guidance to driver
Intolerable: markings are unclear, does not provide adequate information to driver

<u>Traffic Signals</u>

Desirable: excellent visibility, no bulbs out
Tolerable: signal is visible within minimum stopping distance, bulb out

<u>Illumination</u>

Desirable: none of the luminaires out
Tolerable: one or two luminaires out in run
Intolerable: three or more luminaires out in run

Research

Maintenance manuals and other maintenance surveys were examined to determine what items were deemed necessary to be examined. The maintenance manuals were also studied to provide guidelines on how to create such a manual. Ideas and concepts were drawn from various documents. The main source of information was collected from ROCOND 87 the New South Wales Road Authority Australian road maintenance manual.

Processes for acquiring the needed information were studied. Automatic processes for measuring retro-reflectivity of signs and pavement markings were found. These processes were determined to be unnecessary for the level of evaluation to be performed. The only process studied in depth is the ARAN system.

Guidelines are developed using documents provided by the Texas Department of Transportation (Tx DOT) and the Texas MUTCD set of guidelines.

The Manual

This manual consists of two and three level scales ranging from desirable to tolerable to intolerable. Because the levels used are to be basic and not too in depth, photographs of each level are attached. This benefits in two ways. The photographs represents the level and it narrows down the mental image that the evaluator would create in his or her mind if only words existed.

The ARAN System

The ARAN is a modified van which allows for video equipment and other measuring devices to be attached. The setup run during testing was a camera mounted in the front and a VCR and monitor mounted in the ARAN vehicle. Overall, the opinion generated in this report is that the ARAN system can be a valuable tool. A minimum amount of personnel and a minimum amount of time is required on the roadway because the ARAN can record a large amount of information.

The ARAN system contained several faults which are deemed fixable. The first minor fault is that the speedometer of the vehicle was almost constantly registering ten miles per hour different than that of the trailing vehicle during testing. This can be easily calibrated and made aware to the driver of the vehicle. It is also possible that this may not be a problem with other ARAN vehicles. The next problem encountered was that the camera kept drifting out of focus. This was remedied by focusing the camera and the taping the lens so that it did not move. The major problem arising during the testing was camera vibration. Vibration occurred severely on a newly paved road. Frame by frame advancement during review of the tape would allow one to focus on specific objects, but if trying to view the tape at regular speed one could easily miss an object marker or other item. A video with a fixed mount camera was viewed and the video showed no signs of vibration. Another severe problem encountered is the inability of the camera to react to vast differences between light and dark. When trying to video drainage pipes or culverts, one can only see a dark blur which does not reveal if the drain is blocked or not.

When the van drives into the sun, it is sometimes hard to decipher words on signs because the picture is washed out. Concerning the amount of time spent on the road, there was only one camera mounted on the vehicle which then required two trips in each direction of the road. This was in order to get a side view of items and then a front view of items. If two trips are going to be made each way, one might as well send a person out to evaluate the road manually. Therefore, it is suggested that two cameras be mounted on the vehicle.

Legal issues also need to be examined in the use of video tapes. It has been noted that in most court cases it is the lack of evidence that hurts the Department. If a person has an accident on a section of roadway and the video exists of that roadway, the tape can be used in the court as evidence. This concern has been brought up in several discussions. Based on risk management principles, good data is rarely damaging to Tx DOT.

The benefit of the ARAN system is that if these problems are corrected, the vehicle will be able to run at regular highway speeds and tape all of the necessary items. Taping the roadway will decrease the amount of time and the amount of personnel needed on the roadway. If two people are in the van and cover all of the necessary roads in one day, then only one person really needs to analyze the tape in the office. Analyzing the video tape may only take half of a day due to the ability to fast forward past sections which are not needed and stop the camera at necessary items. The other option is that two people may need to be in a vehicle evaluating the necessary items for two or more days not driving at regular speeds. With a video containing a wide variety of items to be looked at, one person can view the tape looking for example at delineators. When done, another person can watch the tape looking for example at object markers. The convenience of video is that the tape can be reviewed at any time. The tapes can be saved and when the next survey is made, comparisons can be performed to determine changes in the maintenance level-of-service. This can help personnel realize when items will need to be repaired. The tapes can also help to determine if contract workers are performing their jobs adequately. If a more developed maintenance survey is developed, the tapes will make it easier to reevaluate the items. The idea of creating an inventory of the items on the roadway has come up. An inventory of roadside items is not included in the scope of this project, but in the future the tapes could be used for this purpose or the tapes could be shared with another group doing this type of project.

Why ARAN Was Chosen

Field maintenance personnel can evaluate the roadway using forms on paper while driving down the road or they can evaluate the roadway using forms on paper or computers using the video tape from the ARAN. One might ask why doesn't one use a computer in the vehicle while driving down the road. It is true that they could do this also. The main fact is that the personnel have to gather the information and any way they do it, they must record the levels-of-service correctly. If the evaluator is driving in the vehicle, they must drive slower than the rest of traffic to accurately evaluate the roadway. If they increase their speed, then they might have to go back and do another run just looking at a particular item. The ARAN can make one pass down the road at or near the posted speed limit and record all of the information onto tape. The vehicle exits the roadway thereby alleviating the problem of being a hazard to other traffic. A minimum amount of time has been spent on the road reducing the cost associated with sending out an evaluation team. Also, the ARAN will record the roadway anyway, thus reducing data collection costs to a minimum. The video is brought back to a home base and reviewed. The tape can be paused at sections to look at an item, and since one is not on the road stopping the tape does not create a hazard. One can also fast forward past sections which do not contain items of interest. When fast forwarding in the office, the tape is driving at 150 miles per hour while if on the roadway this would be quite unsafe and illegal.

The ARAN saves time spent on the road which has a higher cost associated with it than an employee in the office viewing a video. Because the ARAN can make one pass and does not have to slow down for items on the roadway it does not create a hazard on the roadway. For safety reasons, this is very important. A condition survey is performed using the ARAN on high volume roads such as in Houston, Dallas, Ft. Worth, etc. Presently, the ARAN is run on the Interstate System once a year, once every other year on United States and State Highways, and once every four years on Farm to Market Roads. Because the ARAN records information to tape, the tapes can be viewed at any time in the office.

REFERENCES

1. *1980 Texas Manual on Uniform Traffic Control Devices for Streets and Highways*, State Department of Highways and Public Transportation, Austin, Texas, 1980.
2. Ken Porter. *ROCOND 87, Road Condition Manual*, Pavement Management Systems, Department of Main Roads, New South Wales, Australia, 1987.
3. Texas Department of Transportation D-18, 5-92 Guidelines, 1992.

Safety Effects of Limited Sight Distance at Railroad-Highway Grade Crossings

JENNIFER MESSICK

INTRODUCTION

One of the main objectives of the Federal Highway Administration (FHWA) is to ensure the safety of the nations highways. Railroad-highway grade crossings are included in this concern. In the United States there are over 215,000 public railroad-highway grade crossings. In 1990 alone, there were 5,766 train/automobile accidents at these crossings resulting in 2,588 injuries and 798 fatalities. Based on these statistics, the need for crossing improvements is obvious. The FHWA implemented the Federal-Aid Highway Act in 1973 to insure a constant effort in improving the Federal Highway system. Section 203 of this Act authorized each state to use money from the Highway Trust Fund for the sole purpose of upgrading grade crossings. These modifications were intended to reduce the number and severity of accidents that occur at grade crossings each year.

Railroad-highway grade crossings present an interesting problem. Unlike other intersections, grade crossings involve two different modes of transportation; the train and the motor vehicle. Typically, vehicles at a highway intersection have the ability to change their speed and direction in reaction to other drivers in order to avoid collision; however, at a grade crossing the train cannot alter its course or stop quickly. For example, a typical 100 car freight train traveling 60 mph requires over one mile to stop in emergency braking. Thus, the responsibility of avoiding collision falls on the driver of the motor vehicle. All of the crossings cannot be immediately improved; therefore, those crossings which pose the most danger need to be upgraded first.

The state of Texas uses the Texas Priority Index to rank all of the crossings in the state. This index gives each crossings a number based on characteristics of the crossing such as average daily traffic, speeds of the vehicle and train and accident history. The crossings are ranked highest to lowest based on their degree of danger. Many states use available sight distance in their index, however Texas does not. It is believed that accidents may be prevented by ranking the crossings on the basis of available sight distance in addition to accident history.

PROBLEM STATEMENT

The state of Texas has 9,153 grade crossings. With the amount of funds allocated each year, it would take over 70 years to upgrade all of these crossings. The current methods of prioritizing crossings have been successful by using the exposure method. The top third most hazardous crossings have been identified and upgraded. The middle third however, have similar numerical ratings and it less clear as to which crossings should be upgraded next. Another distinguishing factor is needed in the priority index to differentiate these crossings and ensure the most efficient use of the funds allocated for improvements. Intuitively, sight distance should be included in the priority index since it is used in nearly all other aspects of highway design; however, the effects of using available sight distance in the priority index are not known.

OBJECTIVE

The objective of this study was to determine the role sight distance plays in accidents at passive railroad-highway grade crossings. Passive crossings have signs and pavement markings but do not have any active warning devices such as gates and flashing lights. The following steps were taken in order to complete the objective:

1. Select the study sites.
2. Calculate the required sight distances.
3. Measure the available sight distances.
4. Analyze the results.
5. Make conclusions and recommendations.

BACKGROUND

Railroad-highway grade crossing sight distance is defined as the length of the roadway ahead and along the tracks that can be seen by a driver. Ideally, grade crossings would have unlimited sight distance along both the roadway and track. This provision should provide the optimum amount of safety and maximize traffic flow; however, due to the high costs of accumulating and maintaining the necessary right-of-way, this is not always feasible or practical. Three sight distances describe the visibility of a grade crossing. These are illustrated in Figure 1.

The first and most obvious type of sight distance is the visibility of the crossing which is referred to as the approach sight distance. A driver needs to be able to detect the crossing well in advance in order to begin thinking about what he or she should do upon reaching the crossing. This distance is measured along the roadway from the driver of the vehicle to the nearest rail. With adequate sight distance, the driver can stop the vehicle before the tracks and avoid a collision. If this distance is too short, the vehicle will not have enough distance to stop and could possibly have a collision with an approaching train. Another possibility is that the driver will have to slam on the brakes and may be rear-ended by a following vehicle.

The second type, quadrant sight distance, is the visibility in the quadrants to the driver's left and right. A driver must be able to perceive the approaching train from either direction and make a decision to either cross the tracks or wait for the train to pass. Most often this sight distance is blocked by buildings or vegetation. The land surrounding the tracks may not be owned by the railroad. The right-of-way may belong to the railroad, highway, or it could be private property. This may prevent the FHWA from removing the obstruction. In this case, an active crossing is recommended or in the very least warning signs and a reduced speed limit.

The third sight distance is known as the track sight distance and is required for vehicles that are stopped at a crossing. Track sight distance is crucial for passive crossings because the driver of the vehicle must decide when it is safe to cross. A driver stopped at a crossing must have sufficient visibility to observe the train and to decide whether or not to cross the tracks. This distance is perhaps the most important because some vehicles, such as school buses and hazardous material carriers are required by law to stop at the crossings and proceed through the intersection without shifting gears. These vehicles must have sufficient sight distance to allow them enough time to clear the tracks without colliding with a train.

Three formulae are used to calculate the three required sight distances. These formulae are found in both the FHWA Railroad-highway Grade Crossing Handbook (1) and the AASHTO's Policy on Geometric Design of Highways and Streets (2). These formulae are based on the assumption that the vehicle is a 65 foot truck and it is crossing a single track at 90 degrees on a flat surface. These assumptions will result in the maximum required sight distances needed. Adjustments to these formulae must be made for skewed crossings, multiple tracks, abrupt surfaces, or unusual types of vehicles with atypical acceleration and deceleration rates.

1) Approach sight distance along the highway for a moving vehicle

$$d_H = 1.47\ V_v\ t + \frac{V_v^2}{30f} + D + d_e$$

d_H	=	sight distance along the highway (ft);
V_v	=	velocity of vehicle (mph);
t	=	driver perception/reaction time (assumed to be 2.5 sec);
f	=	braking coefficient of friction;
D	=	distance from front of stopped vehicle to the nearest rail (assumed to be 15 ft); and
d_e	=	distance from driver to front of vehicle (assumed to be 10 ft).

2) Quadrant sight distance along the tracks for a moving vehicle

$$d_T = \frac{V_T}{V_v}\ [\ 1.47V_{vt} + \frac{V_v^2}{30f} + 2D + L + W$$

d_T	=	sight distance along the tracks (ft);
V_v	=	velocity of vehicle (mph);
V_T	=	velocity of train (mph);
t	=	driver perception/reaction time (assumed to be 2.5 sec);
f	=	braking coefficient of friction;
D	=	distance from front of stopped vehicle to the nearest rail (assumed to be 15 ft);
L	=	length of vehicle (assumed to be 65 ft); and
W	=	distance between outer rails (for a single track assumed to be 5 ft).

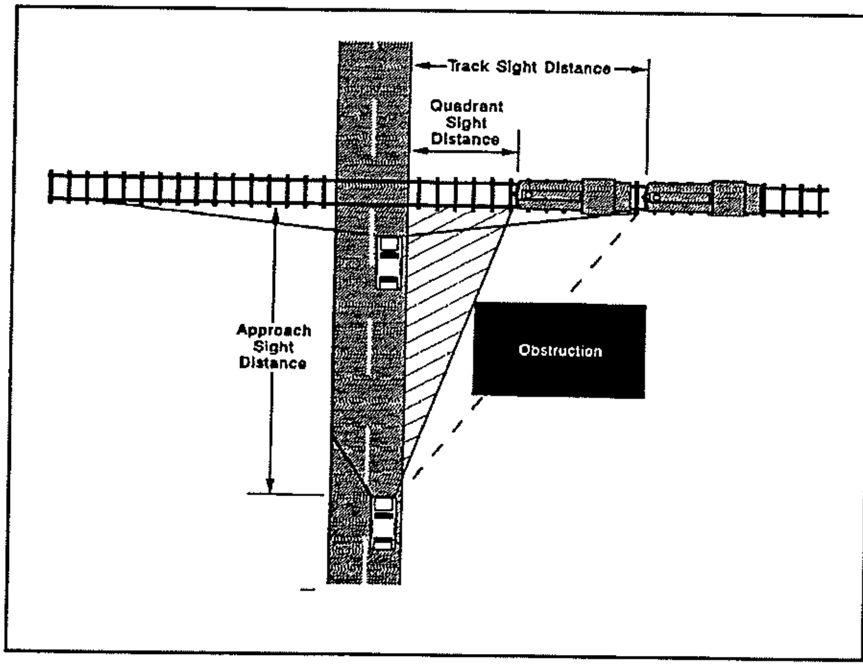


FIGURE 1. TRACKING SIGHT DISTANCE.

3) Track sight distance for a stopped vehicle

$$d_T = 1.47V_T \left[\frac{V_G}{a_1} + \frac{L + 2D + W - d_a}{V_G} \right.$$

d_T = sight distance along the tracks (ft);
V_T = velocity of train (mph);
V_G = maximum speed of vehicle in first gear (assumed to be 8.8fps);
a_1 = acceleration of vehicle in first gear (assumed to be 1.47 ft/sec^2);
D = distance from front of stopped vehicle to the nearest rail (assumed to be 15 ft);
L = length of vehicle (assumed to be 65 ft);
W = distance between outer rails (for a single track assumed to be 5 ft);
d_a = distance vehicle travels while accelerating to maximum speed in first gear (ft); and
J = perception/reaction time, which is assumed to be 2.0 sec.

FIELD STUDY DESIGN

Sight distance is said to be limited when the available sight distance is less than the required sight distance. Active warning devices are typically installed at crossings where the sight distances are known to be limited, therefore; only passive crossings were considered in this study. Eighty one crossings from Brazos, Burleson, and Grimes counties were selected for evaluation. Inventory data was used to calculate the required sight distances using the equations given above. Table 1 shows the required sight distances used for different combinations of train and vehicle speeds.

The pacing method was used to measure the available sight distances at the crossings. The pacing technique involves counting how many paces a person walks in a given distance, measuring the length of the persons stride and converting the number of paces to the actual distance in feet. A three person team measured all three sight distances at the crossings using the following procedure:

The approach sight distance was measured first. Person A walked down the road and stood in the position of a driver until the cross buck first came into view. The number of steps Person A took was converted to the actual distance and recorded.

The track sight distance was measured next. Persons B and C walked down the tracks in opposite directions. Person A remained standing 15 feet away from the tracks at the position the driver of the vehicle would be. Persons B and C continued walking down the tracks until person A was just out of sight. These distances were recorded. Person A then walked across to the other side of the tracks and stood 15 feet from the

TABLE 1. REQUIRED SIGHT DISTANCES FOR COMBINATIONS OF HIGHWAY AND TRAIN VEHICLE SPEEDS

Train Speed (MPH)	Highway Speed (MPH)							
	0	10	20	30	40	50	60	70
	Required Quadrant Sight Distance							
10	162	126	94	94	99	107	118	129
20	323	252	188	188	197	214	235	258
30	484	378	281	281	295	321	352	387
40	645	504	376	376	394	428	470	516
50	807	630	470	470	492	534	586	644
60	967	756	562	562	590	642	704	774
70	1129	882	656	656	684	750	822	904
80	1290	1008	752	752	788	856	940	1032
90	1450	1134	844	844	884	964	1056	1160
	Required Track Sight Distance							
	20	65	125	215	330	470	640	840

nearest rail. Persons B and C adjusted their position until person A was again just out of sight. These distances were also recorded.

The quadrant sight distance was measured last. Person A stood in the roadway at the required approach sight distance. Persons B and C walk down the tracks until person A could no longer see them. This distance was recorded. Person A then walked across the tracks to the required approach sight distance on the other side. Persons B and C adjusted their position accordingly and recorded the distances.

RESULTS

The 81 crossings selected were evaluated based on their available sight distances, accident reports, and inventory data for the past five years. The measured available sight distances were compared to the required sight distances. The crossings were separated into three groups based on their available sight distance. Group A contains all of the crossings that are not limited; they all have the required amount of sight distance. Group B contains the crossings which have at least one limited quadrant sight distance. Group C contains the crossings with both limited track and quadrant sight distances. Two crossings did not fit into these groups and were not used for the frequency or statistical analyses.

Frequency Analysis

Intuitively, if the number of accidents were correlated to limited sight distance, the percentage of accidents at crossings with the required sight distance would be different than the percentage of crossings with limited sight distance. This study included 79 crossings which had 42 accidents over the past five years. Table 2 shows the frequency of accidents at the crossings in groups A, B, and C. Seventy four percent of the crossings had limited sight distance. Of the 42 accidents, 69 percent of them occurred at crossings with limited sight distance. This simple analysis shows that the percentages are approximately equal and does not indicate any correlation between sight distance and the accident rate.

TABLE 2. FREQUENCY ANALYSIS BY TYPE OF CROSSING

Number of Accidents	Group A	Group B	Group C	Total
0	12	16	21	49
1	5	4	13	22
2	2	1	2	5
3	0	0	2	2
4	1	0	0	1
Total	20	21	38	79

Statistical Analysis

The crossings were once again separated into groups A, B and C for this part of the analysis. A Pearson Chi-squared test was performed on the data to check for independence. This test uses the two-way contingency table shown below.

The research hypothesis in this case is that there is a difference between the number of accidents occurring at crossings with limited sight distance and crossings with the required amount of sight distance. The object of the chi-squared test is to either support or disprove this hypothesis. The observed numbers are shown in Table 3 and represent the number of crossing in each group that had either zero or more than one accident in the five year period. The expected values are in parentheses next to the observed values in the table. To calculate these expected values, the sum of the first column is multiplied by the sum of the first row and divided by the sum of rows and columns (3). The next step is to calculate the chi-squared test using the following formula:

$$X^2 = \sum \frac{[\ observed\ -\ expected\ ]^2}{expected}$$

The value calculated for chi-squared is compared to standard theoretical values. If this number is greater than the theoretical value for the desired accuracy the research hypothesis is supported. If the calculated value is less than the theoretical value, the hypothesis is rejected.

The value for chi-squared in this case was found to be 2.557 which is less than the theoretical value of 5.991 (3). Therefore, there is no evidence to support the research hypothesis. In accordance with the frequency analysis, no relationship between accident rate and restricted sight distance was apparent.

Accident Causation Analysis

A total of nine crossings were evaluated for this part of the analysis. Accident narratives for eleven accidents at these crossings were obtained. These narratives as well as inventory data were used to determine any trends in the accidents. Table 4 is a summary of the accident reports and causal factors.

Four of the accidents occurred at crossings where the sight distance was not restricted in any way. These accidents all appear to have been caused by driver error. In these cases, limited sight distance cannot be listed as a causal factor.

Seven of the accidents occurred at crossings with limited sight distance. In three of the accidents, driver error also appears to be the main contributing factor. The accident reports in conjunction with field data indicated a lack of sight distance as the causal factor in the remaining four accidents.

This analysis does not indicate any significant relationship between sight distance and the number of accidents at a crossing, yet it does show four out of ten accidents were directly caused by limited sight distance.

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the data obtained and analyzed, the following conclusions were reached:

TABLE 3. ACCIDENT FREQUENCY FOR CHI-SQUARED ANALYSIS.

Number of Accidents	Group A	Group B	Group C	Total
0	12 (12.41)	21 (23.57)	16 (13.03)	49
1 or more	8 (7.59)	17 (14.43)	5 (7.97)	30
Total	20	38	21	79

TABLE 4. ACCIDENT SUMMARY.

Accident Narrative	Contributing Factors	Limited S.D (Y/N)?	S.D a Factor (Y/N)?
Driver ignored stop sign, hit another car and pushed it onto the tracks where it was struck by a train.	Driver error	No	No
Driver pulled onto the tracks, saw approaching train, tried to reverse off the tracks and was struck by the train.	Driver error	No	No
Driver did not see approaching train.	Driver error	No	No
One car was speeding and hit another car that was stopped at the crossing waiting for a train to pass.	Driver error Dark crossing Raining	Yes	No
No trained involved. Car lost traction while crossings the tracks and hit an oncoming car.	Driver error Dark crossing Gravel road	Yes	No
Tractor pulled onto tracks and stalled when driver saw approaching train. Tractor was struck by the train.	Driver error	Yes	No
Car was abandoned on the tracks when a train hit it.	Abandoned car Dark crossing	Yes	No
Truck did not see the approaching train, pulled onto the tracks and was struck by the train.	Limited sight distance Driver error	Yes	Yes
Truck pulled onto the tracks, saw train, attempted to reverse off tracks and was struck by the train.	Limited sight distance Driver error	Yes	Yes
Car pulled into the path of the train, was stuck by train and fled the scene in the car.	Limited sight distance Dark crossing	Yes	Yes
Car pulled into the path of the train, was struck by the train. Driver fled the scene on foot.	Limited sight distance Dark crossing	Yes	Yes

1. A significant number of grade crossings have limited sight distance; 74 percent of the crossings studied had limited sight distance.
2. No evidence was found to support the hypothesis that the number of accidents is correlated to the available sight distance.
3. Based on the accident causation analysis, sight distance did appear to be a casual factor in four of the ten accidents that were evaluated.

Considering the above conclusions, the following recommendations were made:

1. An expanded data base be used for further research.
2. Programs that incorporate sight distance into the priority index should be looked at to possibly improve the priority ratings.

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Travel Time Reliability of High-Occupany Vehicle Lanes

CHARLES J. NAPLES III

High-occupancy vehicle (HOV) facilities have been shown to increase the person-carrying capacity of a roadway to a degree that would not be possible by adding the same number of general-purpose lanes. The implementation of HOV lanes, however, often results in much opposition by those who believe that HOV lanes are a waste of space that could be better used by adding general-purpose lanes.

Any evidence which shows that HOV facilities offer an advantage over additional general-purpose lanes could help to gain the support of the general public. This is important because as congestion worsens, and space grows short, the implementation of HOV facilities becomes increasingly necessary. One advantage of HOV facilities which is often cited, is that of greater travel time reliability than general-purpose lanes. There is, however, little-to-no data which actually can be provided to support this claim.

Using the lap-top computer methodology for the collection of travel time data, this study sought to provide this support. After data had been collected from both Houston and Dallas, these data were reduced and analyzed statistically. A one-sided test for equal variances showed that not only were the variances of the HOV and general-purpose lanes unequal, but that the variance of the HOV lanes were less. It was concluded, with a high level of statistical confidence, that HOV facilities provide greater travel time reliability than general-purpose lanes.

INTRODUCTION

Decreasing Operational Capacity

The traffic volumes for an increasing number of North American roadway corridors are reaching their capacity level. The adverse effects caused by these high volumes are numerous. The resulting low quality of service is indicated by a decrease in user safety, freedom to maneuver, and convenience. The congestion caused by such high volumes generates excessive fuel consumption, increased exhaust emissions, and increased travel times.

As this condition worsens, the need for more efficient transportation system design solutions becomes greater. Equally as important as the examination of alternatives is a review of the physical and financial limitations imposed upon corridor improvements. These restrictions can significantly affect the choice of alternatives and have become more stringent in recent years.

High-Occupancy Vehicle Facilities: A Viable Design Solution

High-occupancy vehicle (HOV) facilities (priority facilities designated for use by buses, vanpools, and/or carpools) are being increasingly recognized as attractive alternatives to improve urban mobility. HOV facilities have been shown to make efficient use of available right-of-way in roadway corridors where expansion is limited by physical constraints, financial restrictions or both (1). When properly implemented, these priority lanes can increase the person-carrying capacity of a roadway to a degree that would not be possible by adding the same number of general-purpose lanes. However, using space for the implementation of HOV lanes (as opposed to adding general-purpose lanes) often results in much public opposition.

For a transportation alternative to be considered an acceptable design solution, its advantages must be easily seen and understood by individuals who do not necessarily possess a technical understanding of transportation. In the case of HOV facilities, effectiveness can only be achieved through use by those who have normally travelled along general-purpose lanes in single-occupant vehicles. This can typically only be accomplished if an HOV facility has distinct, valid, and easily understood advantages over other transportation alternatives in a given travel corridor.

Travel Time Reliability

In a report completed by Wade, Morris, and Christiansen, travel time savings and travel time reliability were charged as being the two most important factors influencing HOV facility use (2). When implemented in congested freeway corridors, HOV facilities typically offer an advantage over general-purpose lanes in both of these areas. There is a significant amount of data which substantiate the claim that HOV facilities provide travel time savings compared to general-purpose lanes. Little to no data have, however, been documented which compare the travel time reliability offered by HOV facilities to that of general-purpose lanes. It appears that this measure should be investigated, since any data that indicate an advantage of using HOV facilities over general-purpose lanes would help in gaining community support for the implementation of HOV facilities. The objective of this study was, therefore, to investigate the travel time reliability of HOV facilities compared to that of general-purpose freeway lanes.

METHODOLOGY

The investigation concerning travel time reliability required the statistical analysis of travel time data. The following is an overview of the techniques which were used to collect and analyze these data.

Travel Time Data Collection

The two most commonly used methods for the collection of travel time data are the test vehicle method and the license plate matching method. The test vehicle method is the most widely used of all methods. This method uses a vehicle (which is actually driven in the stream of traffic) to determine the travel time along a length of road under the prevailing conditions. Several techniques can be used to approximate the average speed of the traffic stream with the test vehicle. The most commonly used of these techniques is the floating car technique. When applied on freeways, this technique typically involves travelling in one of the middle lanes (i.e., a lane other than the extreme inside or outside lanes). Ideally, the operator of the test vehicle passes only as many vehicles as pass him/her. By doing so, the test vehicle travels at a speed which approximates the average travel speed of the traffic stream (3).

To measure the travel time data, a passenger usually rides along with the operator of the vehicle and measures the elapsed time between predetermined mile points. Sometimes the driver conducts the test alone by using an automatic timing device. This latter approach is usually the exception, because it is felt that the added task of operating the timing device decreases the safety to the driver.

The license plate matching method involves the measurement of travel times for vehicles spanning a test section of roadway. This approach is implemented by positioning observers at the entrance and exit points of the test section. The observers are equipped with the means to record both the license plate numbers and times of passing vehicles. The numbers collected by each observer are then entered into a computer where they can be matched. The travel times and average speeds are then calculated.

The most recent improvement to the license plate matching method is the utilization of lap-top computers in the field. The lap-top methodology provides a quick and relatively accurate method to record license plate numbers and to determine travel speeds of observed vehicles. The observer need only type in the characters of a license plate, and the computer automatically time stamps and records the entry. Not only does the use of lap-top computers in the field aid in data collection, but it also makes the analysis of the data easier and more efficient. The data, which are saved onto a disk, can easily be retrieved by a personal computer in the office. In the past, data were recorded either on paper or on audio tape. These data were then entered manually into the computer. This process was typically long and expensive (4).

Travel Time Reliability

The license plate matching method, or more specifically, the lap-top computer technique lends itself well to the investigation at hand. In a study conducted by Rickman, Hallenbeck, and Schroeder, it was noted that the license plate matching method provided a larger set of observations than the test vehicle method (4). This approach, therefore, provided a higher level of statistical confidence than the test vehicle method in the estimation of mean travel time. The license plate matching technique was, thus, the methodology used to collect the necessary data for this study. For the study of travel time reliability, the large data set which is possible by using the license plate matching method is critical. It may be for this reason that, to date, few investigations of this kind have been conducted.

Data Collection Sites

In order to conduct a travel time study, observers must be positioned at the beginning and end of a test section. The test sections chosen for this study were located along the Katy Freeway (I-10 West) in Houston, Texas and the East R.L. Thornton Freeway (I-30 East) in Dallas, Texas. These locations are shown in Figures 1 and 2.

The first test section along the Katy Freeway spanned the length between the Highway 6 (Addicks) HOV slip ramp and the Gessner HOV slip ramp. Data were collected from both the HOV and general-purpose lanes between 6:00 and 9:00 A.M., and then again from 3:30 to 6:30 P.M. on the same day. The next test section that was observed was between the Gessner HOV slip ramp and the Washington HOV slip ramp. Data were once again collected from both the HOV and general-purpose lanes during the same times as those stated for the first test section. Examples of the locations for data collection are shown in Figures 3 and 4. The vans that were positioned at the two ends of the test sections each contained two observers. One observer collected data from the HOV lane and the other collected data from the general-purpose lane. Each observer was equipped with a lap-top computer with an installed license plate collection program, and the means to power the lap-top computer for the entire data collection period. The positioning of the vans was determined with the safety of the observers being given as much, if not greater, consideration as visibility.

Due to the operation of the freeway, the two test sections along the East R. L. Thornton Freeway were examined in a manner different than those along the Katy Freeway. The first test section on East R.L. Thornton Freeway spanned the length of the entire HOV facility during the A.M. operation. One van was positioned at the Jim Miller Crossover, and the other was positioned at the central business district (CBD) Crossover. This section was observed between 6:00 and 9:00 A.M. The second test section was along the entire section of the HOV facility during P.M. operation; the vans were positioned at the CBD Crossover and the Dolphin Road Crossover. This section was observed from 4:00 P.M. to 7:00 P.M. The positioning of the observers at each data collection location was, once again, determined with both the safety of the observers and the visibility of the HOV and general-purpose lanes given significant consideration.

Data Reduction

The data collection process provided sixteen pairs of license plate files. The next step to be undertaken was the matching of each pair of files. This process utilized a matching program on a personal computer. The program matched identical license plate entries and, using the test section length, calculated the time interval and average speed for each matched pair.

Because the observers collected only four characters from each license plate, spurious matches resulted. Spurious matches are defined as "Matches that occur by random or human error" (5). For example, a spurious match may result if the observer at one end of the test section records the first four characters of license plate 123456, while the observer at the other end of the test section records the first four characters of the license plate 1234AB during the same collection period. The matching program will treat the pair of entries as a single vehicle and provide the match as output. Even though spurious matches result from recording only the first four characters of a license plate, that number of characters is simply the best that can be expected from observations of freeway traffic. In the study conducted by Rickman, Hallenbeck, and Schroeder, it was established that recording four characters of a license plate provides the best compromise between the ease of data collection, and the frequency of spurious matches resulting from random error (4).

Spurious matches were eliminated by two methods. First, the matching program allows the user to set upper and lower speed limits for output. The speed limits utilized in this analysis were 80 mph and 10 mph. Second, after the output has been provided, the user can visually edit the files while referring to field notes which give estimated speeds of the traffic stream during the collection period. The collection program provides a comments field in which to enter these notes during collection.

Data Analysis

After being edited and accepted as being representative of the traffic observed during the respective collection periods, the data were analyzed to determine whether or not the distributions of the average travel speeds were normal. This phase of the analysis was important because the method by which the variances of the test samples could be compared depended upon whether or not the distributions from all of the sample sets were normal.

Test For Normality

The distribution of average travel speeds for each matched data set was checked for normality. This analysis (as were all statistical analyses performed in this study) was performed using the Statistical Analysis Software (SAS). The specific test run in this case was the Shapiro-Wilk test for normality.

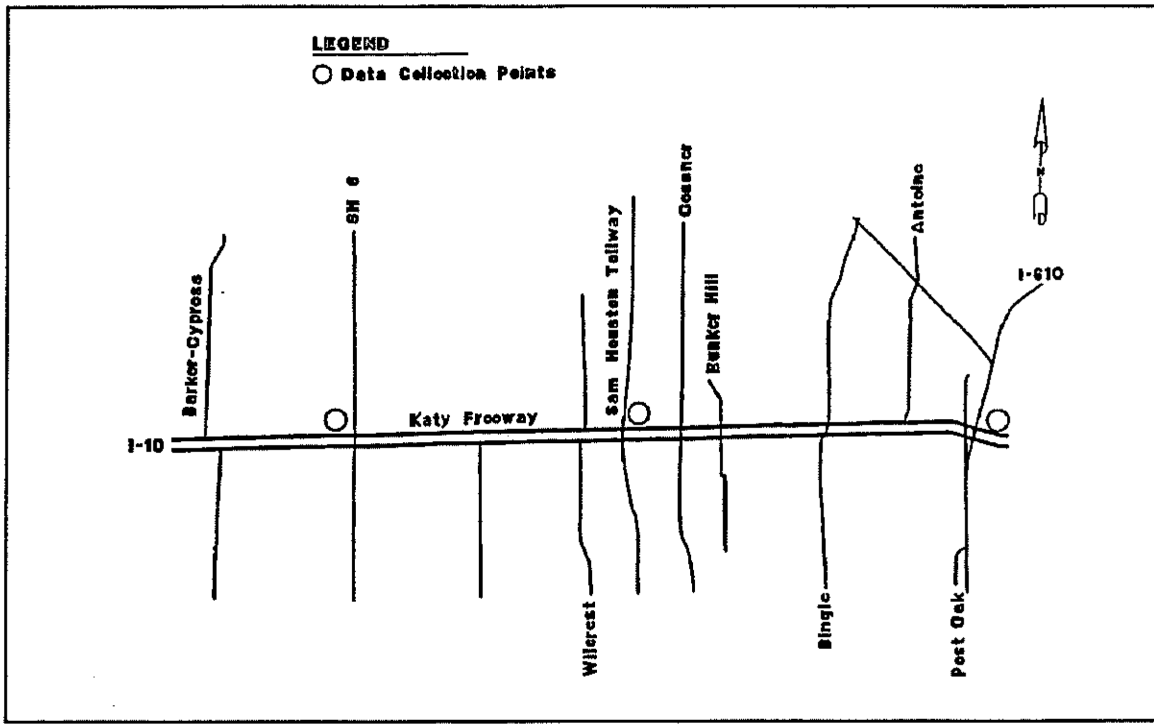

FIGURE 1. VICINITY MAP FOR KATY FREEWAY DATA COLLECTION POINTS.

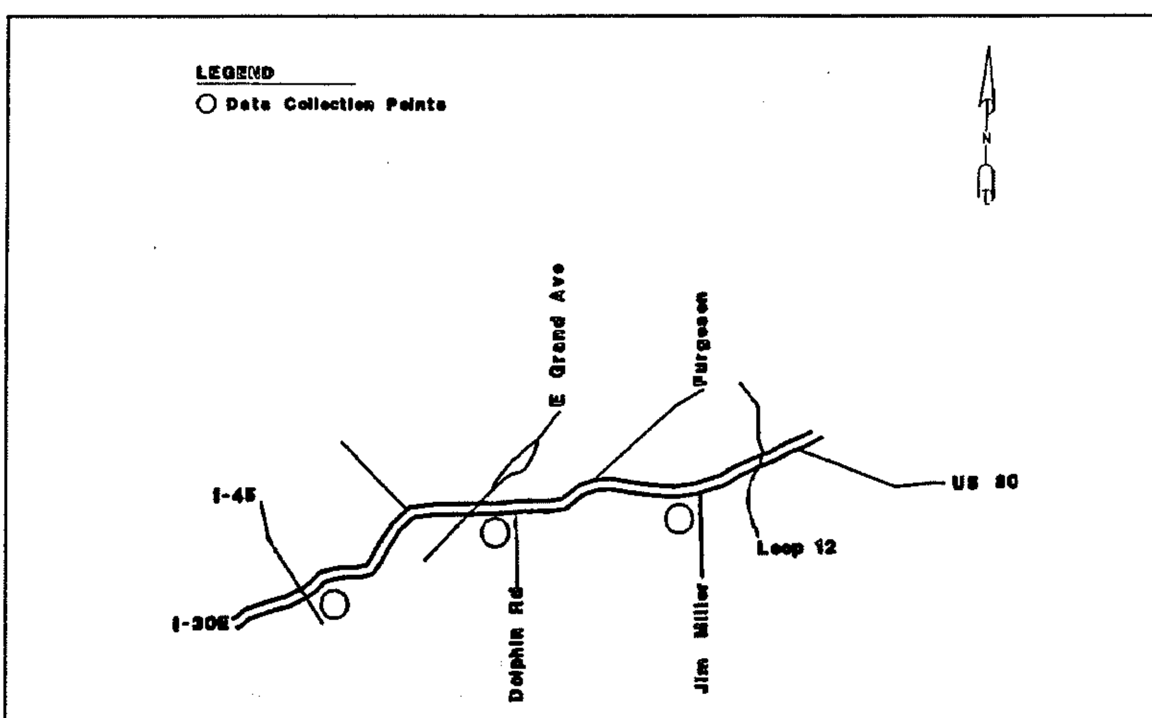

FIGURE 2. VICINITY MAP FOR EAST R.L. THORNTON FREEWAY DATA COLLECTION POINTS.

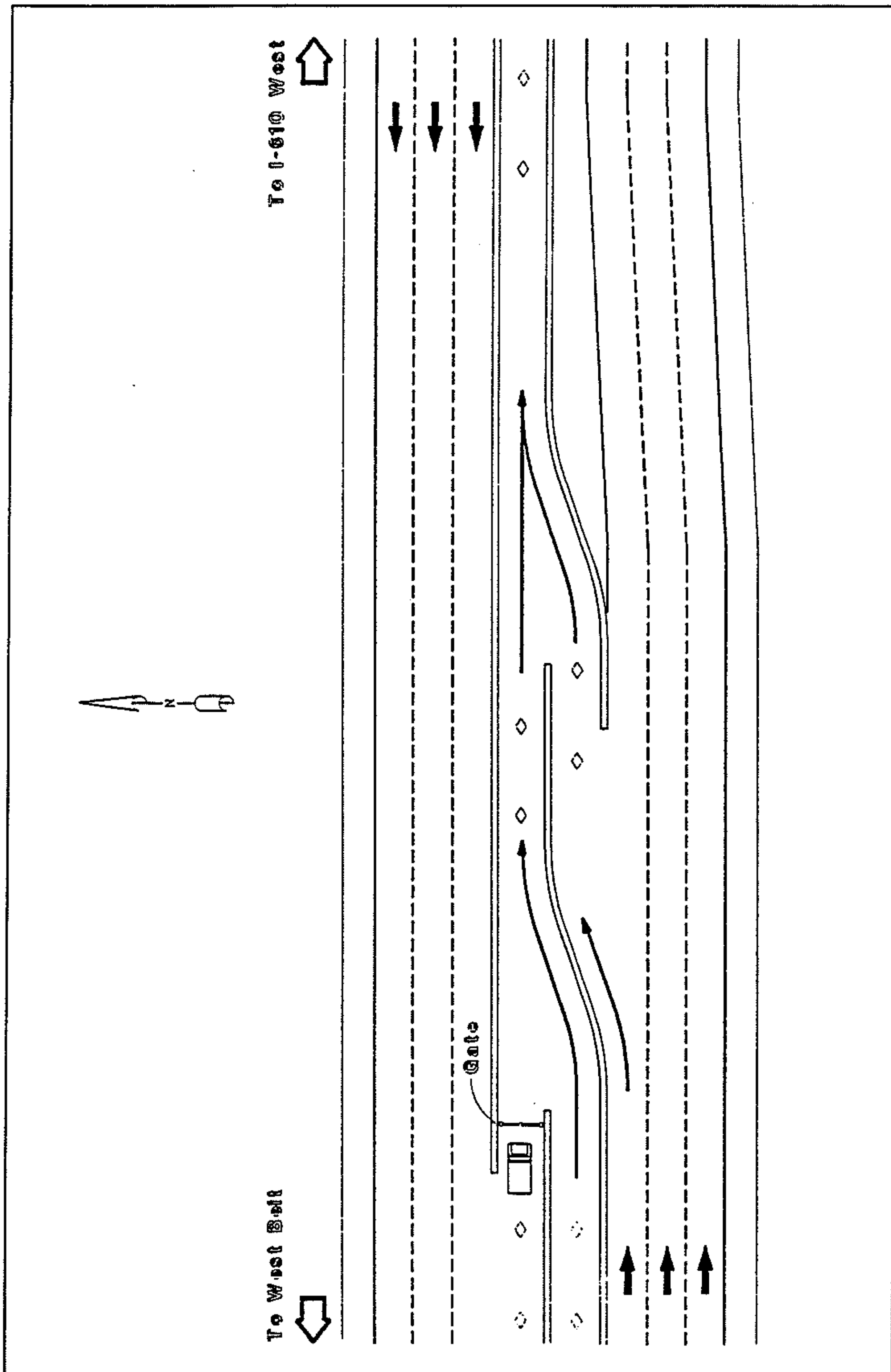

FIGURE 3. GESSNER SLIP RAMP CONFIGURATION DURING MORNING OPERATIONS.

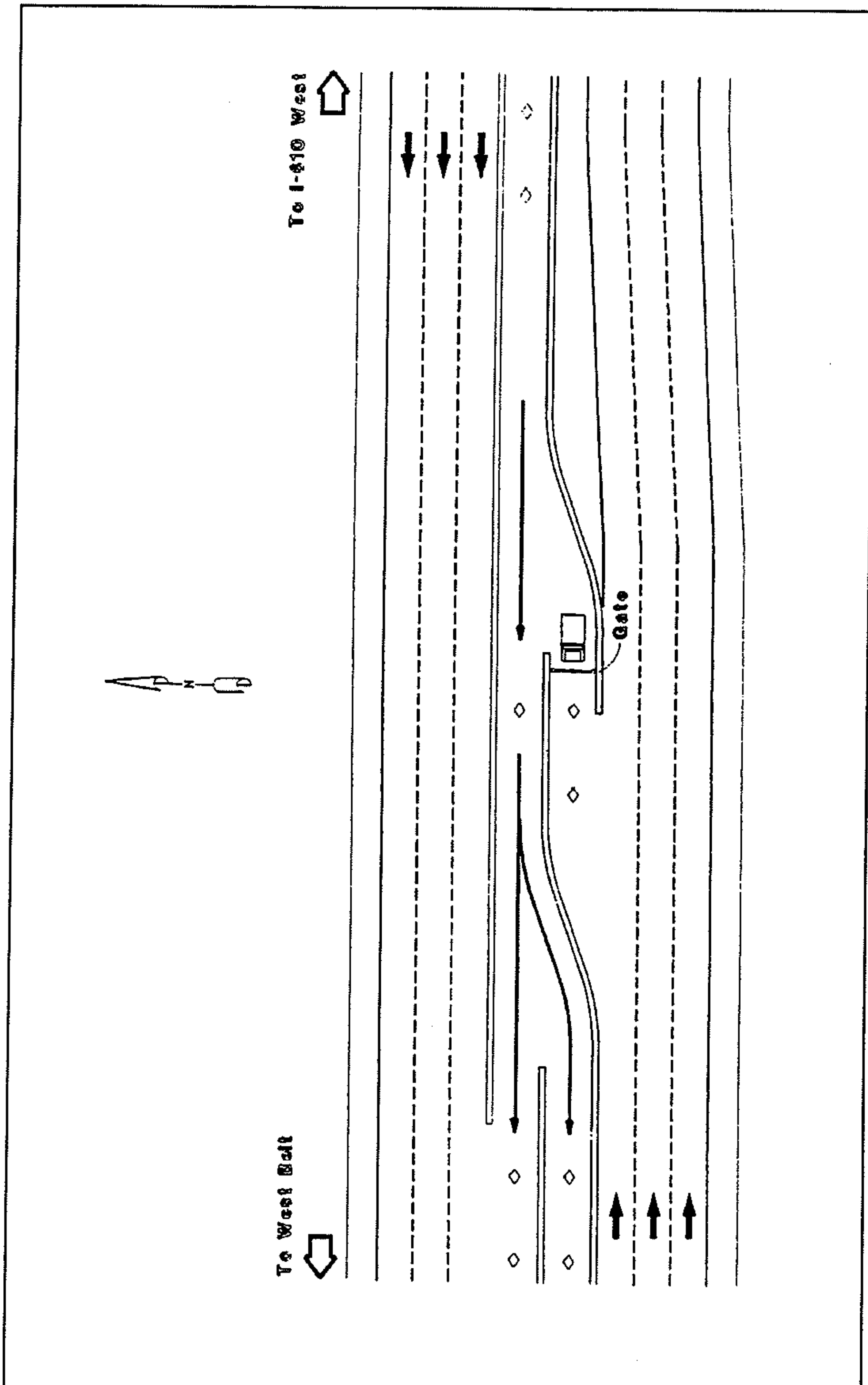

FIGURE 4. GESSNER SLIP RAMP CONFIGURATION DURING EVENING OPERATIONS.

Test For Equal Variances

The test which was used to compare the variance of the HOV sample sets to the variance of the general-purpose sample sets was the Squared Ranks Test for Variances. The process to perform this test included the calculation of the test statistic T_1 for each of the pairs of travel time sample sets collected during the individual collection periods. The overall test error rate (the probability that the null hypothesis was rejected given that the null hypothesis was in fact true) used in this analysis was 0.01 (1 percent). Once the error rate had been chosen, a comparison was made between the T_1 statistic and the tabular value associated with the error rate ($\alpha = 0.01$).

The test statistic was calculated using SAS. The way in which the individual test error rate was determined warrants some explanation. Due to the small number of tests performed, it was decided that it would be necessary to calculate an individual test error rate to insure that the overall experimental error rate could be achieved. By introducing the overall experiment-wise error rate ($\alpha^* = 0.01$), an individual test error rate ($\alpha = 0.0014$) was easily determined. This individual test error rate was then used to acquire a new value with which to compare the test statistic.

RESULTS

Data Reduction

The number of license plates collected during this study varied not only by location, but also by peak period. Although it is evident that the numbers increased slightly as observers became more proficient at recording license plates, it appears that the number of possible license plate matches for each freeway was highly dependant upon the peak period being observed.

The percentage of possible matches which result in audited matches, however, seems to depend not the on peak period, but on the test section. Table 1 contains the number of possible matches and the percentage matched for each test section.

Data Analysis

The results of the normality check established what was expected. All of the distributions of the average travel speeds for the HOV lanes in both the Houston and Dallas areas were normal. However, none of the distributions of the average travel speeds for the general-purpose lanes were normal. An example of one of the HOV speed distributions is shown in Figure 5, while an example of a typical general-purpose speed distribution is shown in Figure 6. Because none of the distributions of the average travel speeds for the general-purpose lanes were normal, non-parametric statistics were necessary to compare the variance of the HOV lane to the variance of the general-purpose lane.

The test statistic (T_1) associated with the individual test error rate ($\alpha = 0.0014$) was -3.48. Table 2 provides the value for the test statistic calculated for each sample set pair. If the test statistic is less than the value -3.48, then the null hypothesis is rejected for that test. The null hypothesis states that the variance of both the HOV lane travel speeds and the general-purpose lane travel speeds are equal.

CONCLUSIONS

Although this study was primarily concerned with the travel time reliability of HOV lanes as compared to general-purpose freeway lanes, much was also learned about the lap-top computer methodology of travel time data collection.

It appears that the size of the data sets collected by this method could be increased in two ways. The first way would be to increase the number of possible matches. This could be done by improving the visibility available to, or the proficiency of, the observers. The second way would entail increasing the percentage of possible matches which result in audited matches. This could be done by carefully choosing the observed test section based upon length between observation points and the number of access points to the freeway. The understanding of measures which help to increase the data sets provided by the lap-top computer methodology could greatly improve this already superior method by which to collect travel time data.

From the results shown in Table 2, it is evident that HOV facilities offer greater travel time reliability than general-purpose lanes during peak period conditions. All statistical tests indicated that the variance of the speed data for all the HOV lanes were lower than the variance of the speed data for the adjacent general-purpose lanes. Remembering that the overall experimental error rate was 0.01, the results can be accepted with a particularly high level of confidence.

The results and high level of confidence achieved by this relatively brief study should warrant further investigation. The intent of this study was to provide the proof that HOV facilities provide greater travel time reliability than general-purpose freeway lanes. That end has been reached within the bounds imposed by the amount of data available. Time restrictions did not allow for the collection of data which would be necessary for

TABLE 1. LICENSE PLATE MATCHING DATA.

Facility Type	Period	Houston Possible Matches[a]	Houston Percent Matched	Dallas Possible Matches[a]	Dallas Percent Matched
General-Purpose	A.M. Peak No. 1	903	5.3	875	8.0
	A.M. Peak No. 2	904	5.1	844	8.3
General-Purpose	P.M. Peak No. 1	1,018	3.2	N/A	N/A
	P.M. Peak No. 2	1,184	5.1	1,292	11.9
HOV	A.M. Peak No. 1	433	45.5	774	36.6
	A.M. Peak No. 2	543	32.2	806	39.3
HOV	P.M. Peak No. 1	417	47.5	N/A	N/A
	P.M. Peak No. 2	926	47.1	920	54.0

a Lowest number of license plates collected for test section during collection period.
N/A A second peak period data collection was not performed; data are, therefore, not available.

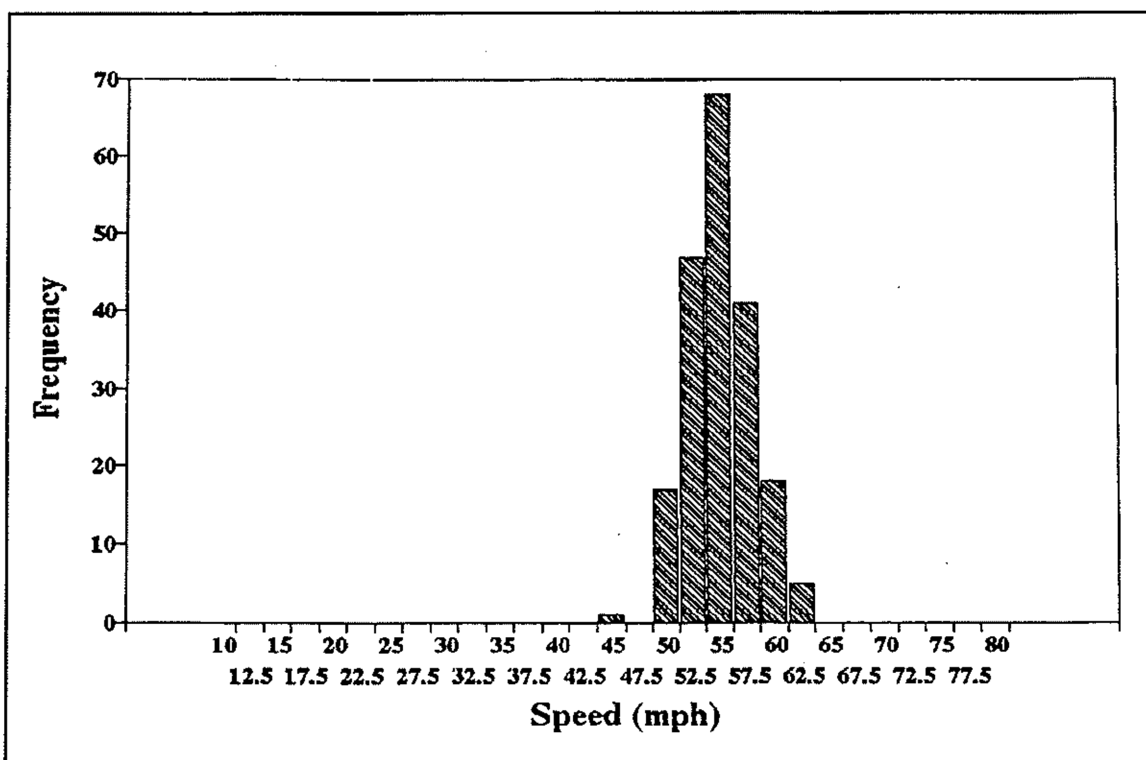

FIGURE 5. DISTRIBUTION OF AVERAGE SPEEDS, KATY FREEWAY HOV LANE. A.M. PEAK PERIOD, WEDNESDAY, JULY 7.

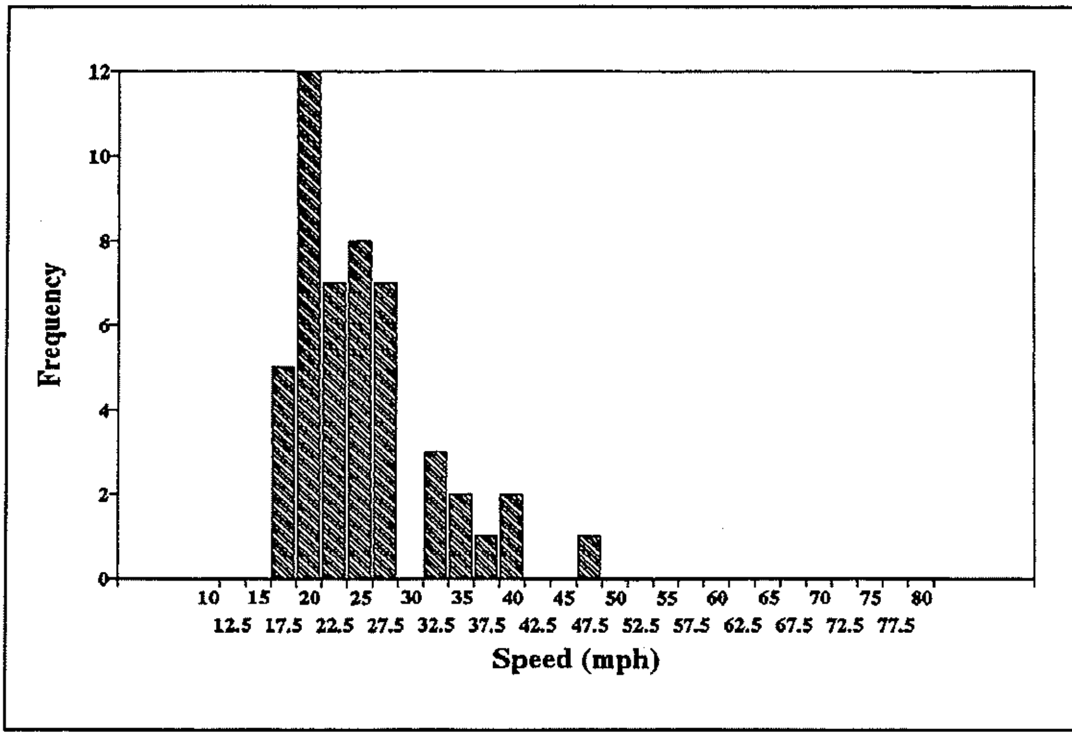

FIGURE 6. DISTRIBUTION OF AVERAGE SPEEDS, KATY FREEWAY GENERAL-PURPOSE LANES. A.M. PEAK PERIOD, WEDNESDAY, JULY 7.

TABLE 2. TEST STATISTICS (T1) ASSOCIATED WITH INDIVIDUAL SAMPLE SETS.

Test	Test Section	Collection Period	T_1
1	Addicks - Gessner	Wed, July 7, AM Peak Period	-6
2	Gessner - Washington	Thurs, July 8, AM Peak Period	-7.4
3	Gessner - Addicks	Wed, July 7, PM Peak Period	-9.3
4	Washington - Gessner	Thurs, July 8, PM Peak Period	-8
5	Jim Miller -CBD	Tues, July 13, AM Peak Period	-5
6	Jim Miller - CBD	Wed, July 14, AM Peak Period	-14
7	CBD - Dolphin	Wed, July 14, PM Peak Period	-17

any comparison based upon conditions other than those involving the variability of travel times within peak periods of travel.

FURTHER RESEARCH

Although this study was limited to peak period conditions, it helped to identify several possible investigations of travel time reliability which could be very beneficial.

An incident occurred during the collection of data along the East R.L. Thornton Freeway during the P.M. peak period. Reference to test number seven in Table 2 shows that the test statistic resulting from the analysis of that period is very low. This indicates that the travel time reliability of HOV lanes may typically be much greater than that of general-purpose lanes during periods affected by incidents. A study to investigate this possibility is recommended, as it could help support the implementation of HOV lanes in corridors where they can improve mobility.

Another interesting and beneficial study could be the comparison of the travel time variability of HOV facilities and general-purpose lanes on a day to day basis. It is suspected that HOV facilities offer the user greater travel time reliability than if general-purpose lanes were used. This investigation would, however, necessitate approximately one-hundred days of data collection and is left for future research.

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Investigation of the Distribution of Taillight and Headlight Heights

C. BRIAN SHAMBURGER

The issue of which object height should be used in calculating stopping sight distance has been a controversial subject for many years. The current Stopping Sight Distance model is based on the detection of a six-inch object. Ideally, parameters should be representative of a realistic driving environment rather than an improbable condition. This study was developed over questions concerning further modifications to the model. A suggested alternative to the current six-inch object is the taillight.

Currently, information is not readily available providing taillight and headlight heights for motor vehicles. Without this data it is difficult to select a representative taillight height to use in the stopping sight distance procedure. It is known that the minimum values for the center of a taillight and headlight heights are 15 inches (38.1 cm) and 22 inches (55.9 cm), respectively. However, how the 15th percentile or average vehicle's taillight and headlight heights differ from the required minimum is not known.

Distributions for taillight and headlight heights were determined in this study. Comparisons were made between the 1993 motor vehicle fleet and a sample a free-flowing vehicles, and *Standard 108* of the *Federal Motor Vehicle Standards* using the center of device.

Fifteenth percentile taillight and headlight heights were noted for both the 1993 motor vehicle fleet and the sample of free-flowing vehicles. Taillight height values for the 1993 fleet and free-flowing vehicles were 73.5 cm (28.9 in) and 73.1 cm (28.3 in), respectively, while headlight height values were 60.5 cm (23.8 cm) and 61.4 cm (24.2 in). Based on a series of statistical tests with a 95% confidence interval, it was concluded that the 15th percentile taillight and headlight heights of the 1993 fleet and the free-flowing vehicles are equal. Using a taillight value of 73.5 cm (28.9 in) in the Stopping Sight Distance model results in a 38 to 43 percent decrease in vertical curve length.

INTRODUCTION

Sight distance is referred to as the length of roadway ahead visible to the driver. For roadway design, it is important that minimum values for sight distance be made available to the driver in all places while on the traveled way (1). In 1940 the American Association of State Highway Officials (AASHO) established a Stopping Sight Distance (SSD) model that is still in use today. Several of the assumed model parameters are presently being questioned concerning the appropriate value that should be used.

The issue of which object height should be used in calculating stopping sight distance has been a controversial subject for many years. The current Stopping Sight Distance model is based on the detection of a six-inch object. Is a six-inch object realistic? Ideally, model parameters should represent a realistic driving environment rather than an improbable condition. Questions concerning object height have led to the development of this study.

Background

Stopping Sight Distance

According to the American Association of State Highway and Transportation Officials' (AASHTO) 1990 *A Policy on Geometric Design of Highways and Streets* (commonly known as the *Green Book*), sight distance is the length of roadway ahead that is visible to the driver. The minimum sight distance available on a roadway should be sufficiently long to enable a vehicle traveling at or near the design speed to stop before reaching a stationary object in its path. Although greater length is desirable, sight distance at every point along the highway should be at least that required for a below-average driver or vehicle to stop in this distance (1,2).

The current procedures for determining stopping sight distance (SSD) are intended to allow a normally alert passenger-car driver, travelling at or near the highway design speed on wet pavement, to react and bring the vehicle to a stop before striking a stationary object in the road. Also under current AASHTO policy, a *critical situation* is defined as an obstacle encountered as the driver is cresting a vertical curve. The object is then recognized as a hazard and the vehicle stops in the required SSD. According to the *Green Book*, a driver travelling between speeds of 48 to 55 mph (77.2 to 88.5 kph) requires approximately 450 to 550 feet (137.2 to 167.6 meters) of visible roadway to stop the vehicle for an object in its path (1). The basic model for these situations was formalized by AASHO in 1940, and over the past 50 years, several of the model's parameters, such as object height, have been modified to account for changes in the vehicle-driver-roadway system.

Object Height

The value for object height within the SSD model was originally set equal to the driver eye height of 5.5 feet (1.7 meters) in 1921 and then drastically reduced to four inches (1.2 cm) in 1940. The last change made was in 1965, when the object height was increased slightly to the present six-inch (1.8 cm) value. The use of a six-inch object was selected as being a reasonable compromise between construction costs and minimal losses in driver visibility. The 1990 and 1984 *Green Books* considered a six-inch object to be "representative of the lowest object that can create a hazardous condition and be perceived as a hazard by a driver in time to stop before reaching it" (1,2,3). Although the possibility of encountering such an object is conceivable, the probability of the driver encountering such an object is low. A previous study found that a driver is 125 times more likely to encounter another vehicle which will result in a reportable accident than a small, unknown object (4).

Visibility

Visibility is an important element to SSD, and the loss of light decreases the length of roadway visible to the driver. At night visibility is limited due to the illumination of the headlight which is approximately 300 feet (91 meters, 4,5).

Viewing a six-inch object is also questionable in relation to driver visual abilities. For the current SSD model a driver with 20/40 static visual acuity is required to see an object 3.5 times smaller than their vision allows at 600 feet (182.9 meters). Why provide additional sight distance to the driver when the driver cannot see the object (2)? If the object height is to be altered, the limiting parameter can be the height of the taillight.

Taillight and Headlight Requirements

Standard 108 of the *Federal Motor Vehicle Standards* (6) provides requirements for lighting equipment and its placement on motor vehicles. Taillights and headlights for all motor vehicles must be located on either side of the vertical centerline of the rear of the vehicle and as far apart as "practicable." Taillights are also required for trailers as well as motor vehicles. Taillights must be mounted no less than 15 inches (38.1 cm) and no higher than 72 inches (182.9 cm). Likewise, headlights may be no less than 22 inches (55.9 cm) and no higher than 54 inches (137 cm). This reference is to be taken as the height above road surface measured from the center of the item on the vehicle. Both the upper and lower beam lamp should be at the same height. Both lighting devices and their height requirements for placement on the vehicle are listed in Table 1.

Problem Statement

Currently, information is not readily available on taillight and headlight heights for motor vehicles. Without this data it is difficult to select a representative taillight height to use in the stopping sight distance procedure. It is known that the minimum values for taillight and headlight heights are 15 inches (38.1 cm) and 22 inches (55.9 cm), respectively (6). The question is to what degree does the 15th percentile or average vehicle's taillight and headlight heights differ from the required minimum.

Objectives

The objective of this research was to determine the distribution of taillight and headlight heights in the 1993 motor vehicle fleet and the current vehicle population. The results from this study were compared to values used in the current AASHTO Stopping Sight Distance model and those governed by the National Highway Traffic Safety Administration *Federal Motor Vehicle Safety Standards*.

FIELD STUDY

To determine the distribution of taillight and headlight heights, two separate samples of vehicles were used. Heights were determined for the 1993 motor vehicle fleet and for a free-flowing volume of vehicles obtained while travelling past a specific site. A listing of the 1993 fleet was found through automotive journals and their listing of sales volumes. This study chose to use only domestic, imported and light-truck vehicles due to the availability of published sales.

TABLE 1. LOCATION OF LIGHTING EQUIPMENT. (6)

Item	Height Requirement*
Headlight	Not less than 22 inches (55.9 cm) nor more than 54 inches (137.2 cm)
Taillight	Not less than 15 inches (38.1 cm) nor more than 72 inches (182.9 cm)

* Height above road surface measured from center of item on vehicle.

Two methods of data collection were chosen for this study. Based on variations in the mounting of the taillight (e.g., later model trucks tend to have a vertical mount and passenger cars are generally mounted horizontally), measurements to the top of device, base of device, center of device and centerline of bulb were determined for the 1993 motor vehicle fleet. For the free-flowing sample, measurements to the top and base of device were recorded due to the ease in distinguishing between the outline of the device and the rest of the vehicle. All measurements are referenced from the pavement surface. Figures 1 and 2 show typical taillight and headlight configurations found in the field.

1993 Motor Vehicle Fleet

To obtain heights for the 1993 fleet, each vehicle was located at auto dealerships. Local automotive dealers in the Bryan/College Station area were initially investigated to determine which vehicles of the 1993 fleet were available. Those vehicles that were not available in the Bryan/College Station area were located in the Dallas Metroplex. The technique involved physically measuring the taillight and headlight of the vehicle with a meter stick. A consistent collection procedure was maintained from one vehicle to the next. From the 1993 fleet, 238 vehicles were located while 11 vehicles from the fleet were not found. These 11 vehicles only represent 0.27 percent of the total number of vehicles sold. Some of these vehicles were actually new 1992 vehicles sold as surplus from the manufacturers inventory.

To determine the distributions of taillights and headlights, sales volumes for the individual vehicles in the 1993 fleet were obtained. The periodical journal, *Automotive News* (7), was the sole source in determining these values. Individual sales for domestic and import cars and light-trucks are published monthly. Determining a current distribution of 1993 vehicles in use is dependent on the date of the initial release of the 1993 motor fleet. Release of the 1993 fleet occurred in August of 1992, therefore sales volumes were acquired from August 1992 to July 1993. A listing of the top ten 1993 vehicles by sales and their respective taillight and headlight heights referenced from the center of device is presented in Table 2.

Determination of 1993 Motor Vehicle Fleet Taillight and Headlight Height Distributions

Cumulative distributions were calculated for taillight and headlight heights of the 1993 motor vehicle fleet. Distributions for the 1993 fleet were determined after a series of matching the measured taillight and headlight height with the individual vehicle sales. It should be reemphasized that four measurements (top, center, base of device and centerline of bulb) were recorded for each taillight and headlight in the 1993 fleet. A distribution was ascertained for each position on the lighting device. This was accomplished by sorting each list in ascending order and calculating a cumulative distribution of heights based on percent of vehicle at or below this height. Figures 3 and 4 show the comparison of distributions determined for taillight and headlight heights measured to the four specified points for the 1993 fleet.

Free-Flowing Vehicle Sample

The study's goal was to have between 100 and 200 vehicles recorded for reduction. Videotape and an image capturing system were employed to determine the taillight and headlight heights of a free-flowing vehicle sample of the general vehicle population. The system requires that a reference marker of known length be placed in the video camera's field of vision. With the taillight or headlight positioned in the same plane as the reference marker, the screen was then calibrated and measurements were made to each lighting device.

Data Collection

The study site consisted of two components: a video camera and a reference marker. The reference

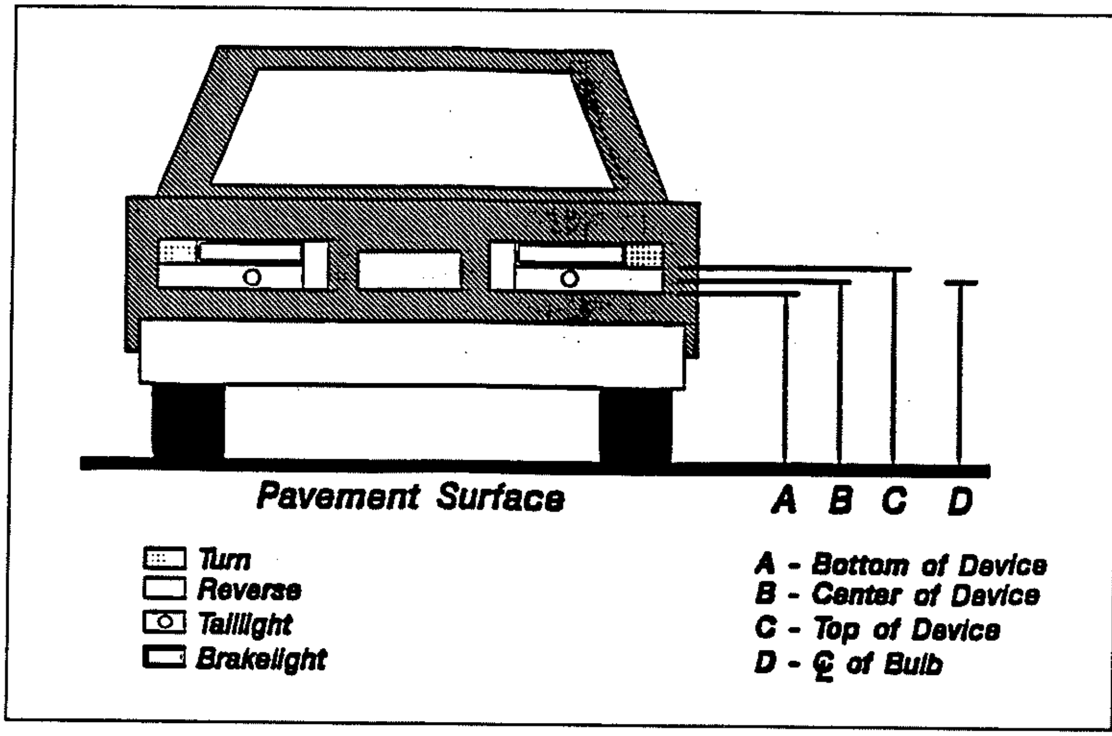

FIGURE 1. TYPICAL TAILLIGHT CONFIGURATION.

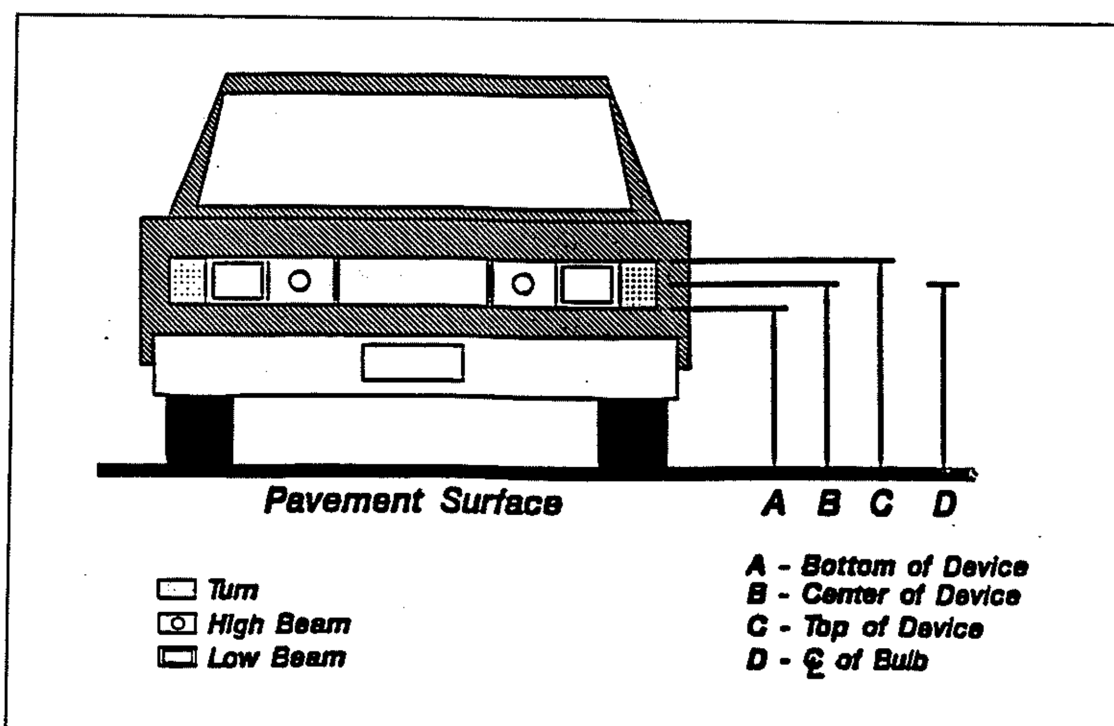

FIGURE 2. TYPICAL HEADLIGHT CONFIGURATION.

TABLE 2. TOP TEN 1993 VEHICLES BY SALES WITH RESPECTIVE TAILLIGHT AND HEADLIGHT HEIGHTS.

Vehicle Make	Vehicle Model	Yearly Sales Aug 92 - July 93	Taillight Height (cm)*	Headlight Height (cm)*
1. Ford	Pickup	512,127	103.0	93.5
2. Chevrolet	Pickup	485,433	103.0	84.5
3. Ford	Taurus	398,577	79.5	64.5
4. Ford	Explorer	295,760	97.0	88.0
5. Honda	Accord	286,795	81.8	59.5
6. Ford	Ranger	267,463	96.0	77.0
7. Dodge	Caravan	255,447	83.5	77.5
8. Ford	Escort	243,774	73.5	62.0
9. Chevrolet	Cavalier	242,009	80.5	61.5
10. Chevrolet	Lumina	222,519	78.0	64.5
All Other Vehicles		10,123,492	-	-

* Height referenced to the center of device

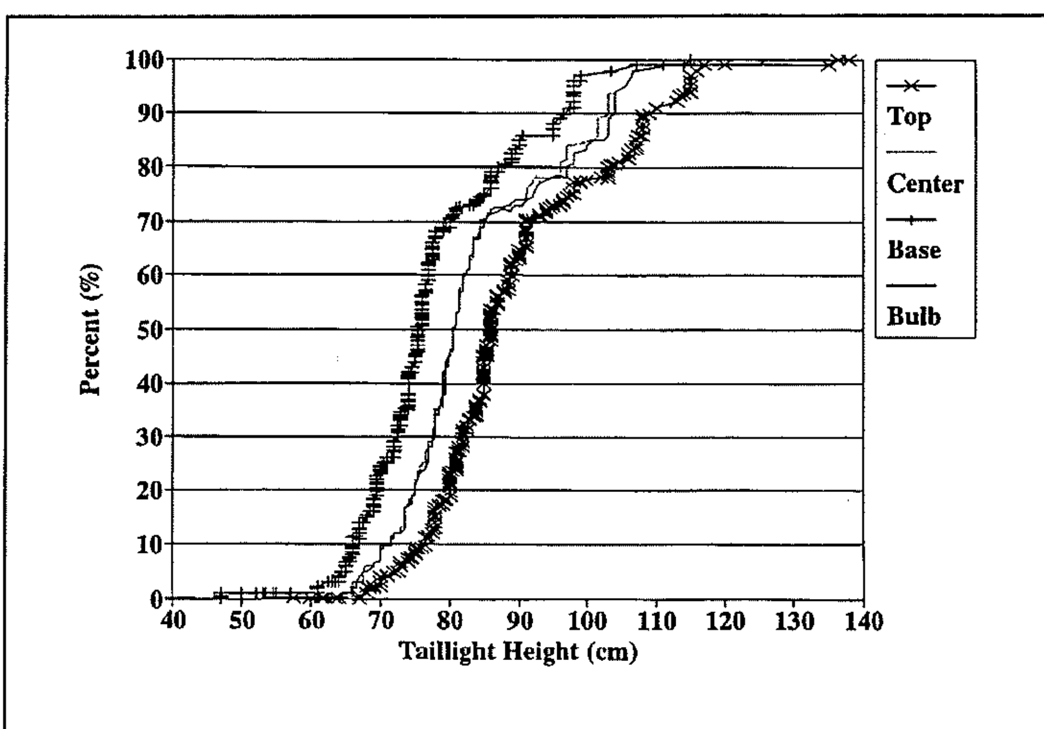

FIGURE 3. 1993 MOTOR FLEET TAILLIGHT DISTRIBUTIONS.

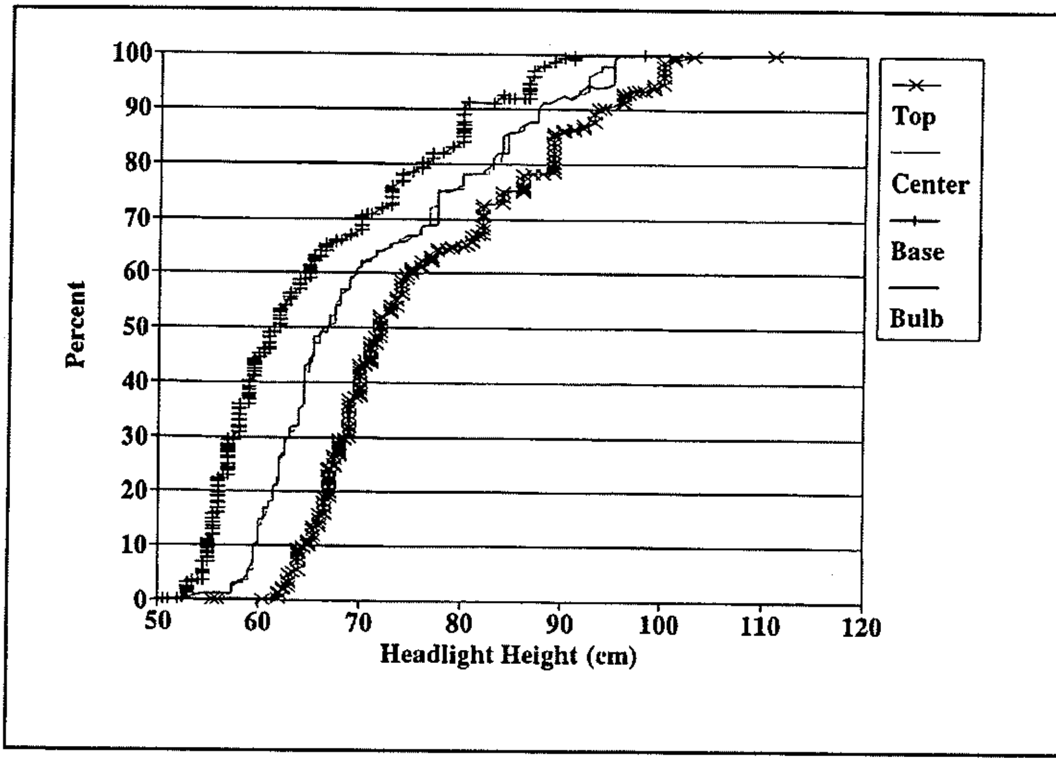

FIGURE 4. 1993 MOTOR FLEET HEADLIGHT DISTRIBUTIONS.

marker can be anything that has a known length such as a yard or meter stick. Four-inch (10.2 cm) wide temporary pavement striping was placed across the entire width of the roadway. This was the most logical alternative since the entire length of the striping could be seen by a single camera at short distances [less than 50 feet (15.2 meters)]. By placing the reference marker on pavement, the taillight or headlight could be positioned over the striping with ease. Procedures mandated that all video taping be performed 90° to the reference marker. Vehicles were filmed as they travelled over the reference marker. A layout of the observation site is shown in Figure 5.

Data Reduction

With the use of a frame editor, the original videotape was rerecorded into individual frames pausing the taillight and headlight directly over the pavement marking. This rerecorded tape was then used with the image capturing system. The image capturing system converts the paused video image into a computer image. It then uses the endpoints of the reference marker (which are selected by the user) to calibrate the dimensions on the screen. All taillight and headlight measurements were referenced from the roadway surface, which is a single point along the reference marker directly under the taillight or headlight. After determining the top and base of each device, an average value between the two was calculated to determine the center of the device.

Determination of Correction Factor

An observation test site was created to have vehicles with known taillight and headlight heights drive through for recording. A subject pool of 15 drivers was contacted and scheduled to drive their vehicle past the site. Before each motorist drove their car through the test site, measurements were taken to the top and bottom of both the taillight and headlight. The drivers were then instructed to proceed past the reference marker and video camera setup. Analysis was performed on the videotape to compare the actual heights to those found with the image capture system. Measurements by the system were consistently on the low side of the actual heights. An average correction factor of 1.05 was determined to be applied to further measurements made by the system.

Determination of Free-Flowing Vehicle Taillight and Headlight Distributions

Procedures for calculating the taillight and headlight distributions for the free-flowing sample were similar to

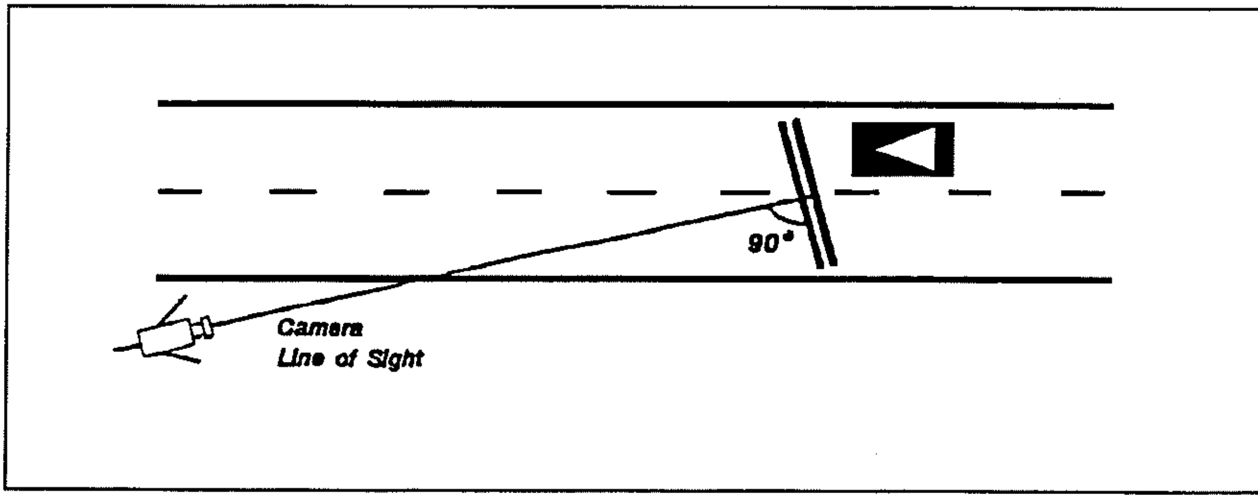

FIGURE 5. OBSERVATION SITE LAYOUT.

those for the 1993 motor fleet. The only difference being that there were no measurements made to the center of bulb and that each vehicle had equal weight within the sample. Figures 6 and 7 show the comparison of distributions of taillight and headlight heights for the free-flowing vehicle sample.

COMPARISON OF FINDINGS

1993 Motor Vehicle Fleet

Center of Bulb and Center of Device

When viewing a taillight or headlight, the point of highest intensity is the center of the bulb. This point is commonly referred to as the "hot spot." References made to both lighting devices are frequently directed to this point. The 15th percentile taillight heights for the center of device and center of bulb for the 1993 fleet are equal at 73.5 cm (28.9 in). Similarly, the 15th percentile headlight heights for the center of device and center of bulb for the same fleet are again equal at 60.5 cm (23.8 in, see Figures 3 and 4). The 15th percentile was selected because it is a conservative estimate for a representative taillight and headlight height of the general vehicle population.

Comparison of 1993 Motor Vehicle Fleet, General Vehicle Population & Required Standards

Developing a relationship between the distributions of lighting devices for the 1993 motor vehicle fleet and the general vehicle population was an initial objective of this study. It is important to know whether or not the inflow of new vehicles into the traffic stream will have a significant influence in raising or lowering the 15th percentile or average vehicle's taillight or headlight height. To determine this, a comparison was made for both taillights and headlights between the 15th percentile values for center of bulb for the 1993 fleet and center of device for the free-flowing vehicles.

Comparison of Taillight Heights

In order to determine whether or not the center of bulb for the 1993 fleet equals the center of device for the free-flowing vehicles, a statistical analysis between the two values is required. The statistical analysis chosen for this comparison was the two-sample t test using a confidence interval of 95 percent. The null hypothesis of interest is H_0: $X_1 - Y_1 = 0$. X_1 and Y_1 represent values for 15th percentile taillight heights at the center of bulb for the 1993 fleet and the center of device for the free-flowing vehicles. Values for X_1 and Y_1 are 73.5 cm (28.9 in) and 73.1 cm (28.3 in), respectively. At a significance level of 0.05, H_0 will be rejected if the test statistic value is less than -1.645 or greater than 1.645. The analysis resulted in a test statistic value of 0.2929. Since 0.2929 is neither less than -1.645 nor greater than 1.645, H_0 cannot be rejected at level 0.05. This indicates that the center of bulb taillight height for the 1993 fleet is similar to the center of device taillight height for an average vehicle in the general vehicle population. A graphical comparison of the center of bulb for the 1993 fleet and center of device for the free-flowing vehicles is shown in Figure 8. Figure 8 also shows the provisions of *Standard 108*.

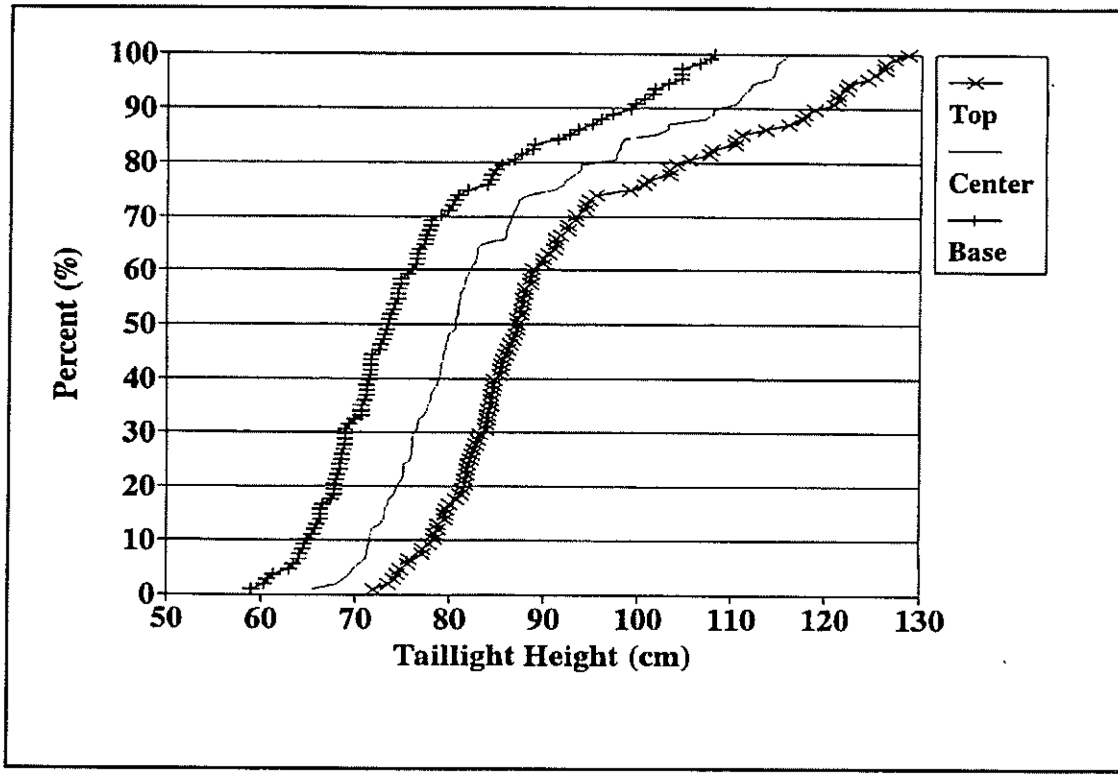

FIGURE 6. FREE-FLOWING VEHICLES TAILLIGHT DISTRIBUTIONS.

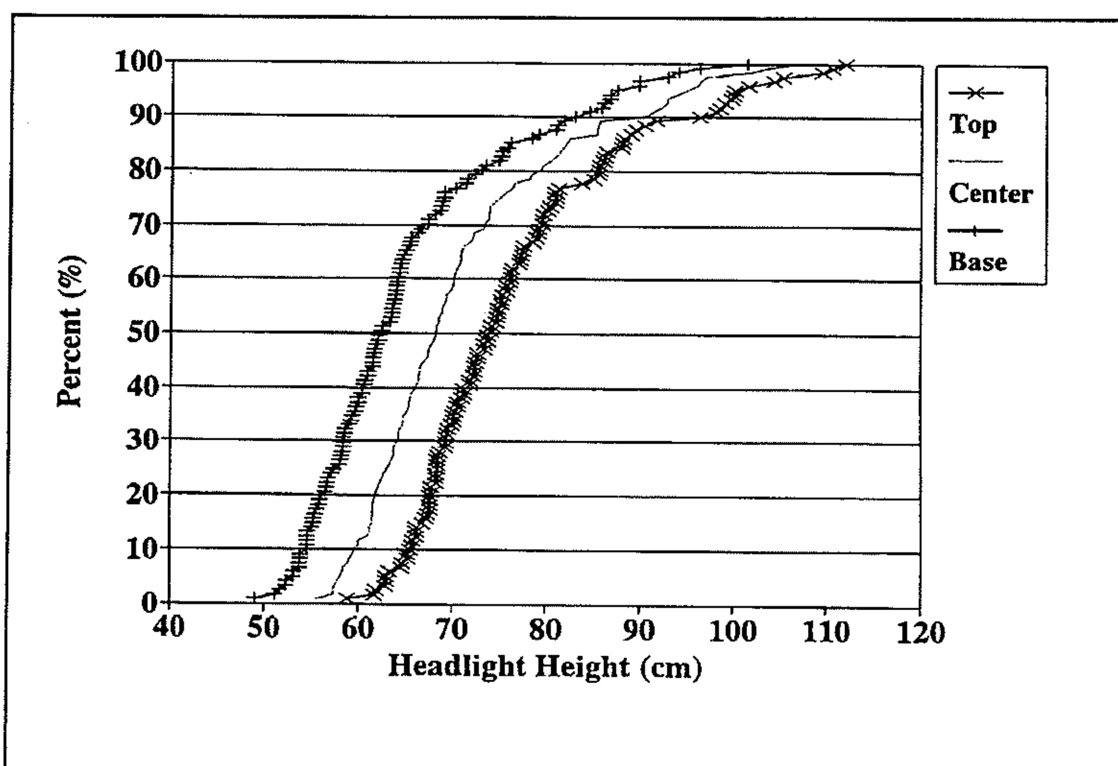

FIGURE 7. FREE-FLOWING VEHICLES HEADLIGHT DISTRIBUTIONS.

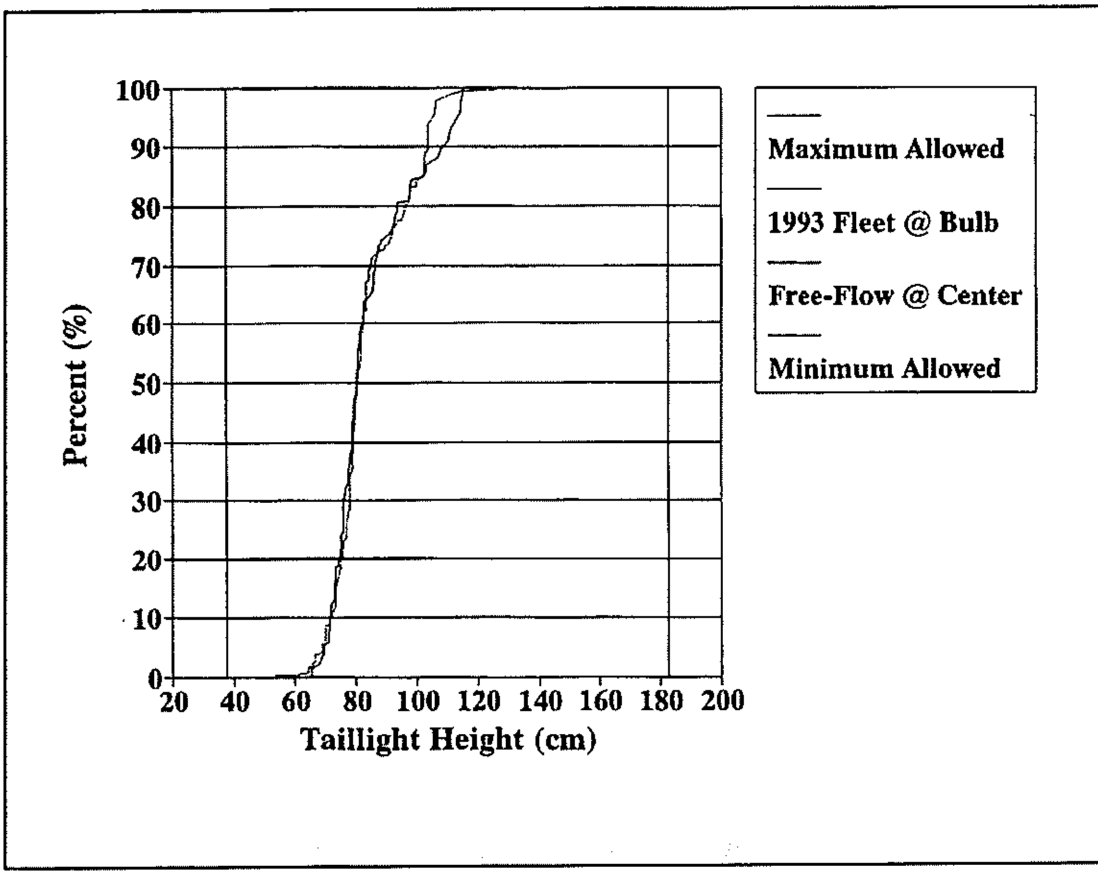

FIGURE 8. COMPARISON OF TAILLIGHT HEIGHT FINDINGS.

Comparison of Headlight Heights

Again, a *t* test was used to verify the equality between the center of bulb for the 1993 fleet and center of device for the free-flowing vehicle sample. Values for 15th percentile headlight heights were 60.5 cm (23.8 in) for the center of bulb and 61.4 cm (24.2 in) for the center of device. With the same parameters as the previous analysis the test statistic value was -1.096. At a significance level of 0.05 (i.e., P-value > 0.05) the null hypothesis that the two values are equal is not rejected. Figure 9 plots both distributions along with the corresponding requirements mandated by *Standard 108*. A summary of the findings comparing the distributions of taillight and headlight heights for the 1993 motor vehicle fleet and the free-flowing vehicle sample to *Standard 108* can be found in Table 3.

APPLICATION TO STOPPING SIGHT DISTANCE MODEL

The basic formulas for length of a parabolic vertical curve in terms of algebraic difference in grade and sight distance follow:

When S is less than L, $L = \dfrac{As^2}{100\times(\sqrt{2h_1}+\sqrt{2h_2})^2}$

(1)

When S is greater than L, $L = 2S - \dfrac{200(\sqrt{h_1}+\sqrt{h_2})^2}{A}$

(2)

where: L = length of vertical curve, ft;
S = sight distance, ft;
A = algebraic difference in grades, percent;
h_1 = driver eye height above roadway surface, ft.
h_2 = height of object above roadway surface, ft.

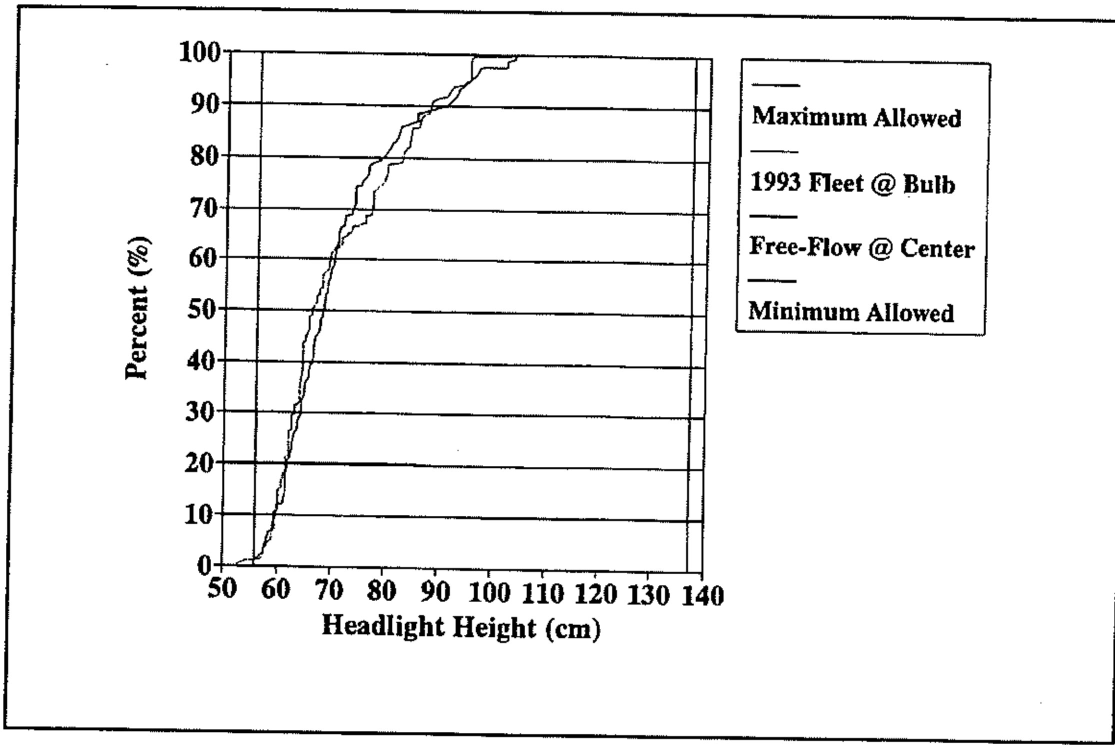

FIGURE 9. COMPARISON OF HEADLIGHT HEIGHT FINDINGS.

TABLE 3. COMPARISON OF 1993 FLEET, FREE-FLOWING, & STANDARD 108 FINDINGS.

Range	Vehicle Lighting Equipment Height (cm)					
	Taillight			Headlight		
	1993 Fleet	Free-Flowing	*Standard 108*	1993 Fleet	Free-Flowing	*Standard 108*
Minimum	53.8	65.5	38.1	53.0*	55.7	55.9
15th	73.5	73.1		60.5	61.4	
50th	80.5	80.6		66.8	68.3	
Maximum	126.5	115.9	182.9	105.5	106.8	137.2

* 0.005% of the 1993 vehicle fleet sold is below the *Standard 108* minimum value of 55.9 cm.

The current value for object height, h_1, within the Stopping Sight Distance model is 6.0 inches. A viable option for the replacement of the current 6.0 inch object height is the 15th percentile taillight height, referenced from the center of the device, for the general vehicle population. This would signify an increase from 6.0 inches (15.24 cm) to 28.8 inches (73.1 cm). If the value for object height is increased, the length of vertical curve will inversely decrease. Table 4 compares vertical curve lengths determined by the existing SSD model and those calculated from the application of the proposed changes in object height. Assumptions made for this example include (1) A is 6.0 percent and (2) sight distance is assumed to be less than the length of curve, L. Figure 10 illustrates the results of Table 4.

SUMMARY AND CONCLUSIONS

Distributions for taillight and headlight heights for the 1993 vehicle fleet and for a sample of free-flowing vehicles were determined in this study. These distributions were compared to the *Standard 108* of the *Federal Motor Vehicle Standards*. *Standard 108* is based on taillight and headlight height measurements referenced to the center of the device on the vehicle. To ensure consistency, distributions for the center of device were used. With the exception of a single vehicle (representing 0.005 percent of the 1993 vehicle fleet sold), both distributions fell within the range of minimum and maximum allowed taillight and headlight heights provided by *Standard 108*.

Fifteenth percentile taillight and headlight heights were determined for both the 1993 motor vehicle fleet and the sample of free-flowing vehicles. Taillight height values for the 1993 fleet and free-flowing vehicles were 73.5 cm (28.9 in) and 73.1 cm (28.8 in), respectively, while headlight height values were 60.5 cm (23.8 in) and 61.4 cm (24.2 in). Based on a series of statistical tests with a 95 percent confidence interval, it was concluded that the 15th percentile taillight and headlight heights of the 1993 fleet and the free-flowing vehicles are equal. Using a taillight value of 73.5 cm (28.9 in) in the Stopping Sight Distance model results in a 38 to 43 percent decrease in vertical curve length.

RECOMMENDATIONS FOR FURTHER RESEARCH

A limitation of this study was the number of free-flowing vehicles measured. Data from additional free-flowing vehicles will increase the reliability of the findings of this study. The video data collection procedures used in this study could be applied to other vehicle dimension-related studies (for example, a driver eye height study to determine a typical value for driver eye height). A visual angle study that determines the partial height of a taillight required by a driver to detect and recognize the rear of a vehicle could assist in the selection of an appropriate taillight height to be used in SSD calculations. Studies on visibility range provided by headlights and/or taillights during nighttime operations could also assist in selecting an appropriate object height.

ACKNOWLEDGEMENTS

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TABLE 4. VERTICAL CURVE LENGTH FOR PROPOSED OBJECT HEIGHT.

Design Speed (mph)	Vertical Curve Length (ft)*		Percent Change (%)
	Existing Model	With Changes	
30	173	103	-41
35	279	160	-43
40	444	271	-39
45	663	411	-38
50	962	579	-40
55	1308	726	-41
60	1816	1084	-40
65	2370	1349	-43
70	3190	1584	-42

* Assumed object height = 28.8 inches, A = 6 percent, driver eye height of 3.5 feet, Sight distance is less than length of vertical curve, and desirable design speed.

Predicting Air Quality and Fuel Consumption Benefits From HOV Facilities

EDWARD D. SULAK, JR.

This study is intended to determine the impacts on air quality and fuel consumption from various freeway congestion improvements. Three corridors in the Houston area were analyzed using a freeway simulation model. The output from each of the models was used to make a comparative analysis between not only the different cases for each of the three freeways, but it also enabled the study to contain an analysis from one freeway to the next to determine if a particular scenario was truly an advantage.

It was found that the exclusive, barrier separated HOV lane yielded the best results for the more severely congested freeways. The moderate to severely congested freeways did provide evidence that a 10% increase in the capacity may have given better results. However, the evidence was not as extensive as the case with the severely congested freeways and requires additional research.

INTRODUCTION

Due to increasing environmental concerns in this country, there has been more of an effort to deal with many of the problems which have been identified. The transportation field has been held responsible for many of the present day air quality and fuel consumption problems. There has been a substantial amount of legislation dealing with these issues within the last five years. The Clean Air Act Amendments (CAAA) of 1990 and the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991 are two such measures which are bringing about more environmental awareness for those involved with transportation. The CAAA provides many suggestions on how different transportation agencies should go about reducing the emissions of mobile sources (1). It is through the funding of the Congestion Mitigation/Air Quality Improvement Program and other sections of ISTEA that a significant portion of the CAAA will be carried out (1).

The Clean Air Act sets specific targets to reduce tailpipe emissions, calls for cleaner fuels and requires new pollution controls on automobiles (2). These measures will especially be seen in the areas which are considered to have significant air pollution problems. In Texas for instance, there are four major population centers which have been identified as problem areas. These centers include Houston/Galvaston, Dallas/Fort Worth, Beaumont/Port Arthur and El Paso. All of these have levels of pollutants which are considered a threat to public health (2).

Three aspects of transportation that are sources of environmental concern include traffic congestion, mobile source emissions, and energy consumption. Traffic congestion is the source of both a site specific and an area wide problem; higher emission levels can be found in the areas where congestion occurs and area wide problems are a result of mobile emissions which, in part, are related to congestion. In addition, fuel consumption is greater in areas of stop-and-go traffic congestion.

The higher populated areas of the country are especially known for poor air quality and inefficient fuel consumption. Areas such as Los Angeles, New York City, and Houston would be included in this category. Many of the problems that the CAAA and ISTEA consider are a direct result of the traffic congestion which is found in these areas. With vehicles driving in congested conditions during the peak traffic periods, the emissions and the fuel consumption values are much larger when compared to areas with limited mobility problems. One element of air quality and fuel consumption improvements is to implement transportation operations which will provide reductions in the travel delay.

Traffic Congestion

Congestion is defined as travel time or delay in excess of that normally incurred under light or free-flow travel conditions (3). To deal with the problems of poor air quality and inefficient fuel consumption caused by freeway congestion, various options must be evaluated to determine what type of an alternative is best. It must be well suited to deal with the environmental concerns, as well as having the capability to provide other beneficial results.

PROBLEM STATEMENT

The air quality and fuel consumption problems can be dealt with effectively if we look at traffic congestion improvements and determine the effects that these improvements have on air quality and fuel consumption. There is a lack of research on the effects of traffic congestion improvements on air quality and fuel consumption. Since this information is available it is just a matter of obtaining the data and performing the analysis on the outcome. TTI has data on the Houston HOV lane system for both the before and after scenarios and can be used to determine the impacts of using an HOV lane, as opposing to not making any operational improvements. Once this analysis has been performed the effects of other types of traffic congestion alternatives can be investigated to determine if these cases give any better results.

OBJECTIVE

The purpose of this research report is to perform an analysis of the Houston HOV lane system to determine the impacts of the travel mode changes that have taken place relative to the case of no operational improvements. Air quality and fuel consumption benefits will then be estimated, and compared to other improvement alternatives.

METHODOLOGY

Freeway Models

Freeway simulation models can provide information on the effects of different geometric and operational improvements. For each of the freeways under investigation there were six separate scenarios modeled to give a variety of options. All of the freeways which were used in the study had at least one of the six scenarios set up at the Texas Transportation Institute (TTI) prior to the beginning of the research.

The three Houston freeways used in the study included the Katy, Gulf, and North Freeways (Figure 1). All of these freeways carry large volumes of traffic, and, as a result, have had problems with traffic congestion. To address the problems that have occurred, the Houston area has implemented HOV lanes on these freeways. This is advantageous to the study because it will provide three different locations with varying freeway characteristics to analyze.

Installation of Model

The FREQ10 model was used to simulate the inbound freeway lanes from 6:00 a.m. to 12:00 noon. FREQ10 is a macroscopic computer simulation model that uses traffic volumes and geometric characteristics to estimate freeway operating conditions. The model performs Highway Capacity Manual calculations to determine such characteristics as speed, volume-to-capacity ratios, queue lengths, fuel consumption, vehicle emissions, and many other characteristics. The data entry is simplified by the use of input screens which provide a menu driven format which relieves the user from having to look up columns of numbers (4). The primary data which is needed for the model to run effectively includes:

- Mainlane entrance volume - volume of all lanes at the beginning of the model
- Mainlane exit volume - volume of all lanes at the end of the model
- Entrance and exit ramp volumes - volumes on the ramps of the freeway
- Mainlane and ramp capacities - the capacities for the mainlane and the ramps
- Percentage of trucks in the traffic flow
- Percentage of trucks that are diesel
- Freeway geometry:
  1. Number of lanes - number of lanes for each section of the freeway
  2. Percent grade of roadway
  3. Length between subsection limits - freeway model is broken into subsections by length
  4. Ramp locations - usually signify the end or the beginning of a subsection
  5. Number of lanes on ramp

Secondary data which can be added to enhance the modeling of the freeway includes:

- Arterial roadway information - to determine how the arterial effects the freeway
- Activation of weaving analysis - performs weaving analysis along the course of the model
- Growth factor - uniformly increases (or decreases) all of the demands of the model
- HOV operational data - characteristics of the HOV lane (if required)
- User supplied speed-v/c curves - values which are characteristic of the Houston area
- User supplied fuel consumption rates-speed curves - characteristic of the Houston area

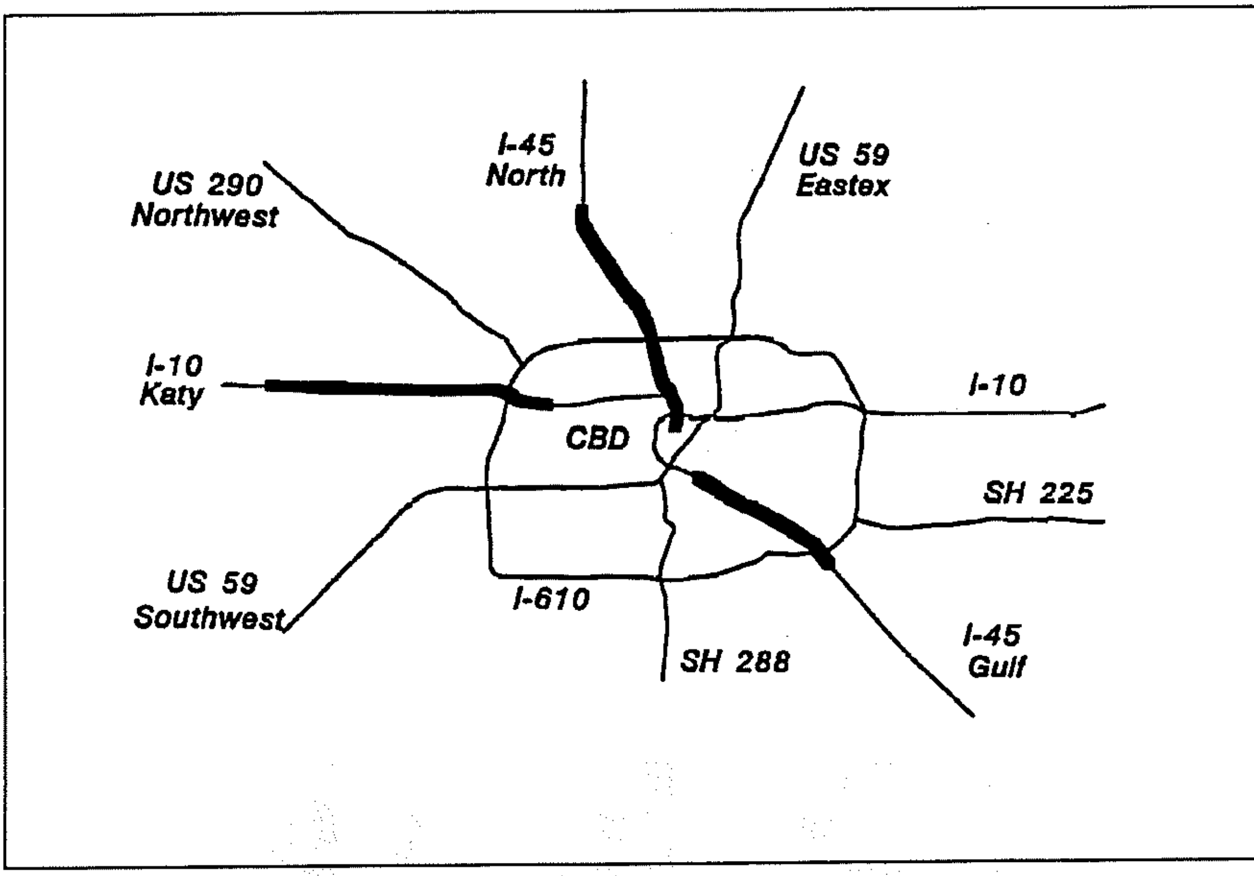

FIGURE 1. HOUSTON MAP DISTINGUISHING KATY, GULF, AND NORTH FREEWAYS.

- User supplied emission rates-speed curves - characteristic of the Houston area
- Spatial shift data - to determine driver's response to improvements in the freeway
- Modal shift data - to determine the amount of persons that choose a different means of travel
- Cost-benefit coefficients - used to formulate a performance index of the freeway

The models which were available at TTI for the Katy, Gulf, and North freeways are described in Table 1, with each of the three freeways having the same scenarios. The table is broken up case-by-case into the first two options which were considered. Each case is further explained by the alternative, or the description of the case. This is then followed by a brief description of the model which was used to represent each case.

The first case includes a model without any improvements made to the freeway. This case is shown in Figure 2A for the Katy Freeway. As seen in the figure there are a total of three lanes with no HOV lanes present. Case 1 is the initial version of TTI's freeway model before an HOV lane was implemented, with the exception of applying a growth factor to the initial volumes to bring the values to the present date. This was done for all of the models so that each case could be compared at an equivalent point in time. For example, the impacts of the do nothing case modeled for 1980 cannot be compared on an equal basis with Case 2 modeled in 1993.

Case 2, as depicted in Figure 2B, is the exclusive HOV lane alternative. To represent this case, TTI actually set up two different FREQ models, one for each roadway, and took the summation of both model's output to determine the effects of the system. This is due to the fact that FREQ10 assumes that there is no barrier between the HOV lane(s) and the mixed-flow lanes (5), which is not the case for any of the HOV lanes in the Houston area. Once again, the volumes for these models had to be adjusted by a growth factor to bring the models to the present date.

Each of the TTI models was provided with the primary input data and also included some of the data from the secondary list such as a user supplied speed-v/c curve, a fuel-speed curve, and an emission rate-speed

TABLE 1. DESCRIPTION OF AVAILABLE MODELS.

CASE	ALTERNATIVE	DESCRIPTION OF MODEL
1	Do nothing	No improvements to the freeway
2	Exclusive HOV lane	Summation of two models: (1) Mainlane model (2) HOV lane model

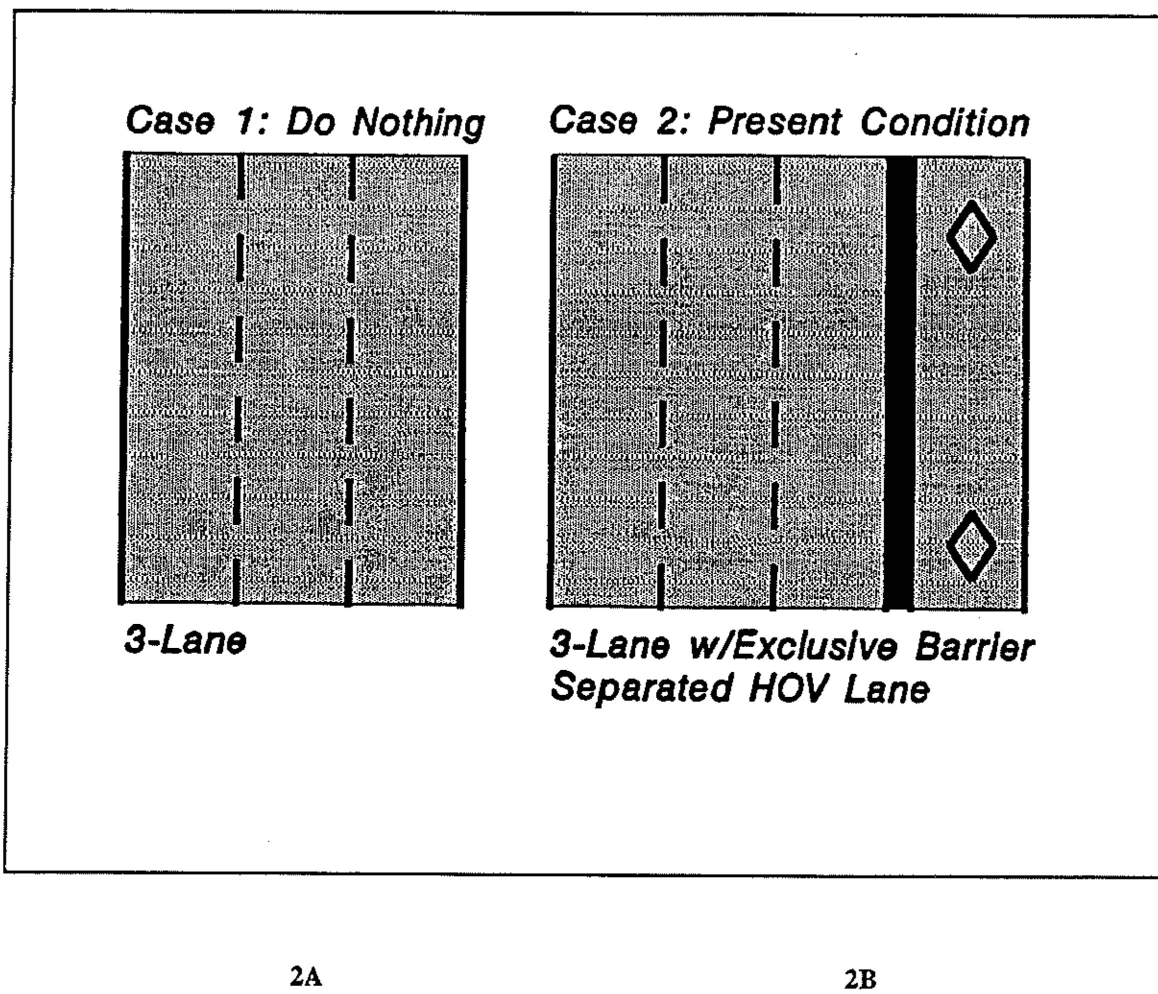

FIGURE 2. KATY FREEWAY ALTERNATIVES (CASES 1 AND 2).

curve, characteristic of the Houston area. If input data was not available for any of the secondary data values listed earlier then the default values of the program were used, except in two cases (Figure 3).

The first case where the default value was not used was the disengage weaving analysis option. The weaving analysis in the FREQ program uses the algorithm from the 1965 Highway Capacity Manual (4). This calculation limits capacity through the weaving sections more than has been observed in Houston. The weaving analysis was therefore disengaged for this study.

The growth factor was the second value which was not available from any of the previous models done at TTI. This value was important due to the fact that it enabled all of the scenarios to be shifted to the present day conditions. For example, if the model for the case prior to HOV lane use on the Katy Freeway was based on volumes which were determined in 1980, then the growth factor had to be used to adjust the volumes to approximate the values which would be present, the day the study was made. The growth factors were obtained from Texas Department of Transportation district vehicle volume maps by calculating the average percentage increase in volume per year, from the year the model was created to the present year, 1993.

Other than the option to disengage the weaving analysis and the growth factor option, all of the other information was simply transferred from the TTI models, which were set up in FREQ7 on a mainframe computer, to the FREQ10 PC version of the model. After Case 1, the do nothing alternative, was established for each of the three freeways, it was then used as a base for the other cases under investigation. Adjustments were made to the Case 1 alternative to represent other freeway improvements. The remainder of the cases are shown in Table 2 along with the first two cases discussed.

In Case 3, one general purpose lane was added to every subsection of the model as well as an additional 1800 vehicles were added to the capacity (Figure 4). The same process was carried out for Case 4, except that 2 general purpose lanes were added with a corresponding 3600 vehicle capacity increase to the Case 1 model (Figure 4). In Case 5, the take-a-lane alternative, a general purpose lane was taken out of the geometry of the original model, and an HOV lane was set up within the same model (Figure 4). The sixth case was set up by adding 10 percent capacity to the freeway. This would be a result of operational improvements to the freeway (6).

Once the models were set up for the six scenarios on the three different freeways the models were then run and output was collected for all of the cases run. The FREQ modeling program does provide the user with various types of output information to choose from. For the purposes of this study, the output data which was used included the travel time, the travel distance, the average speed, the gasoline consumed, the hydrocarbon emissions, the carbon monoxide emissions, and the nitrous oxide emissions.

RESULTS

Following the completion of the FREQ10 modeling, the output was collected and is presented in Tables 3 - 8. The results were then analyzed to determine the impacts of each alternative on air quality and fuel consumption.

The FREQ10 model can give detailed output for each of the subsections or a summary of the subsections can be selected out of the options which are available in the output section of the program. The results are summarized in the tables below (Tables 3, 5, 7). Before an accurate analysis could be carried out, the output had to be adjusted to compare all of the values on an equal basis. This is done by holding the demand level constant. Since the demand is measured in passenger miles, this was carried out by determining the factor by which it took to multiply the passenger miles, so that the value that is left is equal to an arbitrary value for passenger miles for each of the three freeways. This factor was then used to multiply by all of the other values in the output tables (excluding average speed) for each of the six cases. The adjusted totals used for the analysis are represented in Tables 4, 6, and 8 for the three freeways.

Case 5 for two of the three freeways was unable to give any output using the FREQ models. The reasons for the inability to provide output are not known and further study into the models would have to be undertaken to determine this. The Gulf Freeway did provide output for the take a lane option and was used in the analysis of the results. This one model may be adequate to determine if this case is considered more or less of an advantage, however there is a large degree of uncertainty in using the results of only one of the three freeways to be representative of this alternative when a comparative analysis is being performed. Therefore other models need to be established to determine if the results of the one model are reasonable or not.

The adjusted values for fuel consumption and air quality measures are shown for each of the three freeways (Figures 5-8). These values are grouped in such a way that the cases can be compared against each other for not only one of the freeways used in this study, but to also determine if this trend was similar for the three freeways under investigation. It could be

```
                         OPTIONAL DATA

          To select  type  1         To return to default type  0

Engage arterial              : 0

Disengage weaving analysis   : 1

Calculate mainline delays    : 1       Speed used for calculations: 55

Engage freeway growth factor : 1       Freeway growth factor      : 1.08
(0=no, 1=constant, 2=by ramp)

User supplied speed curves   : 1       Default speed curve        :  1
                                        (50 55 60 65 70)
                                        (user curve(s) 1)
User supplied fuel rates     : 1

User supplied emission rates : 1


F1: EDIT SCRN  F2: HELP  F3: ARTERIAL  F4: RAMP GROWTH  F5: SPEED CURVES
F6 FUEL RATES  F7: EMISSION RATES  F10: EXIT            Choose your option:
```

FIGURE 3. FREQ10 OPTIONS SCREEN-KATY FREEWAY MODEL (CASE 1).

TABLE 2. DESCRIPTION OF ALL FREEWAY IMPROVEMENT ALTERNATIVES.

CASE	ALTERNATIVE	DESCRIPTION OF MODEL
1	Do nothing	No improvements to the freeway
2	Exclusive HOV lane	Summation of the two models: (1) Mainlane model (2) HOV lane model
3	Add 1 lane	Case 1 with 1 lane added and a capacity increase of 1800 vehicles
4	Add 2 lanes	Case 1 with 2 lanes added and a capacity increase of 3600 vehicles
5	Take-a-lane	Case 1 with 1 lane taken out and the FREQ-HOV lane added
6	Add 10% capacity	Case 1 with an additional 10% capacity

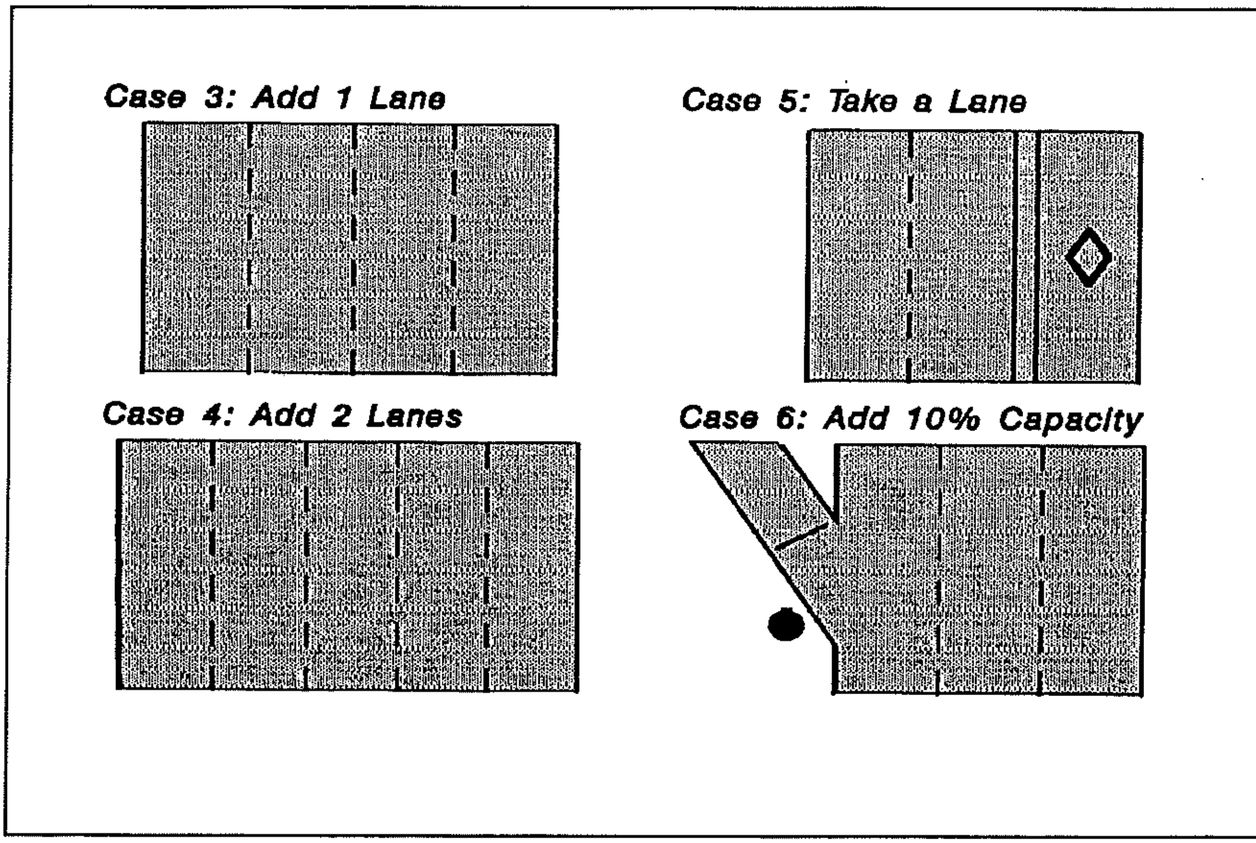

FIGURE 4. KATY FREEWAY MODELS (CASES 3-6).

TABLE 3. UNADJUSTED OUTPUT FOR THE KATY FREEWAY.

	Do Nothing Case 1	HOV Lane Case 2	Add 1 Lane Case 3	Add 2 Lanes Case 4	Take Lane for HOV Case 5	Add 10% Capacity Case 6
Total Travel Time, Veh-Hours	40829	20084	25371	17636	*	40337
Total Travel Time, Pas-Hours	51395	31493	32010	22271	*	50779
Total Travel Distance, Veh-Miles	393167	490665	519875	513609	*	452930
Total Travel Distance, Pas-Miles	495018	797623	654545	646657	*	570280
Average Speed, MPH	17.3	30.6	30.1	55.9	*	21.2
Gasoline Consumed, Gallons	30864	29736	35347	39922	*	33988
Hydrocarbon Emissions, Kg	2141	1296	1678	1479	*	2253
Carbon Monoxide Emissions, Kg	18711	8750	12362	10456	*	18962
Nitrous Oxide Emissions, Kg	1274	1211	1439	1660	*	1406

* Output not provided

TABLE 4. ADJUSTED VALUES FOR THE KATY FREEWAY.

	Do Nothing Case 1	HOV Lane Case 2	Add 1 Lane Case 3	Add 2 Lanes Case 4	Take Lane for HOV Case 5	Add 10% Capacity Case 6
Total Travel Time, Veh-Hours	53612	16367	25195	17727	*	45976
Total Travel Time, Pas-Hours	67486	25664	31788	22386	*	57877
Total Travel Distance, Veh-Miles	516261	399853	516266	516264	*	516246
Total Travel Distance, Pas-Miles	650000	650000	650000	650000	*	650000
Average Speed, MPH	1703	30.6	30.1	55.9	*	21.2
Gasoline Consumed, Gallons	40527	24233	35102	40128	*	38739
Hydrocarbon Emissions, Kg	2811	1056	1666	1487	*	2568
Carbon Monoxide Emissions, Kg	24569	7131	12276	10510	*	21613
Nitrous Oxide Emissions, Kg	1673	987	1429	1669	*	1603

* Output not provided

TABLE 5. UNADJUSTED OUTPUT FOR THE GULF FREEWAY.

	Do Nothing Case 1	HOV Lane Case 2	Add 1 Lane Case 3	Add 2 Lanes Case 4	Take Lane for HOV Case 5	Add 10% Capacity Case 6
Total Travel Time, Veh-Hours	21973	18960	12972	12532	18694	7072
Total Travel Time, Pas-Hours	23564	20853	13911	13439	20047	7584
Total Travel Distance, Veh-Miles	387075	401868	430810	430573	257009	357182
Total Travel Distance, Pas-Miles	415092	460998	461992	461738	275611	383035
Average Speed, MPH	21.4	41.6	55.2	57.5	18.8	56.8
Gasoline Consumed, Gallons	25550	26611	32433	32985	17781	25797
Hydrocarbon Emissions, Kg	1254	1183	1130	1126	1004	747
Carbon Monoxide Emissions, Kg	9257	8313	7198	7656	7971	4244
Nitrous Oxide Emissions, Kg	960	1019	1269	1382	674	1028

TABLE 6. ADJUSTED OUTPUT FOR THE GULF FREEWAY.

	Do Nothing Case 1	HOV Lane Case 2	Add 1 Lane Case 3	Add 2 Lanes Case 4	Take Lane for HOV Case 5	Add 10% Capacity Case 6
Total Travel Time, Veh-Hours	21703	16863	11512	11128	27809	7570
Total Travel Time, Pas-Hours	23275	18546	12345	11933	29822	8118
Total Travel Distance, Veh-Miles	382327	357411	382327	382327	382328	382327
Total Travel Distance, Pas-Miles	410000	410000	410000	410000	410000	410000
Average Speed, MPH	21.4	41.6	55.2	57.5	18.8	56.8
Gasoline Consumed, Gallons	25237	23667	28783	29289	16451	27613
Hydrocarbon Emissions, Kg	1239	1052	1003	1000	1494	800
Carbon Monoxide Emissions, Kg	9143	7393	6477	6798	11858	4543
Nitrous Oxide Emissions, Kg	948	906	1126	1227	1003	1100

TABLE 7. UNADJUSTED OUTPUT FOR THE NORTH FREEWAY.

	Do Nothing Case 1	HOV Lane Case 2	Add 1 Lane Case 3	Add 2 Lanes Case 4	Take Lane for HOV Case 5	Add 10% Capacity Case 6
Total Travel Time, Veh-Hours	17273	11081	32625	32402	*	35757
Total Travel Time, Pas-Hours	20004	14324	37940	37680	*	41373
Total Travel Distance, Veh-Miles	436900	453039	498760	498760	*	456151
Total Travel Distance, Pas-Miles	505620	585178	580367	580367	*	528253
Average Speed, MPH	28	43.1	55.8	57.2	*	32.1
Gasoline Consumed, Gallons	26311	29787	44675	45284	*	35818
Hydrocarbon Emissions, Kg	1154	977	2254	2264	*	2217
Carbon Monoxide Emissions, Kg	7496	5256	17876	18561	*	18130
Nitrous Oxide Emissions, Kg	1009	1096	1755	1923	*	1469

* Output not provided

TABLE 8. ADJUSTED VALUES FOR THE NORTH FREEWAY.

	Do Nothing Case 1	HOV Lane Case 2	Add 1 Lane Case 3	Add 2 Lanes Case 4	Take Lane for HOV Case 5	Add 10% Capacity Case 6
Total Travel Time, Veh-Hours	19472	10794	32042	31823	*	38583
Total Travel Time, Pas-Hours	22551	13952	37262	37007	*	44643
Total Travel Distance, Veh-Miles	492530	441288	489851	489851	*	492200
Total Travel Distance, Pas-Miles	570000	570000	570000	570000	*	570000
Average Speed, MPH	28	43.1	55.8	57.2	*	32.1
Gasoline Consumed, Gallons	29661	29014	43877	44475	*	38649
Hydrocarbon Emissions, Kg	1301	952	2214	2224	*	2392
Carbon Monoxide Emissions, Kg	8450	5120	17557	18229	*	19563
Nitrous Oxide Emissions, Kg	1137	1068	1724	1889	*	1585

determined if the relative performance of one case was consistent for the three different situations.

The gasoline consumption values were in favor of Case 2 (Figure 5). The least amount of fuel is consumed using the exclusive HOV lane alternative. Since this is the condition which is presently in operation the most effective method of conserving fuel is presently in use.

The results indicate that the exclusive HOV lane alternative produced the lowest hydrocarbon (HC) and carbon monoxide (CO) emission levels for the Katy and North Freeway Corridors (Figures 6 and 7). The Gulf Freeway had lower (HC) and (CO) emission levels for Case 6, the 10 percent added capacity alternative. This result may have been representative of the less severe congestion which is found on the Gulf Freeway compared to the Katy and the North Freeways. The fact that this case deals with operational improvements may have been significant because these improvements would cause more of an impact in an area of less severe congestion. This alternative would have to be further explored by modeling additional, less congested freeways.

Nitrous oxide (NO) emissions indicate, much like fuel consumption, that the lowest amount of emissions occurs with the exclusive HOV lane alternative (Figure 8). The exclusive HOV lane would be considered an advantage as far as (NO) emissions are concerned because this alternative consistently provided results in favor of this freeway improvement technique.

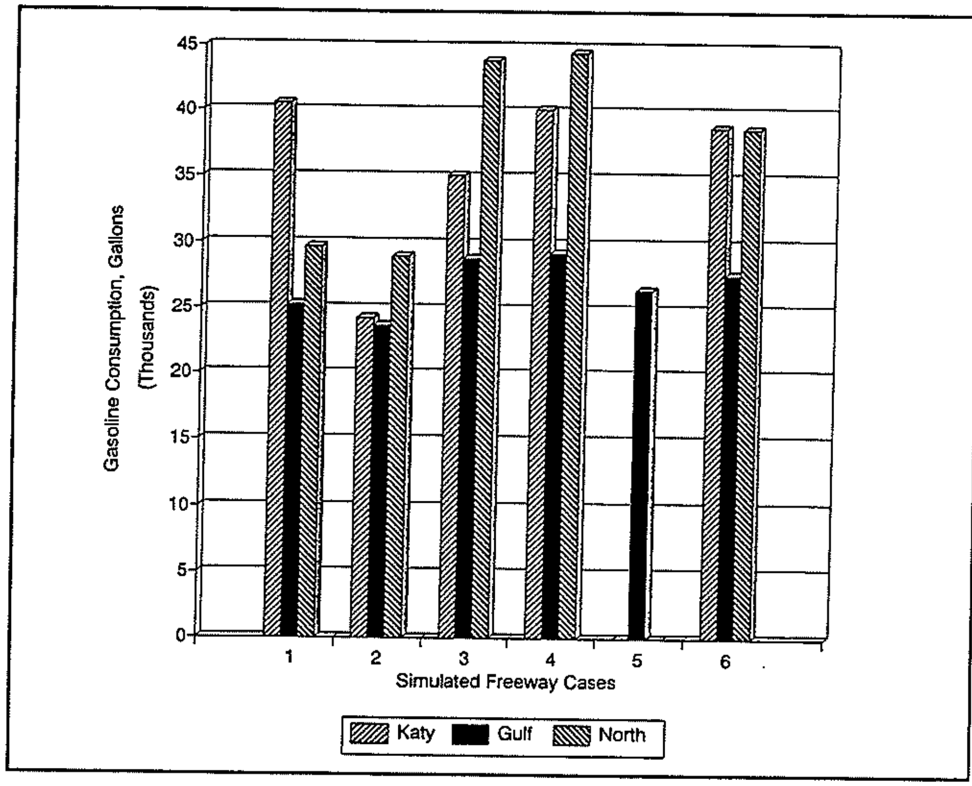

FIGURE 5. GASOLINE CONSUMPTION FOR THE KATY, GULF, AND NORTH FREEWAYS.

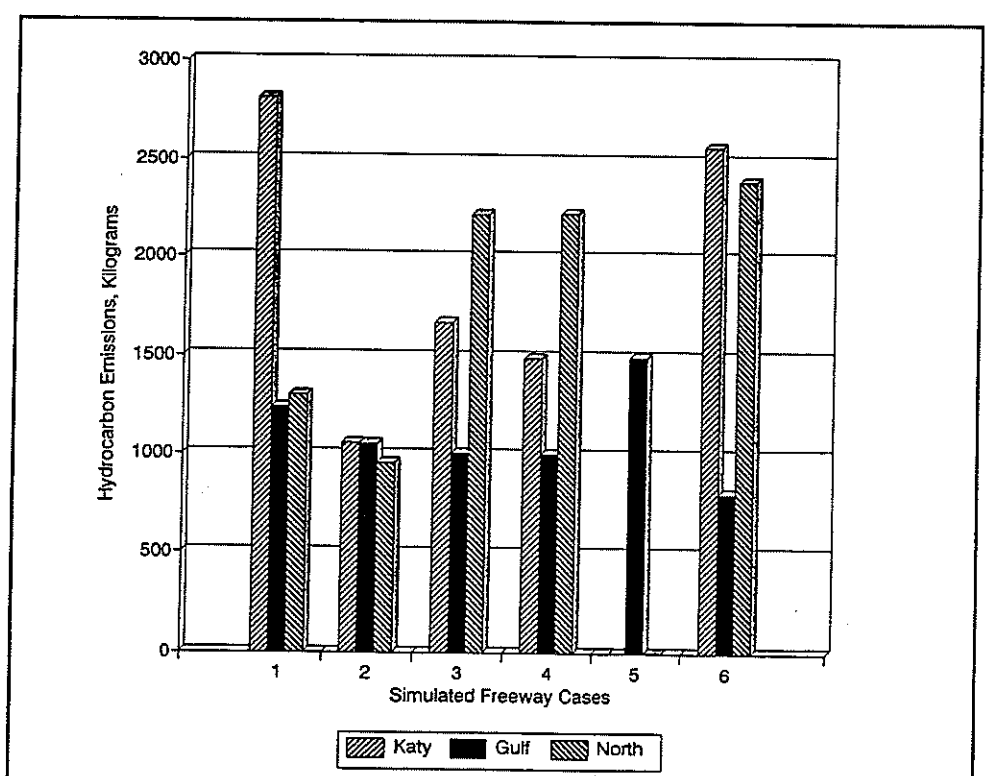

FIGURE 6. HYDROCARBON EMISSIONS FOR THE KATY, GULF, AND NORTH FREEWAYS.

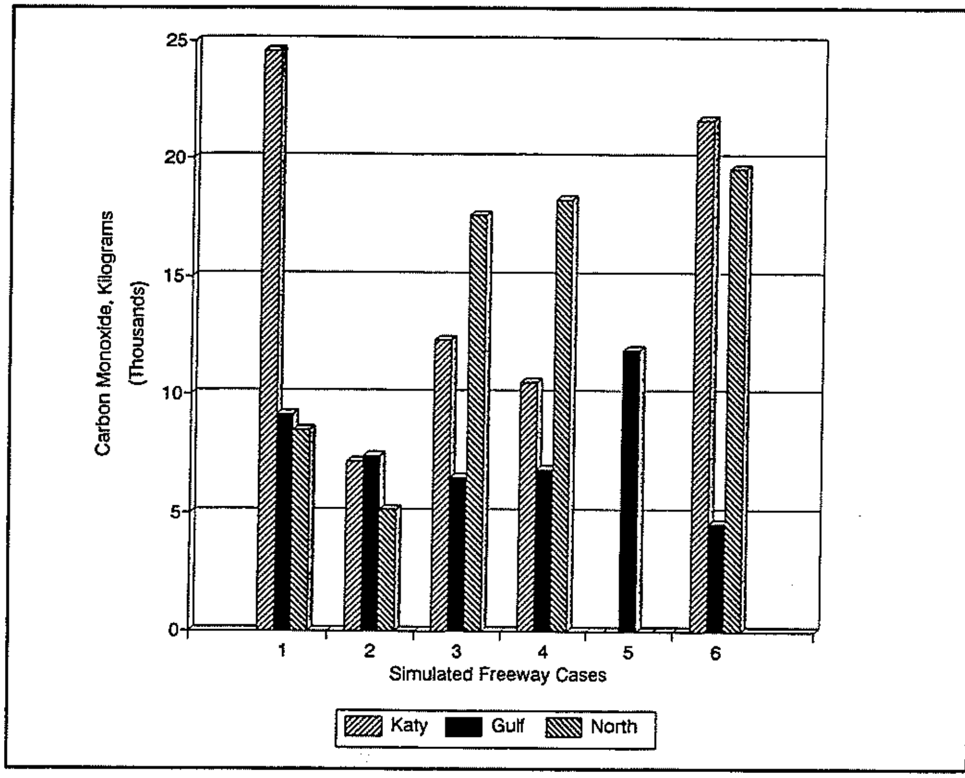

FIGURE 7. CARBON MONOXIDE EMISSIONS FOR THE KATY, GULF, AND NORTH FREEWAYS.

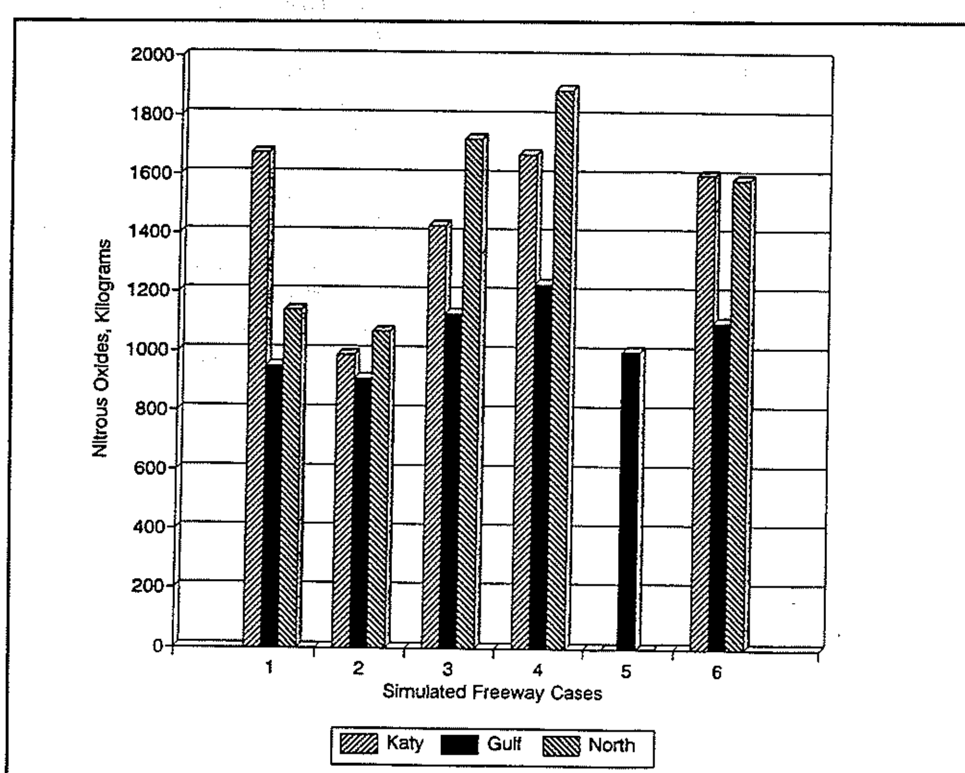

FIGURE 8. NITROUS OXIDES FOR THE KATY, GULF, AND NORTH FREEWAYS.

CONCLUSION

The results of this research indicate that for two of the three freeways used in this study, the Katy and the North Freeways, the best alternative of those analyzed is the exclusive HOV lane, based on fuel consumption and air quality benefits. The exclusive HOV lane alternative for the third freeway under consideration, the Gulf freeway, also resulted in having minimum values of gas consumption and nitrous oxide emissions for the exclusive HOV lane case. This information indicates that HOV lanes may be acceptable techniques for addressing environmental problems. It should be recognized that this analysis was performed with congested freeway corridors. There may be more advantages from operational improvements with less severe levels of congestion, modeled by an additional 10% capacity in this study. Both cases would have to be explored more completely in various freeway conditions to determine the effects these alternatives have on the environmental concerns.

RECOMMENDATIONS

As it has been mentioned in the research report there were certain areas which could have benefitted from more study. For instance, output for the take-a-lane case was obtained only for the Gulf Freeway; this did not make it possible to determine the effect of this alternative in a broader context. It would be more beneficial to determine if this case could be modeled for all three of the freeways in this study to provided accurate predictions of the take-a-lane scenario.

Another area which may be considered for further research is to model the Northwest and the Southwest Freeways, both of which are found in the Houston area. These freeways have also been modeled at TTI and therefore would not require extensive amounts of time and effort to set all of the cases up for these two freeways. This would give two more levels of confidence in the results which were already obtained and may provide insight into other traffic congestion levels. Modeling the Southwest Freeway would provide information on another freeway with lower congestion levels. Since the Gulf Freeway has similar congestion levels the results may be helpful in determining if the operational improvements are more effective in less severely congestion levels.

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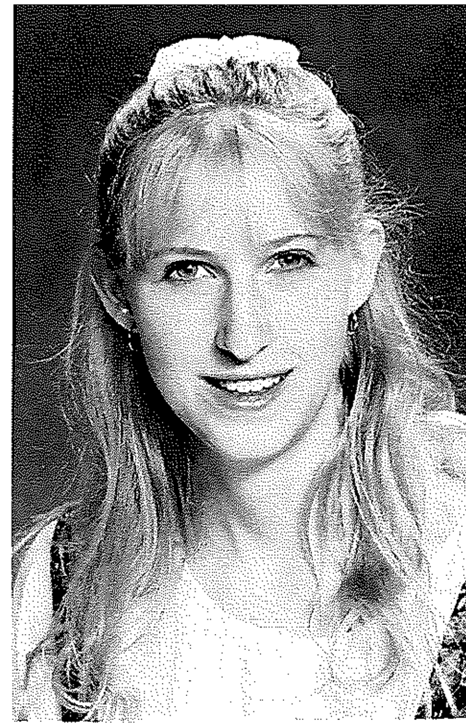
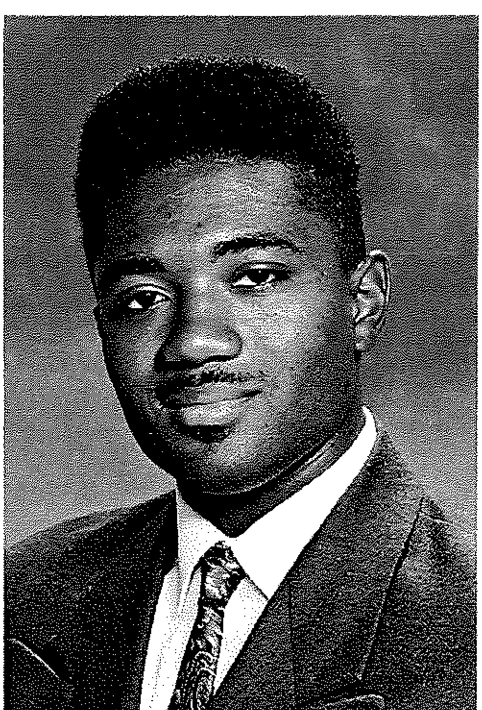
Biographical Data

Allison Christine Cherry was born in Moline, Illinois, on August 15, 1971. A few years later her family moved to Round Rock, Texas, where she grew up and attended Leander schools. She graduated from Leander High School in June 1989 and started at Texas A&M University that fall. She will receive an undergraduate degree in Civil Engineering from Texas A&M in December 1994.

In May 1991, Allison entered the cooperative education program at Texas A&M University. She worked one co-op term with the Texas Transportation Institute in College Station, Texas, and two co-op work terms with the Texas Transportation Institute in Arlington, Texas. Allison received an undergraduate transportation engineering fellowship at Texas A&M University for the summer of 1993. She is currently employed as a student technician at the Texas Transportation Institute in College Station.

Allison is a member of Chi Epsilon, the Civil Engineering honor society, and Tau Beta Pi, the Engineering honor society. She is also a member of the Texas A&M chapter of the Institute of Transportation Engineers. After graduation, Allison is planning to attend graduate school to study transportation engineering.





Juene Knoll Franklin was born in Houston, Texas, on February 6, 1971. He attended the High School for the Engineering Professions on the Booker T. Washington High School campus.

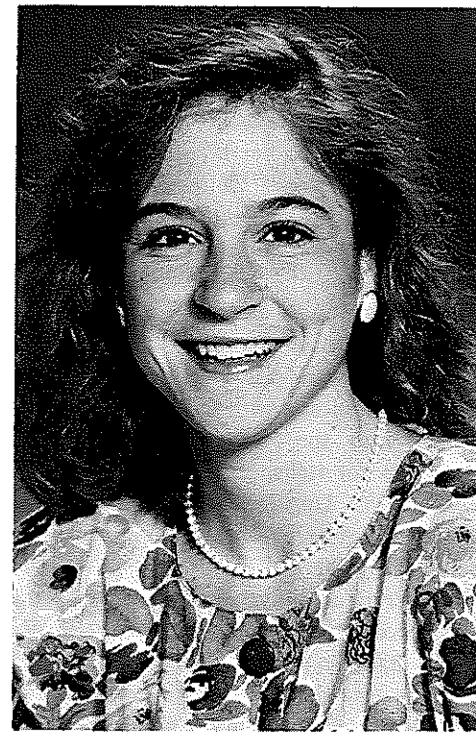
He is currently attending Texas A&M University where he is pursuing a degree in Civil Engineering. Juene will graduate in May 1994 with a Bachelor of Science in Civil Engineering. Juene was once a member of the National Society of Black Engineers (NSBE); he was a member of the INROAD's organization for three years.

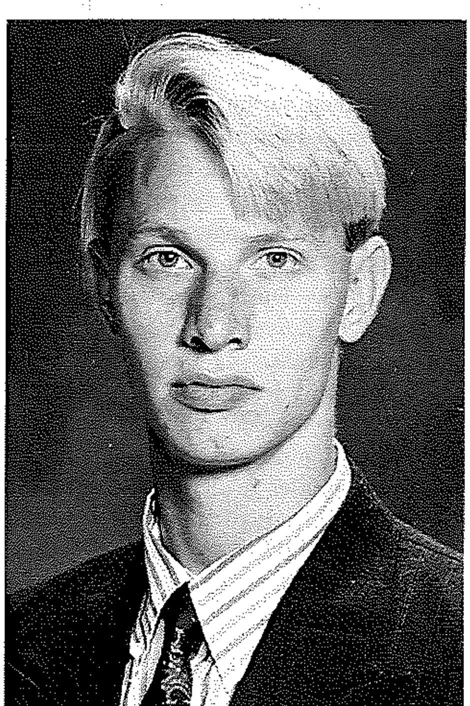
For the three years that Juene was a member of the INROAD's organization he interned with the Shell Oil/Pipeline Company before he accepted the TTI Fellowship this summer. At the culmination of the TTI fellowship, he will continue with TTI as a student worker during the upcoming school year.

Bridgette DeWees Keller was born May 31, 1972, in Columbia, South Carolina, and grew up in Cayce, South Carolina. She is a 1990 graduate of Brookland-Cayce High School. Bridgette entered Clemson University in August of 1990 as a freshman engineering major and will receive her bachelor of science degree in civil engineering in May of 1994.

Bridgette's work has chiefly involved highway materials and testing. For two summers, she was employed by the South Carolina Department of Highways and Public Transportation Research and Materials Laboratory where she performed tests on soils. In July of 1992, Bridgette was hired by the Clemson University Civil Engineering Department where she has done research and testing on rubberized and fly ash modified asphalt. Bridgette received a transportation fellowship for the summer of 1993 from Texas A&M University. During her fellowship she was employed at the Texas Transportation Institute as a research assistant.

While at Clemson, Bridgette became involved in the American Society of Civil Engineers and served as treasurer and vice president of Chi Epsilon, the civil engineering honor society. After graduation, Bridgette plans to pursue a master's degree in the transportation area of civil engineering.





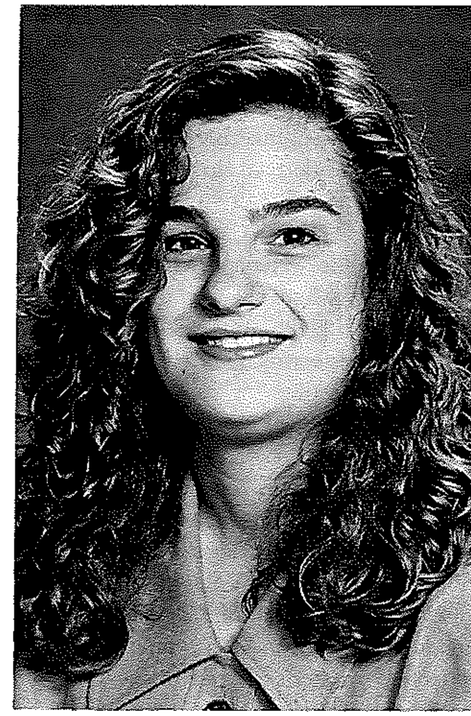
Joshua Martin was born in Bellefonte, Pennsylvia. He spent five years living in the northeast before settling down in Houston, Texas. Josh graduated from Memorial Senior High School in 1990 and is now a senior at Cornell University in Ithaca, New York. Josh is majoring in Civil Engineering. His hobbies including running and cycling. He is a member of the varsity cross-country and track teams at Cornell.

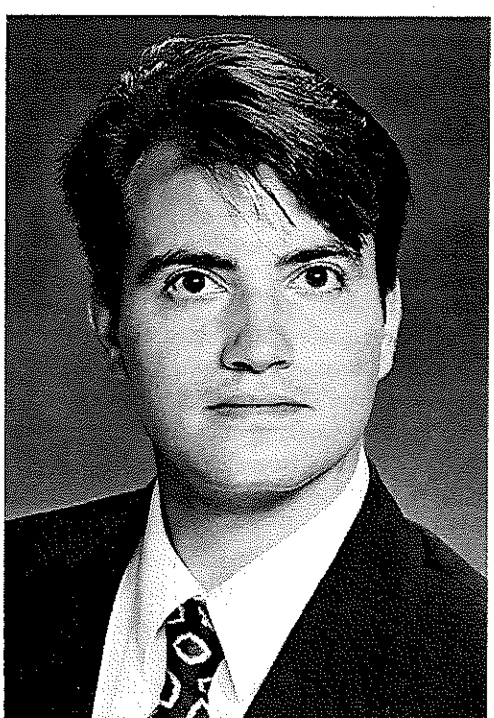
His freshman and sophomore summers were spent with Conoco and Conoco-DuPont in Houston with their offshore marine structures division. Josh had two projects each summer. The first summer consisted of providing summaries and abstracts of reports and analyzing ice forces on a downward cone model of an offshore platform. The second summer with Conoco-Dupont included working on a finite element model of an offshore platform and developing a program in Lotus 1-2-3 used to solve environmental forces on structures and produce a report ready results table. Also created with the program was a user's manual and a programmer's manual. His junior summer was spent with Texas A&M University in their Undergraduate Transportation Engineering Fellows Program working on a maintenance level-of-service project. Josh produced a report indicating the need for a uniform maintenance procedure and manual containing photographs and descriptions of each level-of-service for necessary items on Texas highways.

Jennifer Messick was born January 9, 1972 in Houston, Texas. She was raised in Holland, Pennsylvania. Jennifer graduated from the Pennington School, Pennington, New Jersey, in 1990. She entered the University of Delaware College of Engineering in September of 1990 and is currently pursuing a bachelor of science in civil engineering. Jennifer plans to graduate in May of 1994 and continue her education by going to graduate school.

Jennifer worked for Plant Operations at the University of Delaware for one year surveying the campus for the Americans with Disabilities Act. Jennifer was accepted into the Texas A&M Undergraduate Fellows Program for transportation engineering for the summer of 1993. During this fellowship she was employed as Research Assistant at the Texas Transportation Institute.

Jennifer is an active member of the Society of Women Engineers student chapter at the University of Delaware. She has also served as membership officer and is currently the President of Delaware's student chapter of the American Society of Civil Engineers.





Charles Joseph Naples III was born on October 30, 1968. He grew up in Schwenksville, Pennsylvania, and graduated from Perkiomen Valley High School in June 1986. He then began to pursue his undergraduate degree both full and part-time. He will receive his undergraduate degree in civil engineering from Temple University in May 1994.

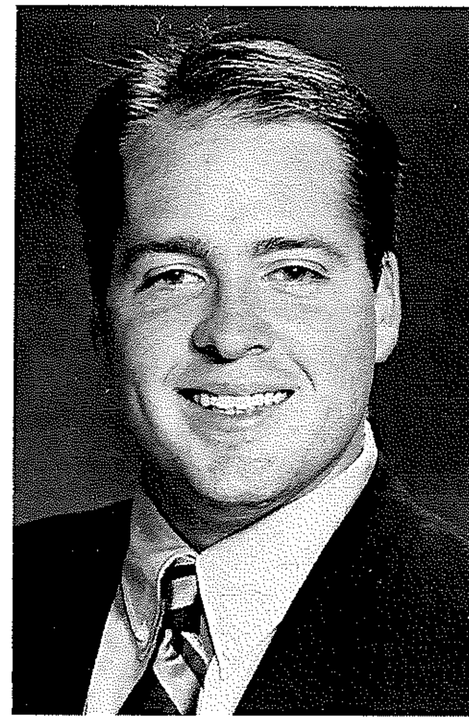
Charles received an undergraduate transportation engineering fellowship at Texas A&M University for the summer of 1993. During his fellowship, he was employed as a Research Assistant at the Texas Transportation Institute.

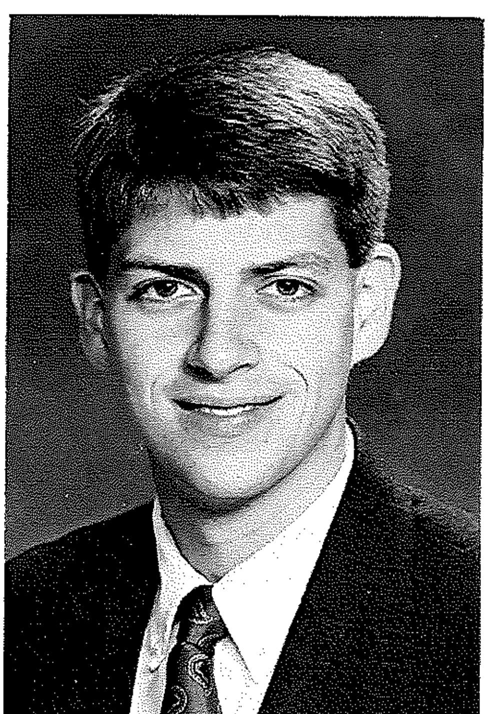
Charles will serve as the Vice-President of the Temple University Student Chapter of the American society of Civil Engineers for the 1993-94 term. After graduation, Mr. Naples is planning to pursue a master's degree in civil engineering but has not yet decided on the university he would like to attend.

Carl Brian Shamburger was born in Waco, Texas, on August 31, 1970. Brian grew up in Waco and attended primary and secondary schools in the Waco Independent School District. He graduated from Waco High School in May 1989. In August of 1989, he entered the college of engineering at Texas A&M University and will receive his undergraduate degree in Civil Engineering in May 1994.

In May of 1992, Brian began work for the Texas Department of Transportation (TxDOT) in the Advanced Planning division where he continued to work until January of 1993. Currently while enrolled at Texas A&M, he is employed by the Texas Transportation Institute (TTI). Brian received an undergraduate transportation engineering fellowship at Texas A&M University for the summer of 1993. During his fellowship, he was employed as a Research Assistant with TTI.

Mr. Shamburger is a member of the American Society of Civil Engineers (ASCE) and the Institute of Transportation Engineers (ITE). Within the local chapter of ASCE at Texas A&M, he is a representative to the Student Engineer's Council. After graduation, Brian is planning to pursue a masters degree in Transportation Engineering and would like to attend Texas A&M University in doing so.





Edward Daniel Sulak, Jr. was born in Houston, Texas, on August 28, 1970. He lived in Houston and Dallas for three years and then moved to Waco, Texas. He attended public schools in the Waco area and graduated from Reicher Catholic High School in 1989. After graduating from high school he was admitted to the College of Engineering at Texas A&M University in the fall of 1989.

In the spring semester of 1992, Edward began working for TTI as a student worker. He then worked for the Texas Department of Transportation at the district office in Waco. Edward was accepted into the Undergraduate Transportation Fellows Program at Texas A&M during the summer of 1993 and will continue to work for TTI upon completion of the Fellowship.

Edward is a member of the Texas A&M Student Chapter of the Institute of Transportation Engineers. He is currently serving as the chapter's Correspondence Secretary. He is also an active member of the American Society of Civil Engineers and the Texas Society of Professional Engineers.

Edward will receive his Bachelor's Degree in Civil Engineering in May 1994. After graduation, he has plans to enter a master's program but has not yet decided on the university he will attend.