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<td>Program Coordinator: Daniel B. Fambro</td>
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<td>The engineering research reports in this document resulted from the third year of the Undergraduate Transportation Engineering Fellows Program during the summer of 1992. 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&amp;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.</td>
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<td>This compendium contains reports on a wide variety of transportation research. Reports on transportation operations covered such current issues as analysis of the performance of vehicle detectors; incident detection using travel time information; effects of light rail transit being located in an arterial street system; and, an evaluation of the traffic operations at exit lane drops. Planning issues were addressed in reports on comparisons of observed and theoretical trip length frequency; the driver behavior impacts of freeway reconstruction; an investigation of the relationship between congestion and air quality; and, an analysis of the Houston Metro electronic information system. Research in the field of materials produced reports on lime stabilization of roadway subbase and microscopic analysis of fiber modified asphalt concrete. Driver and safety research is reflected in reports on mental workload in highway work zones, the effects of vehicle and road type in run-off-road crashes, and the determination of driver capability in the detection and recognition of objects.</td>
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Transportation Engineering Research Reports

1992 Undergraduate Transportation Engineering Fellows

SPONSORED BY THE
Advanced Institute in Transportation Systems Operations and Management

IN COOPERATION WITH
Texas Transportation Institute
Civil Engineering Department
Texas A&M University
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Foreword

The Southwest Region University Transportation Center (SWUTC) through the Advanced Institute Program, the Texas Transportation Institute (TTI) and the Civil Engineering Department at Texas A&M University recently established the Undergraduate Transportation Engineering Fellows Program. The program design allows undergraduate students in Civil Engineering to learn more about transportation engineering through participation in a transportation research program under the supervision of a faculty member or TTI researcher.

The intent of the program is to introduce transportation engineering to student that have demonstrated outstanding academic performance, thus, developing a critical resource: capable and qualified future transportation professionals.

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

**STUDENTS**

Kent M. Collins  
Texas A&M University

Brian P. Cronin  
Virginia Polytechnic Institute and State University

James L. DeSanto  
Ohio University

Glen A. Hanks  
University of Delaware

Amy R. Kohls  
Catholic University of America

Gregory D. Krueger  
Colorado State University

Marty T. Lance  
Clemson University

Mark Luszcz  
University of Delaware

Ronald L. Nowlin  
Texas A&M University

Dale L. Pica  
Texas A&M University

Janet Ricci  
The Cooper Union

Tricia A. Thomason  
California State University - Fresno

**MENTORS**

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Dr. Lindsay I. Griffin, III  
TTI Research Psychologist

Dr. Daniel B. Fambro  
Asst. Professor of Civil Engineering

Dr. Kay Fitzpatrick  
TTI Asst. Research Engineer

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Dr. Daniel B. Fambro  
Asst. Professor of Civil Engineering

Dr. Timothy J. Lomax  
TTI Assoc. Research Engineer

Ms. Katherine Turnbull  
TTI Systems Planning Program Manager
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.

[Signature]

Daniel B. Fambro, Ph.D., P.E.
Assistant Professor of Civil Engineering
Undergraduate Fellows Program Coordinator
US-75 North Central Expressway Reconstruction: User Response Analysis

KENT M. COLLINS

This report is a summary of the results for the May 1992 phase of user response analysis for the North Central Expressway reconstruction effort in Dallas, Texas. It deals with reported driving behavior and perceived changes in driving behavior on North Central Expressway during a three-year period starting May 1990, one month prior to the start of construction. Reported driving behavior and trends that have developed are compared with perceptions of changes that have taken place. The responses to the most recent (May 1992) survey are compared with the responses to previous surveys. The results indicate that the reconstruction effort has not impacted driving behavior significantly.

INTRODUCTION

North Central Corridor

The reconstruction of US-75 North Central Expressway in Dallas, Texas began in June 1990. The North Central Expressway is the primary link from the Dallas central business district to areas in North Dallas including Highland Park, University Park, Plano and Richardson. The effects of the reconstruction are not confined to North Central Expressway travel, however. An area that may be directly impacted by the construction has been delimited by the North Central Mobility Task Force. The border of the North Central Expressway Corridor is defined by I-635 LBJ Freeway on the north, Dallas central business district on the south, Dallas North Tollway on the west, and Audelia, White Rock Lake and Buckner on the east (1). Figure 1 illustrates the North Central Corridor.

History of Project

The Texas Transportation Institute is currently conducting a longitudinal study of traffic patterns and driver behavior within the North Central Expressway Corridor. The monitoring effort began during May 1990. At that time, a screen-line license plate survey and transit user survey were conducted to establish a panel of users that could participate throughout the project. At the same time, a set of pre-construction responses was obtained for comparison purposes.

The May 1992 survey is the fourth completed since May 1990 (2,3,4). The surveys are sent out at six-month intervals. The purpose of the questionnaires is to determine any change in reported driving behavior and perceptions of conditions due to the reconstruction. This is achieved through the comparison of the May 1992 survey responses to the responses of users from October 1991 (4) and from prior to reconstruction. Therefore, changes are based on both the entire construction project and the construction that has taken place since the previous survey.

OBJECTIVES

The purpose of this project was to analyze the results of the May 1992 surveys of automobile and transit user panels in the North Central Corridor and to compare the results to those of previous surveys.

BACKGROUND

A survey instrument similar to that utilized in the four previous surveys was used for this study. Two study panels are surveyed: one of automobile users and the second of transit users. The automobile users panel answers questions pertaining to their overall tripmaking behavior and more specific information concerning home-to-work and work-to-home trips. This particular questionnaire is used to examine departure times and travel times to and from work, any stops made en route to or from work, which paths of travel have been used, and vehicle occupancy.

The transit user panel is questioned about travel behavior with regard to the Dallas transit system. The
FIGURE 1. NORTH CENTRAL EXPRESSWAY CORRIDOR MODEL AREA
panelists are asked about the overall quality of transit service, changes in quality of transit service, travel times, and whether travel times have changed since construction began.

The aim of this study was to identify trends that have developed throughout the reconstruction project. This report also compares the trends to panelists' perceptions about the changes that have taken place.

STUDY METHODOLOGY

The first step in the overall monitoring effort was developing a panel of Expressway users that could be questioned throughout the monitoring effort. The panel was established in May 1990 by conducting a screen line license plate survey and transit survey and selecting a panel from the information obtained.

The next step, performed for each survey, was questionnaire circulation. A certain amount of attrition has taken place throughout the study and it was determined from the onset not to replace those that dropped out with representative panelists. The number of surveys returned for the May 1992 analysis dropped from 1825 panelists in May 1990 to 380 for the automobile user survey and from 597 original transit panel members to 109 for the most recent survey (1).

Next, a coding system for the database of responses had to be designed and developed. It was imperative that corresponding questions be coded similarly and logically to facilitate direct comparisons throughout the monitoring effort.

The last step was the actual analysis of the responses and interpretation of their meaning. The analysis was performed using PC-SAS analysis software. In performing the analysis, only those panelists that returned usable responses for each of the five surveys (May 1990 and each subsequent survey) were used. Due to the fact that some of the respondents had either changed residence, changed place of employment, or no longer used the transit system, the number of usable responses differs from the total number of responses returned. The numbers of usable responses for the May 1992 survey were 355 and 93 for the automobile and transit panels, respectively.

RESULTS

The results for the May 1992 survey are presented below according to three separate categories:

- Overall Automobile Panelists Tripmaking Behavior
- Automobile Panelists Work Related Tripmaking Behavior
- Transit Panelists Tripmaking Behavior

Overall Tripmaking Behavior

Figure 2 displays the average tripmaking frequency of panelists in May 1992. The total number of trips being made per day and the number of those trips being made on the Expressway are both displayed in the figure. The results illustrate a 4 percent drop in total tripmaking frequency relative to pre-construction values. This figure reflects a smaller drop than was reported for the October 1991 survey. The second part of the illustration depicts a 5.5 percent increase in trip rates for travel on North Central relative to pre-reconstruction values. Neither the increase in rates on North Central itself nor the drop in total rates are statistically significant using an \( \alpha = .05 \).

In Figure 3, user perceptions about their tripmaking frequency indicate that a majority of panelists believe that their tripmaking has not changed since reconstruction began. However, a growing minority believe that they are making more trips per day.

In comparison, Figure 4 shows the perceived change in tripmaking frequency, which illustrates that a majority of panelists believe their tripmaking frequency on North Central has not changed. Again, there is a gradual increase in the proportion of panelists believing that they are making more trips than before construction. Finally, it appears that the percentage of panelists who perceive a decrease in tripmaking on North Central may begin to decline after a high in October of 1991.

Work Related Tripmaking Behavior

Departure Times

The average departure times for home-to-work and work-to-home trips were compared in aggregate and were also segregated on whether or not the panelist used the Expressway to make the trip. For work-to-home trips, departure times have remained consistently at 5:00 pm. However, home-to-work departure times have shown considerable variance between surveys. Even so, no consistent pattern has emerged in terms of departure time changes.

Most panelists continue to perceive no change in departure time for work as compared to pre-construction conditions, as can be seen in Figure 5.
FIGURE 2. AVERAGE TRIPMAKING FREQUENCY OF AUTOMOBILE PANELISTS

FIGURE 3. PANELISTS' PERCEIVED CHANGE IN TOTAL TRIPMAKING FREQUENCY
FIGURE 4. PANELISTS' PERCEIVED CHANGE IN TRIPMAKING FREQUENCY ON NORTH CENTRAL EXPRESSWAY

FIGURE 5. PANELISTS' PERCEIVED CHANGE IN HOME-TO-WORK DEPARTURE TIMES
There has been no change in the percent of panelists indicating they were leaving earlier. However, there has been a slight increase in the percent that perceive they have adopted a later departure time relative to the October 1991 value.

**Travel Times**

Travel times for home-to-work and work-to-home trips are presented in Figures 6 and 7, respectively. Each figure illustrates the average travel time of all panelists for each survey, as well as averages stratified according to whether or not the panelists used North Central Expressway. Home-to-work travel times have decreased for all categories with respect to October 1991 values. This may indicate an overall decreasing trend in mean travel times for work-related trips within the corridor during the last six months. The most notable change in travel times has been with respect to other routes. For work-to-home trips, there has been an increase in travel times for the entire panel considered together and for those panelists that use other routes. However, there has been a slight decrease in travel times for those who use the Expressway for work-to-home trips. None of the changes in travel times with respect to October 1991 or with respect to May 1990 are statistically significant with an $\alpha = .05$.

Analysis of user perceptions of changes in travel times are shown in Figures 8 and 9. The majority of the panelists have noticed no change in their travel times. Interestingly, this group increased in number with respect to the October 1991 survey responses. A group of panelists that continues to be a large minority believes that its trips are taking longer. This group makes up nearly twenty-one percent for home-to-work trips and almost twenty-four percent for work-to-home trips. Though these are considerable percentages, they are down from October 1991 values for both home-to-work and work-to-home trips. Less than three percent of panelists believe that their travel times have decreased.

**Mode Choice**

The percentage of panelists (Expressway users versus non-users) that drive alone during home-to-work and work-to-home trips is illustrated in Figures 10 and 11. A majority of the panelists continue to drive alone for both home-to-work (95 percent for NCE users and 93.1 percent for non-users) and work-to-home trips (94.6 percent for NCE users and 95.5 percent for non-users). There are no obvious trends in these values from survey to survey.

**Route Utilization**

Finally, the percentage of the total panel that use the Expressway is illustrated in Figure 12. Since May of 1991 there has been a steady, gradual increase in the percentage that use North Central both for home-to-work and work-to-home trips. Percentages for May 1992 are above the pre-construction level for both trip categories. This result may indicate that as the monitoring effort continues and sample sizes decrease, more non-North Central users than users are dropping out. If this occurs, it would suggest a skewed sample and indicate higher percentages that use North Central for their trips.

**Transit Trip Behavior**

When the transit travel panel was questioned about overall quality of transit service in May 1992, the majority (87.2 percent) reported either a "good" or "excellent" rating as illustrated in Figure 13. There has been a reduction in those perceiving "fair" service from October 1991, and there were no panelists reporting a "poor" transit service rating.

Shown in Figure 14 is an examination of the panelists' perceptions about the change in quality of the transit service. There have been small increases (relative to October 1991) in those panelists that perceived both "Worse" and "Better" quality. This indicates that more and more panelists are noticing change, whether it be good or bad, in the transit service due presumably to the construction on North Central.

**SUMMARY**

The following is a report of the principal findings of the most recent analysis of the North Central Expressway user panel responses:

1. There has been a 4 percent drop in total tripmaking frequency and a 5.5 percent increase in trip rates on the Expressway with respect to May 1990 results. The percentage of panelists that believe their trip making in total and on North Central has increased is rising steadily while a majority still believe that their trip rates have not changed.

2. Whereas a majority of panelists still believe that they have not changed their departure time for work, there has been a slight increase over October 1991 values in those that believe they are leaving later. The average reported departure time from
FIGURE 6. AVERAGE HOME-TO-WORK TRAVEL TIMES

FIGURE 7. AVERAGE HOME-TO-WORK TRAVEL TIMES
FIGURE 8. PANELISTS’ PERCEIVED CHANGES IN HOME-TO-WORK TRAVEL TIMES

FIGURE 9. PANELISTS’ PERCEIVED CHANGE IN HOME-TO-WORK TRAVEL TIMES
FIGURE 10. PERCENTAGE OF PANEL THAT DRIVES ALONE FOR HOME-TO-WORK TRIPS

FIGURE 11. PERCENTAGE OF PANEL THAT DRIVE ALONE FOR WORK-TO-HOME TRIPS
FIGURE 12. PERCENTAGE OF PANELISTS THAT USE NORTH CENTRAL; HOME-TO-WORK AND WORK-TO-HOME TRIPS

FIGURE 13. OVERALL RATING OF TRANSIT SERVICE QUALITY
from work continues to be 5:00 pm. However, for home-to-work trips, those that use North Central leave earlier than those who use other routes.

3. Expressway users consistently report the highest home-to-work and work-to-home trip travel times. With respect to October 1991, there appears to be a general decrease in travel times for home-to-work trips while there is a general increase in travel times for work-to-home routes.

4. The majority of the panelists (75 to 80 percent) perceive no change in travel times to and from work. However, those that believe their trips are taking longer make up nearly twenty-one percent for home-to-work trips and nearly twenty-four percent for work-to-home trips.

5. A majority of panelists still drive alone in passenger cars for both home-to-work and work-to-home trips.

6. The percentage of the panelists that use the Expressway for work-to-home and home-to-work trips now exceeds the pre-construction level. There has been an upward trend in these values since a low in May 1991. These values may be biased due to the decrease in panel sizes.

7. The overwhelming majority of the panel (87.2 percent) report either "good" or "excellent" while there has been a reduction in panelists reporting "fair" transit service rating with respect to October 1991 results. No panelists reported "poor" service from the transit system. When questioned about the perceived changes in service quality, the result was increases in both "Worse" and "Better" responses, indicating that the perceptions of changes are random occurrences and not indicative of degraded transit service.

REFERENCES


Analysis of Detectors

BRIAN P. CRONIN

This report presents the results of a series of tests on the percent change in inductance caused by vehicle passage over deep buried and surface inductance loops. The long term effects of water on inductance loops is discussed. Finally, the ability of the Whelen and Sumitomo detectors to determine the speed of vehicles is discussed.

Deep buried and surface loops showed a definite difference between the percent change in inductance and the type of vehicle passing over a loop. Large cars tended to provide the largest percent change, while trucks had the smallest percent change in inductance.

The Whelen detector is not a reliable device to be used to detect vehicles and their speeds. Although the detector did seem to work occasionally at lower speeds, more testing on the detector is needed.

The Sumitomo detector worked better than the Whelen detector. It still did a poor job. It detected large cars a mere 50 percent of the time and small cars only 80 percent of the time. The detector unit did however give accurate speeds as long as the vehicles passed from under the front side of the unit.

INTRODUCTION

Purpose

Accurate vehicle detection is needed to keep traffic moving into the future. Already, vehicle detectors are being used to measure vehicle speeds, vehicular demand, vehicle direction, and to control traffic signal patterns. Induction loops, one type of detection device, play a major role in vehicle detection. Induction loops detect the presence or passage of vehicles. One aspect of induction loop detectors is the ability to detect different vehicle sizes. It is necessary to determine if different vehicle types cause different inductance changes while passing over a loop.

Surface induction loops are the most widely used type of induction loops in use today. Induction loops require a lot of maintenance, and as a result more deep buried loops are being constructed. In a recent study it was found that deep buried loops detect vehicles as accurately as surface loops (1). Therefore when looking at deep buried loops for measuring percent change in inductance upon vehicle passage, one also must look at surface induction loops to see if the same sort of trends hold.

Much effort has been placed on protecting and sealing inductance loops so they do not get damaged. In order to provide protection from water, inductance loops have been sealed very tightly. It is believed that sealing and protecting inductance loops from water may be unnecessary and is very costly. Therefore the long term effects of water on inductance must be tested.

Another way to detect vehicles is by using radar or ultrasonic technology. These types of detectors allow speed measurement along with conventional traffic counting. One big advantage of these kinds of detectors is that they can be placed off the road surface, making installation and maintenance much easier. Therefore, it is necessary to see if any of these kinds of detectors work accurately and consistently.

Objective

The objective of this research is to study a few aspects of vehicle detection, and the devices used to perform these tasks. The specific objectives are as follows:

1. Test deep buried and surface induction loops for inductance change variability upon vehicle passage.
2. Compare the percent change in inductance for surface and deep buried loops.
3. Evaluate the long term effects of water on inductance loops.
4. Test the Whelen detector for speed and detection capabilities.
5. Test the Sumitomo detector for speed and detection capabilities.
BACKGROUND

Inductance Loops

Inductance loops are the most widely used type of vehicle counter in use today. The flexibility in loop detector design allows for a broad range of vehicle detection (2). Inductance loops can be placed on the surface of a road, or up to twenty inches into the road surface. Many different shapes and sizes of inductance loops are used. A diamond, circle, or a square loop are some types. The most common loop in use today is the 6 foot by 6 foot rectangular loop.

Induction loops are controlled by detectors. The detectors energize the loop system through a lead in wire at a certain frequency. The loop then becomes an inductive element. As vehicles travel over the loop the inductance decreases until it passes a threshold level preset on the detector, at which time a detection is noted. The inductance of a loop can be increased by adding turns of wire to the system. A turn of wire is one wrap of wire around the loop, and each wrap can be combined to make multiple turns of wire. The threshold sensitivity also can be changed on the detector, making it easier or harder to pick up vehicles. When setting the sensitivity, caution is necessary, so that the sensitivity is not too high, causing vehicles in other lanes to be detected.

Long Term Effects of Water on Inductance Loops

Water has posed a threat to the reliability of inductance loops in the past. A great deal of effort and money has been spent in order to protect loops from the damaging effects of water. However, in a recent laboratory study, it was found that water around a loop produces only a small change in capacitance and frequency (1). According to this study's conclusions, this small change is not enough to cause poor operation of the detector.

The Federal Highway Administration (FHWA) questioned that study because it did not consider the effect that soil would have on the water saturated loop. The FHWA staff believes that water around a deep buried loop will change the inductance enough to trigger a false detection (3). Therefore, the study was moved to an active inductance loop on University Drive in College Station, Texas. After flooding the loop and conducting tests on the loop it was found that water still had no damaging effects on inductance loops (4). The FHWA still questioned the study; this time because the long term effect of water on the loop wire insulation was not studied.

Whelen Detector

The Whelen detector uses a radar beam to detect vehicles and their speeds. The detector works on the basis of the Doppler Effect; the detector records a frequency shift upon the passage of a vehicle as a detection. The Whelen detector has many advantages, such as being placed overhead, or at the side of a road. Also the radar beam can pass through plastic, therefore the detector can be protected from the weather by placing the beam in a plastic protective box. The detector is easy to install, and is basically maintenance free. Finally, the Whelen detector comes with the choice of a wide angle beam for multiple lane detection, or a narrow beam for single lane detection.

Sumitomo Detector

Unlike the Whelen detector, the Sumitomo detector uses an ultrasonic beam to detect vehicles. The Doppler Effect also is the basis of this detector. Unfortunately, an ultrasonic beam cannot pass through plastic. The Sumitomo detector therefore has two parts; a control box, and a cone for the beam which is exposed to the elements. The Sumitomo detector can be placed overhead at a height of up to 6 meters.

STUDY DESIGN

Deep Buried Inductance Loop

To test inductance variability upon vehicle passage a 15 inch deep buried loop located on University Drive and Avenue A in College Station, Texas was chosen. Two kinds of detectors were used to test the inductance loops, a Detector Systems TX Series Model # 813-10355 and a Sarasota TX Series Model # 515TX/MS detector. Two, three, and four turns of wire were used. Vehicles were classified into five different categories: small car, medium car, large car, van, and truck. As the vehicles passed over the loop, the base frequency of the loop, the occupied frequency of the loop, and the type of vehicle were recorded. Approximately 2200 vehicles were used in this study with about 100 vehicles in each vehicle classification test set. The frequencies were then converted into inductance's, and the percent change in inductance was calculated. From these data sets, it was possible to determine if different vehicles caused different percent changes in inductance.

Surface Inductance Loop

A surface inductance loop located at the Texas A&M Riverside Campus was used to test inductance variability
upon vehicle passage. Two different sized vehicles were used, a large car and a small car. The cars passed over the loop 12 times each at five different speeds. The frequency was measured and then converted into inductance so the percent change in inductance could be calculated. This test also allowed for the testing of the effects of speed and percent change in inductance for inductance loops.

**Long Term Effects of Water on Inductance Loops**

To test for the long term effects of water on inductance loops an inductance loop on University Drive was flooded in the summer of 1991. The loop was connected to a new electronic loop detector unit in the summer of 1992. The detector should now be able to detect vehicles as they pass over the loop, and allow recording of a reliable change in frequency which can be converted to change in system inductance.

**Whelen Detector**

The Whelen detector was tested at the Texas A&M Riverside Campus. The detector was attached to a 20 foot high catwalk. Three classes of cars: small, medium, and large, passed under the detector at varying speeds, lanes, and directions. The detector would pick up the vehicles and send a message to a computer which would display the speed of the vehicle. From this information it was possible to determine if the detector detected the vehicles, and how accurately it recorded the speed. A radar gun was used to verify the speed of the vehicle.

To test the Whelen detector for use at the side of the road, a 50' tall light post located at the Riverside Campus was used. The detector was attached to a mechanism that could be raised or lowered along the light post. The detector was then placed to aim 45 degrees to the roadway, and 45 degrees down toward the road surface. Cars drove past the detector at varying speeds, directions, and distances from the light post. It was possible to tell if the detector was working, and under what conditions it worked, by noting the speed displayed on the computer screen.

**Sumitomo Detector**

The Sumitomo Detector was also tested at the Riverside Campus. It was tested much like the Whelen detector, except that it did not have to be attached to a computer, because it displayed the speed of a passing vehicle on the unit itself. Only the overhead position was used. The detector had the cone with the ultrasonic beam placed at a height of 20' and pointing 45 degrees down towards the road. The accuracy and precision of the Sumitomo detector were observed.

**RESULTS**

**Deep Buried Inductance Loop**

After testing the deep buried inductance loop, it was clear that different vehicles produced different percent changes in inductance. The Detector Systems detector data showed that large cars produced a much higher percent change in inductance than did small cars, or even trucks (Figure 1). Upon testing the Sarasota detector the same trends appeared (Figure 2).

Figures 1 and 2 show the relationship between different vehicle types and the percent change in inductance, while using different turns of wire and different detectors. The percent change in inductance is the difference between the base inductance and the occupied inductance represented as a percentage. It is evident that a large car will produce a much higher percent change in inductance than a small car. A reason for this is that a large car provides more magnetism, thus causing a high change in inductance. A truck on the other hand causes only a small percent change in inductance, since trucks ride higher off the road surface than most cars.

**Surface Inductance Loop**

The surface inductance loop proved to have a much higher percent change in inductance than the deep buried inductance loop did. As shown in Figure 3, the surface loop had a percent change in inductance of 1.726%, while the deep buried loop only changed 0.201%. When the surface loop was tested at 50 mph the loop also showed variability in the percent change in inductance between large cars and small cars. The percent change was very slight, 1.17% for the large car, and 0.990% for the small car. This is not a very significant change, since only data for one speed and only 12 passes for the large car and 8 for the small car were collected. More testing is necessary for these results to be verified.

The surface inductance loop also showed another significant trend. The large car passed over the loop 12 times each at speeds of 60, 50, 40, 30, and 20 mph. After looking at the percent change in inductance for each speed it was evident that at faster speeds, the inductance changed only 1%, while at slower speeds, the inductance changed 3% (Figure 4).

This is an important finding, and could tell why some loops are not picking up some vehicles passing over the loop. If the loop is set in a place where vehicles are traveling fast, the inductance might not be changing enough to pass the threshold level set on the detector, and therefore a vehicle might not register a detection. This
**Figure 1.** Percentage change in inductance for different vehicle classes (Detector Systems Detector)

**Figure 2.** Percentage change in inductance for different vehicle classes (Sarasota Detector)
FIGURE 3. PERCENTAGE CHANGE IN INDUCTANCE FOR A DEEP BURIED VS. A SURFACE LOOP (DETECTOR SYSTEMS DETECTOR WITH 4 - TURNS)

FIGURE 4. PERCENTAGE CHANGE IN INDUCTANCE AS SPEED VARIES FOR A SURFACE LOOP
detector, and therefore a vehicle might not register a
detection. This also raises a question on the suitability of
inductance loops for use in detecting fast moving vehicles.
More research must be conducted on this topic.

Long Term Effect of Water on Inductance Loops

After water had been sitting in a 15 inch deep buried
loop for one year the long term effects were tested. The
inductance loop was hooked to a Detector Systems
detector and it was clear that the detector worked.
Vehicles were detected every time that they passed over
the loop, and the frequency of the loop was the same as
it had been when it was dry. Clearly water has no long
term effect on the accuracy of inductance loops.

Whelen Detector

The Whelen detector provided far less than positive
results. The detector was tested on five different
occasions and only worked once. The time that the
detector worked, parts from the wide beam and narrow
beam detector were interchanged to give a detector that
would work. The detector had a wide beam, and was
located in the overhead position. The Whelen detector
never worked from the side fire position.

The results from the one time that the Whelen
detector worked are shown in Table 1. The results show
that the detector did not work consistently at speeds
greater than 30 mph. When the detector did work at the
higher speeds, the speed recorded was much lower than
the actual speed.

Sumitomo Detector

The Sumitomo detector also showed rather
disheartening results. The detector worked about 50
percent of the time on large cars (Table 2). The speeds
that it gave were accurate for speeds of 30 mph and
slower, but for a speed of 50 mph the speed was about 4
mph low. This is most likely do to the fact that when the
vehicle drove underneath the detector from the back side
the displayed speed was about 8 to 10 mph slower than
the actual speed. The detector works much better when
vehicles travel from the front side (Table 2).

The small car was detected at high speeds and low
speeds on an average of about 80 percent of the time
(Table 3). The speed for the vehicles, especially at high
speeds, was also much closer to the actual speed once
the vehicles that were passing from the back of the
detector were deleted.

FINDINGS AND RECOMMENDATIONS

Inductance Loops

Inductance loops are the least expensive vehicle
detection devices in use today. They work accurately
most of the time, but they also need to be researched
more to see if some of the variations that occur when
vehicles pass over the loop can be controlled and used to
help in the placement and construction of inductance
loops. For example, it is clear that the percent change
in inductance for inductance loops is a function of the
depth of the loop, the speed of a vehicle, and the size of
a vehicle. It is recommended that more studies be
conducted on the relation to speed and the percent change
in inductance. To find that a fast moving truck might not
be detected by an inductance loop, because the threshold
level was too high, is a very disturbing idea, and it might
create a rather dangerous situation.

Since inductance loops are not affected by water as
much as was once believed little effort will be needed to
insure 100 percent sealing of inductance loops. While
inductance loops must be sealed to protect against the
breaking of the wires, a tight seal against water is not a
necessity.

Whelen Detector

The Whelen detector could be a very helpful device
to use in detecting vehicles. The problem is that it does
not work accurately and precisely 100 percent of the time.
The detector needs to be tested a lot more and maybe
even redesigned so that it works consistently. One such
addition to the Whelen detector that should be made is
that some kind of display on the detector unit itself should
be added so that one knows if the computer is at fault or
if the detector itself is not working.

Sumitomo Detector

The Sumitomo detector is another device that has
great promise. It worked better than the Whelen detector
did, but it still does not work all the time and has
problems detecting some vehicles. Much more research
is needed to be done on this unit, English symbols on the
device would make use in the United States much easier.
### TABLE 1. WHELEN DETECTOR

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### TABLE 2. SUMITOMO DETECTOR

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### TABLE 3. SUMITOMO DETECTOR

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### REFERENCES


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4. Robert A. Hamm. *Analysis of Inductance Loop Detector Sensitivities*, Texas Transportation Institute, Texas A&M University, College Station, Texas, Summer 1991.
Detection of Incidents Using Real-Time Travel Time Data

JAMES L. DESANTO

It is proposed that an algorithm comparing real-time travel time data and historical travel time data from a given segment of urban freeway could automatically detect incidents. The Texas Transportation Institute is currently collecting real-time travel time data from three expressways in the northern corridor of Houston, Texas, through the implementation of cellular telephone technology. To evaluate the feasibility of such an algorithm, a single section of freeway was selected and a statistical analysis was performed on the travel times and incident reports from the selected segment. Data influenced by incidents were removed and mean travel time profiles and 95 percent confidence intervals were generated based on time-of-day and day-of-week divisions. Time-of-day classifications demonstrated travel time differences throughout the peak-period. Divisions by day-of-week illustrated significant differences in the intensity of traffic and peak period travel times throughout the week. It was also determined that statistically significant differences exist between the average travel times of incident-influenced data and data which had all incident-affected travel times removed. However, it was found that the project’s current means of collecting and assimilating incident data is insufficient to create a satisfactorily accurate set of data for the task of detecting incidents in real time using only segment-to-segment travel times.

PROBLEM STATEMENT

Whereas recurrent congestion can be generally attributed to geometric bottlenecks, freeway incidents are the major cause of non-recurrent congestion (1). When an incident occurs, it must be cleared from the traffic stream as quickly as possible to allow the resumption of normal flow. Also, freeway traffic demands should be lowered, if possible, by suggesting alternate routes for drivers to divert to before they reach the congested area (2).

Rapid detection and verification of incident location and the type of response needed is essential to relieving the incident’s impact upon traffic flows. Many studies have been performed to determine an accurate way to detect freeway incidents. Traditionally, incident detection has relied on loop detectors imbedded in the pavement from which spot measurements of volume, speeds, and loop occupancy can be obtained (3). This method is not foolproof, however, and research continues for improved methods of incident detection.

A new technique that may prove useful for incident detection is the utilization of real-time travel time data collected using vehicles in the traffic stream as moving probes. Tracking probe vehicles along a route allows travel times to be directly computed. Theoretically, as segment travel times increase beyond normal levels, the likelihood of an incident having occurred in that segment or in a segment downstream increases. Thus, the comparison of real-time travel time data to historical data could be used to estimate whether an incident has occurred. This information could then be automatically relayed to local traffic management authorities. In addition, the real-time travel data could be displayed to motorists to help them make better diversion decisions.

Of course, a comparison of real-time to historical data is somewhat more complicated than simply computing a ratio or a difference in values. Travel times are stochastic by nature. Variations can be attributed to any number of roadway and traffic factors and also are individual to each driver. In order to compare real-time data to historical data, confidence intervals must be established for a set of historical travel times that are unaffected by incidents. Real-time data that exceeds the established range could then be used to indicate an incident condition.

Unfortunately, very few databases are available from which an estimate of the normal variation in travel times can be obtained. Research is needed to investigate the magnitude of the normal variations in travel time data, and to investigate whether real-time travel time data might serve as a useful incident detection tool.
OBJECTIVE STATEMENT

The goal of this research was to determine the normal day-to-day and time-of-day variations in real-time travel time data for a designated segment of urban interstate highway, and to establish confidence intervals that would enable real-time travel time data to be used to detect incidents. The first objective of this research was to determine the mean travel times by time-of-day and day-of-week for a single section of roadway within the study corridor. The second objective was to determine the variation in travel times by time-of-day and day-of-week so as to compute the 95 percent confidence limit of the mean travel time data. The third objective was to determine whether presumed incident-affected travel times (as defined by incident reports made by probes) differ significantly from the mean travel times as computed in the first objective.

BACKGROUND

Currently, the Texas Transportation Institute is conducting an experiment collecting real-time travel time information in the city of Houston. Three highways make up the north Houston Corridor - Interstate 45, U.S. 59, and the Hardy Toll road (Figure 1). Each highway is divided into three- to five-mile sections. Travel time data are collected through the use of cellular telephone owners calling in to a central data processing center at posted section divisions. The location of the user (probe) and the time the call is placed are recorded into a database by an operator. The locations and brief descriptions of incidents reported by probes are recorded in a separate file. Travel time information can then be directly derived from these databases. This system has generated an extensive database of travel time and incident data from which "normal" travel times and confidence intervals on these roadways can be directly computed and evaluated.

STUDY METHOD

The first stage of research was to identify an appropriate roadway segment to study. A situation where free-flow conditions normally exist and where a capacity reducing event could quickly induce congestion was desired for this study. Upon consultation with TTI-Houston personnel, the north- and south-bound lanes of Interstate 45 between Beltway 8 and Shepherd Road were selected.

Next, travel time data and incident reports were acquired from the travel-time-data-collection database system in Houston. Data were collected from the system between December of 1991 through the first week of June 1992. The travel time data set consisted of the following information:

- Facility from which the travel time information originated,
- Date and time of each motorist probe call to the system,
- Quarter-hour of the call,
- Starting and ending location identification numbers of the roadway segment just traversed,
- Distance traveled,
- Travel time measured, and
- Identification number of the probe making the call.

An incident data set was created from probes' reports of roadside or road-blocking accidents and stalls. The incident data set contained the following information:

- Date and time of the report,
- Location where the report of the incident was made,
- A short description of the type of incident and corresponding traffic conditions, and
- Identification number of the probe making the call.

Incident reports from the study section or the sections immediately upstream or downstream of the study section were then matched to the dates and times of travel time reports. Incidents occurring on sections of interstate immediately upstream or downstream were pulled from the travel time database due to potential upstream metering effects and downstream queuing effects which could have altered travel times through the study section. All travel time observations reported within 30 minutes prior to, or 60 minutes after, a reported incident were then deleted from the study set. This reduced the study set sample size by approximately 25 percent.

From this reduced data set, statistical calculations were then made. All data from each particular 15-minute period on a particular weekday were grouped together in subsets. The sample sizes, means, and standard deviations for each subset were compiled. From these calculations, a 95 percent confidence interval was generated for each weekday quarter hour. The mean travel time for each 15-minute period was plotted for each day of the week in histogram form. The upper limit of the 95 percent confidence interval was then plotted above the mean. Figure 2 shows an example of a travel time profile and its accompanying confidence limit.

RESULTS

Comparisons were first made between the original data set and that set from which all reported-incident related data had been removed. Second, differences between the overall filtered data set and subsets based on
FIGURE 1. LOCATION OF STUDY, CITY OF HOUSTON, TEXAS
day of the week were examined. Finally, travel times associated with reported incidents were matched with their respective established travel time profiles to examine any relationships that may exist.

Separation of Incident Data

From the onset of this research, the filtration of incident-related data from the original data set has been a high priority. By removing the travel time data for that segment occurring around the time of a reported incident, unusually high or low travel times would be eliminated from the database, and result in mean travel times that were closer to representing "normal" travel times. In addition, the variability in travel times should be lowered as well. An examination of Figures 3a through 4b supports these conclusions. Figures 3a and 4a allow for a comparison between the mean travel times for the unfiltered original sample of travel times and those of the filtered sample for each respective quarter-hour interval. The upper limit of the 95 percent confidence interval of the filtered travel times is plotted on the graphs as a dashed line. Figures 3b and 4b show the standard deviations by quarter-hours of the same data sets. Note the significantly higher standard deviations of the original data set than that of the filtered set.

To quantify the overall differences between the original and filtered sets, average differences in quarter hour means and standard deviations were computed (Table 1). The differences between the original and filtered data on both the graphs of the means and standard deviations illustrate the effect of eliminating incident data. The filtered data set resulted in somewhat lower means and smaller, more stable standard deviations. Note that standard deviations were reduced by over one-half of a minute when incident data were eliminated.

Classification

Data were classified further by day of the week. Figures 5 and 6 plot the mean travel times for each day of the week and for morning and afternoon peak periods, respectively. A histogram was developed (Figures 7 and 8) showing the means and upper 95 percent confidence limits of the filtered peak period data sets. Comparing means from each day (Figures 5 and 6) to the confidence limit from Figures 7 and 8, it can be seen that statistically significant differences exist between day-to-day data and the total filtered set. It is important to note that while each day's means follow the same general pattern, enough significant differences in day-to-day patterns exist to warrant the use of day-specific profiles for further analysis.

Incident Analysis

Finally, to test the particular usefulness of the established travel time profiles, incident-related travel times, as defined previously, were matched against their respective mean travel time profiles and plotted on Figures 9 through 12. Figures 9 and 10 examined all incidents, accidents and stalls, reported to have occurred in the study section or the section immediately downstream of the study section. Figures 11 and 12 focused only on those incidents that were reported to be accidents. The travel times examined were limited to only those of the probes reporting incidents. This was done to eliminate any question as to whether the probes actually passed through sections during times when those sections were affected by incidents.

Next, the selected incident travel times were then compared to the established travel time profiles from the respective quarter hours and days of the incident reports. The established mean was subtracted from the probe's travel time through the section and the resulting difference was divided by the established standard deviation. The distances of the travel times from the profile means in units of standard deviates were calculated and plotted. It was hoped that a definitive line would become evident above which most incident-related travel times would reside. It was also hoped that this line would be a certain constant number of standard deviates from the profile mean travel time.

Unfortunately, this was not found to be the case, as illustrated by Figures 9 and 10. Plots of all incidents reported, accidents and stalls, lane blocking and non-blocking, showed that the data tended to cluster near the mean travel time. Plots of accidents alone (Figures 11 and 12) revealed the same results. Due to the method used in compiling incident reporting data and limitations due to time constraints, a detailed study of travel times directly affected by lane-blocking incidents versus travel times affected by non-lane blocking incidents could not be performed.

Summary of Results

This research has determined that mean travel time data for a given section of freeway does differ significantly by time of day and by day of the week. Upon computing the variations of these mean travel times, it was determined that eliminating travel time data influenced by incidents from the database improves the reliability of the database and increases its sensitivity to abnormalities. However, it does not appear that the
FIGURE 2. TRAVEL TIME PROFILE FOR MONDAY AFTERNOON PEAK PERIOD

FIGURE 3A. AM TRAVEL TIME MEANS - ORIGINAL DATABASE VERSUS NON-INFLUENCED SAMPLE
FIGURE 3B. AM TRAVEL TIME STANDARD DEVIATIONS - ORIGINAL DATABASE VERSUS NON-INFLUENCED SAMPLE

FIGURE 4A. PM TRAVEL TIME MEANS - ORIGINAL DATABASE VERSUS NON-INFLUENCED SAMPLE
FIGURE 4B. PM TRAVEL TIME STANDARD DEVIATIONS - ORIGINAL DATABASE VERSUS NON-INFLUENCED SAMPLE

TABLE 1. AVERAGE DIFFERENCES IN MEANS AND STANDARD DEVIATIONS

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<td>Standard Deviations</td>
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<td>Average Percent Difference</td>
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FIGURE 5. AM TRAVEL TIME MEANS - COMBINED SAMPLE AND DAILY SETS

FIGURE 6. PM TRAVEL TIME MEANS - COMBINED SAMPLE AND DAILY SETS
FIGURE 7. AM TRAVEL TIMES - ALL NON-INFLUENCED DATA

FIGURE 8. PM TRAVEL TIMES - ALL NON-INFLUENCED DATA
FIGURE 9. STANDARDIZED INCIDENT-RELATED TRAVEL TIME DISTRIBUTION. REPORTED ACCIDENTS AND STALLS. AM PEAK PERIOD.

FIGURE 10. STANDARDIZED INCIDENT-RELATED TRAVEL TIME DISTRIBUTION. REPORTED ACCIDENTS AND STALLS. PM PEAK PERIOD.
FIGURE 11. STANDARDIZED INCIDENT-RELATED TRAVEL TIME DISTRIBUTION. REPORTED ACCIDENTS ONLY. AM PEAK PERIOD.

FIGURE 12. STANDARDIZED INCIDENT-RELATED TRAVEL TIME DISTRIBUTION. REPORTED ACCIDENTS ONLY. PM PEAK PERIOD.
current database can be utilized for automated incident detection.

While the use of this particular database in its current form to detect incidents in real time has not been supported by this study, it is not to say that real-time incident detection can not be done. The information contained in this database, combined with other traffic monitoring devices such as loop detectors and closed circuit television, could conceivably be a very effective tool in the field of incident detection.

RECOMMENDATIONS

Several factors limited the scope and effectiveness of this research, thereby influencing its conclusions. The following are some suggestions for further use of the study's database and for any similar research conducted in the area of incident detection with travel time information.

Future research of the database should utilize a longer period of data collection. This should result in lower standard deviations and therefore add to the accuracy of the travel time profiles. This, in turn, should increase the database's sensitivity to abnormalities. Also, further classification of the data by season may illuminate additional trends, however, this procedure would require many additional months of data collection.

Increasing the number of probes on a given freeway could also have a positive effect on the database. An examination of the database has revealed that a fraction of the probes that pass an incident will report it. This suggests that some incidents may go completely unreported during periods containing large probe headways. Increasing the number of probes in the system should decrease probe headways and increase the probability that incidents will be reported.

Finally, an evaluation of the manner that incidents are recorded should be considered. Further examination of the database's records of lane-blocking and non-lane-blocking incidents and their respective effects on travel time would be beneficial. However, important incident characteristics are inconveniently coded in the current system, making critical information difficult, if not impossible, to obtain. Individual codes classifying specific information about incidents need to be developed and implemented to increase the effectiveness and expedite future research.

REFERENCES


ACKNOWLEDGEMENTS

The author would like to express his appreciation to Mr. Gerald Ullman of the Texas Transportation Institute for providing invaluable direction, technical support, vision, and guidance throughout this project. Thanks also to Dr. Conrad Dudek and Mr. Kevin Balke for their technical support and suggestions in the development of this study. The author would also like to thank all the other undergraduate fellow at the Texas Transportation Institute for their support and advice.
Mental Workload In Highway Work Zones

GLEN A. HANKS

Accident rates in highway work zones are significantly higher than the statewide rates. Two reported accident rates in work zones were 74 and 257 percent higher than the state-wide average. From 1982 to 1988 fatalities in highway work zones increased 55 percent, while, in the same time total highway fatalities increased only 7 percent. The Federal Highway Administration recognizes that highway work zones impose unexpected demands on the driver. This study was designed to investigate if an increase in the mental workload is a possible cause of the increased accidents in the work zones.

This study determined the driver’s workload in a “normal” tangent and two work zones sections. The mental workload was obtained by measuring the visual information requirement of the driver. The mental workload (percent) was modeled as the ratio of vision access time to the total time.

This study found a significant increase in the driver’s mental workload in a work zone when compared to a “normal” section. There was also a significant difference between genders. Females had significantly higher workloads than males. Differences, although not significant, were found in the widths of the work zone and in the direction of travel.

INTRODUCTION

Background

Recently an increasing number of articles promoting increased highway work zone safety have been appearing in engineering trade journals. Editorials have appeared in Civil Engineering (1), ITE Journal (2), Engineering News Record (3,4), and Traffic Safety (5,6). Articles analyzing the effectiveness of channelization devices, barricades and barriers, and collection of work zone accident data have been published in Transportation Quarterly (7) and in the Transportation Research Record (8). All of these articles stress the same theme, highway work zones are dangerous.

Few statistics are kept specifically on work zones. Very little is known about the accidents and deaths that occur in work zones. The Department of Transportation Fatal Accident Reporting System (FARS) publication reports that from 1982 the number of fatalities have increased 7.15 percent from 43,945 deaths in 1982 to 47,087 in 1988 (9). In that same time period fatalities in highway work zones have risen 55 percent from 489 in 1982 to 756 deaths in 1988 (5).

Few states keep accident rates in work zones. The states that do publish accident rates in work zones derive them from studies of several work zones in the state and compare them to the statewide accident rates. Two states that reported high work zone accident rates, when compared to the statewide rates, were Virginia and Iowa.

Virginia experienced a 74 percent increase in work zone accidents over the statewide accident rate (7). Iowa’s difference between work zone and statewide accident rates was larger. The accident rate in Iowa’s work zones was 95 accidents per million vehicle miles, while the overall rate was 37 per million vehicle miles, a 257 percent increase in accidents (8).

The few statistics regarding work zone accidents and fatalities are due to the difficulty in collecting data. The accident rates from Virginia and Iowa are the result of studies conducted on several work zones. It would be very difficult to collect traffic data on each work zone. It is not feasible to perform a traffic count at each work zone, patch job, trash removal, mowing or pavement restriping.

A number of questions arise when considering that highway work zones have higher accident rates than the highway accident rate. What is causing this? Why are the number of fatalities in work zones increasing more than the overall fatalities? Why are the accident rates in work zones higher than the overall rates? Is it because there are more work zones? Are the drivers not heeding the warning signs or slowing down? Are the combinations of heavy traffic, changed traffic patterns and reduced lane...
widths causing the accidents, or are more people dying because there are more highway work zones?

**Problem Statement**

The Federal Highway Administration recognizes that typically a work zone disrupts the normal flow of traffic. Work zones change the highway environment. The traffic pattern may change. A merge or lane shift might be required (10). The work zone may cause drivers to perform unexpected maneuvers (10). What is it that causes these unexpected maneuvers? Are these maneuvers caused by failure on the driver's part to recognize a change in alignment? Or is it due to an excessive mental workload requirement or because the workload changes too quickly?

**Hypothesis**

The unexpected maneuvers that occur in highway work zones are due to an increased mental workload imposed on the driver by the conditions of the work zones. Mental workload may be defined as the mental capacity required to perform a task or tasks. In the driver's case mental workload or driver workload refers to the mental capacity expended to operate the vehicle. The driver must, in the execution of the task of driving perform the following sub-tasks:

- guide the vehicle
- control the vehicle to avoid collisions
- process information for navigation (Brackett and Roush, unpublished data).

To successfully navigate upcoming roadway features the driver must anticipate the mental capacity required by the feature. Should the driver underestimate the required mental capacity the potential for driver error is created. It is this driver error that can result in crashes (Brackett and Krammes, unpublished data).

In a normal highway environment driver error doesn't always result in an accident. Sometimes there is enough room for the driver to correct his error and thus avoid an accident. In a highway work zone there is little room for driver error and even less for a driver to correct his error. Work zones should be analyzed to determine what mental workloads they impose on the driver. By determining what requirements are placed on the driver work zones may be better designed, reducing the possibility of driver error.

**Objectives**

This project was designed to measure the driver mental workload imposed by a simulated work zone. From the information obtained, recommendations for changes and or further study will be made. The mental workload required to negotiate the feature was determined by measuring the amount of visual information requested by the operator of the test vehicle. This study followed Brackett and Krammes' test by modeling the percent mental workload as equal to the percent of time that the subject looks at the roadway.

\[
\text{Mental Workload (\%)} = \frac{\text{Access time}}{\text{Total time}}
\]

**METHODS**

**Apparatus and Materials**

The equipment used in this pilot study will consist of:

- Test car, 1989 Ford Taurus station-wagon equipped with cruise control, power steering, automatic transmission, and "buddy brake"
- Occluded vision apparatus
- IBM compatible computer
- Foot pressure switch
- Safety shaped concrete median barriers
- Plastic work zone barrels
- Raised pavement markers.

A computer and an occluded vision apparatus will be used to accurately measure the amount of time that a subject is able to see. The occluded vision apparatus consists of a plastic face shield that is clear or opaque depending on the presence of an electrical current coupled with an IBM compatible computer. The subject will be wearing the occluded vision face shield in such a manner that all vision, including peripheral, is shielded. The face shield is designed in such a manner such that there is no distortion, nor loss of vision when in its clear state, but totally blocks all vision when opaque.

The test subject requests vision by means of a floor mounted switch that is operated by the left foot. The foot switch prompts the computer for a "look" through the visor. The computer controls the duration of the "look", which is preset by the test administrator. In this test the
length of a look will be .5 seconds. This length of time allows for one complete sweep of the roadway. The system is set up to allow for the participant to request as many looks as feels necessary. For each vision request the foot switch must be pressed and released completely.

An IBM compatible computer will record the duration and number of looks for each work zone section. It is the assumption that the percent time of vision is equal to the percent of mental workload. From this calculated information a mental workload profile will be created for each section of the simulated work zone.

The test track was designed to simulate a highway work zone where work is being performed on the shoulder. The work zone was delineated by portable concrete barriers and plastic work barrels, while the travel lanes were delineated by raised pavement markers. The simulated work zones had travel lanes that were 10.5 and 12 feet wide (see Figure 1).

Participants

This study used six participants. All participants were employees of the Texas Transportation Institute and work at the Riverside Campus. Three of the participants were female and three were male. All subjects were between the ages of 21 and 50 years old and were not compensated for their time. Data from only five of the six participants were used. One of the participants did not return for the second day of testing.

Procedure

Prior to the start of any testing each subject was required to fill out driver history and informed consent forms. Each test session began with the subject driving out to the testing area to become familiar with the operation of the test vehicle. Once at the "normal" tangent the subject s donned the occlusion goggles and drove tangents. Once the participants were accustomed to the fit of the goggles in the clear state they practiced driving with the goggles working.

Before the subjects were permitted to participate in the experiment they were informed as to the purpose of this research, the track configuration, safety precautions, operation of the vehicle and apparatus, and the testing procedures. The test administrator instructed each subject to take as many "looks" as they felt were necessary to maintain lane integrity. The participants were told that deviation from the travel lane would void that test and that test would have to be rerun. If the subject repeatedly left the travel lane the subject would be dismissed. The administrator reminded them of the importance of not leaving the travel lane to the work zone side.

Following the instructions the subjects began the testing. The order of the test was as follows:

1. Two runs through the tangent section
2. Six through the simulated work zone, three each direction
3. Two addition runs through the tangent.

The testing order was repeated for the second work zone width. The test was randomized with respect to lane width and location of the work zone.

RESULTS

One participant's data was excluded from the analysis, since the data set was incomplete. A couple of participants remarked that they did not enjoy driving next to concrete barriers, whether it is at the test facility or on the highways.

Data analysis was performed on various data configurations. Tangent data was analyzed separately from the work zone data; left from right; and narrow from wide lane widths.

Data from the four tangent trials were analyzed in two groups. Average workloads were calculated and compared. They were compared to determine the presence of a learning curve. If there was a learning curve the mental workload values for the second set of tangents would be lower. No significant difference was found between the two sets of tangent data. The mean mental workload for the first set of tangents was .3221, while the mean for the second set was .3560. This, although not statistically significant, does show a trend. There appeared to be a carryover effect of the stress of the work zones. The carried-over stress may prove to be significant in larger studies.

There was a significant difference (Pr>F = .0062) in the driver workloads for work zones and "normal" tangent sections. The mental workload while driving in a work zone was found to be .3764, while the mental workload requirement of the tangent section was found to be .3390.

The gender of the participant significantly affected the data. The average workload for females for all features was significantly higher than males. The mean for females was .3981 while males had an average mental workload of .2954 (Pr>F = .0001). The difference between genders for mental workload values in work zones was smaller. For females the average mental workload in a work zone was .4097 and for males it was .3131. The difference, although smaller, was still significant.
Pavement Marker Spacing 20 ft. o/c
220 ft. Work Zone Taper
630 ft. Work Zone
190 ft. Concrete Barriers
10.5 or 12.0 ft. Travel Lane

FIGURE 1. TEST TRACK CONFIGURATION.
No significant difference was found between wide and narrow work zones. The average workload values in a wide work zone was .3551 while for narrow work zones it was .3911. No significant difference was found between the direction of travel in the work zone.

DISCUSSION

In the process of designing this experiment it was thought that a learning curve associated with the use of the occluded vision apparatus would be found. The analysis of the two sets of tangents sections, before and after the work zone sections, revealed no significant difference between the two sets of tangent data. The average mental workload for the tangent sections after the work zone sections was higher than the previous tangent sections. The difference might prove to be significant if a larger number of participants were used.

The differences, although not statistically significant, may indicate the presence of a carry over stress effect. Some of the stress of driving in a highway work zone might be carried over into the final tangent sections, resulting in higher workload values. The higher workload values might also be due to the anticipation of the end of the test. The subjects may have been anxious to finish the test and this may have tainted the results.

The work zones exhibited a significantly higher mental workload requirement than the tangent sections. The work zone required more mental processes capacity for both males and females. The average increase in mental workload for all subjects was .0374. One subject had a change in workload as high as .0788, while the smallest change between work zone's and tangent's was .005. For individual workloads please see Figure 2. On average the mental workload increased 10 percent in the work zone.

Gender accounted for a lot of the variability in the data. Women had significantly higher workloads in both roadway features than their male counterparts. The difference between the sexes may be due to a combination of two factors. The gender difference in workloads may be due to the visual search processes and risk acceptance behaviors. Women tend to be more field dependent than men (11). Persons who are field dependent find it more difficult to extract pertinent visual information from a complex background. In this experiment a field dependent person may have to allocate more mental capacity to extract the visual information. They have to work harder to "see" the raised pavement markers delineating the travel lanes. Risk acceptance behavior may also play a part in the lower mental workloads in the males. Males tend to be more willing to accept risk and therefore may be willing to have their eyes occluded for a longer period of time, reducing the mental workload.

In this experiment the width of the travel lane next to the work zones did not significantly affect the mental workload values obtained. The workload values for narrow travel lanes in the work zones were higher than the wide lanes. Previous studies have shown that the lane width does affect the drivers mental workload (Brackett, unpublished data). A narrow lane results in higher workloads. The results of this test seem to agree with the previous studies regarding lane width.

The direction of travel or the location of the work zone relative to the car did not significantly affect the mental workload. The workload values for the work zone on the right of the car were slightly higher than the values for the work zone on the left of the car. It was not anticipated that the location of the work zone would have a significant effect on the mental workload.

A possible explanation for the results of the variables not showing significant effects is the small number of individuals tested. A larger sample size would have yielded better statistics. The way the test and statistical analysis was set up, many variables that were thought might be significant turned out not to be so. A better study design would have been to run more people and to reduce the number of variables. By testing only one lane width in the work zone, such as a 12 ft. lane, a better correlation would have possibly resulted.

The effectiveness of this test was also limited by the physical constraints of the test track. There was only 210 feet of portable concrete barriers available for the test. Of the 210 feet, 20 feet were used as angled portions to simulate the end treatment of a highway work zone. A longer work zone section delineated by concrete barriers would have potentially resulted in being able to determine if the workload increased within the work zone. This would have shown an additive effect of the stress, if any.

Other physical constraints beside the length of the work zone reduced the effectiveness of this study. The simulated work zone lacked many of the features of a "conventional" work zone. The simulated section did not have any construction equipment behind the barriers, there was no traffic in either direction, it was a relatively short, strait section, and it had ample room on one side of the travel lane. If the experiment had traffic or curves the workload may have been higher. Since this experiment did not have all of the stressing factors, it provides a conservative estimate of the driver workload in a highway work zone.
FIGURE 2. AVERAGE WORKLOADS.

FIGURE 3. WORK ZONES AND CURVES.
The mental workload imposed on drivers by the work zone were compared with the mental workload requirements of two horizontal curves. The mental workload for a work zone was less than the workload for a 3 degree curve, but not for the 9 degree curve. The workload for the work zone was 0.348, while for the 3 and 9 degree curves respectively were 0.374 and 0.401 (Brackett and Roush, unpublished data). See Figure 3.

To correlate the results obtained in this pilot study with the "real" world the occluded vision test could be used in conjunction with some physiological testing procedures, such as the galvanic skin response test. Both the occluded vision and physiological tests could be performed at a testing facility to establish a correlation between the results. Then, the physiological test could be performed in actual highway work zones to determine how much additional workload is required in the "real" world.

This experiment investigated only the mental workload in a highway work zone at 45 mph on cruise control. Additional studies should be performed to determine the mental workload at different speeds. Eventually it might be determined that some combination of location of the concrete barriers and the travel width have an affect on the mental workload. There may be design manuals based on the driver workloads. For example, recommending 10.5 feet lanes when the work is on the left, but 12 feet lanes when the work is on the right.

This experiment, although limited in its size, has shown that there is a significant increase in the mental workload in a highway work zone. It has also shown that there is a significant difference in the workloads between the sexes. From this research one might suggest that other methods of traffic channelization may be investigated, methods that protect the work zone like concrete barriers, but do not increase the mental workload.

REFERENCES

Single and Multiple Vehicle Ran-Off-Road Rollover Crashes: A Study of the Effect of Vehicle and Road Type

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This paper documents the results of a statistical analysis of vehicle overturn in ran-off-road crashes. The Illinois State Data Base was the source for the subset of the data used in the study. Two sets of models were developed to predict the probability of overturn: one for passenger cars (large, small) and the other for trucks and vans (single unit trucks, tractors and semi-trailers, vans, and pickup trucks). The predictor variables included were vehicle type, location (urban, rural), highway type (interstate, other), and accident type (single vehicle, multi-vehicle) with overturn (yes, no) as the dependent variable. A statistical model was developed for each of the vehicle types to determine the probability of driver injury. The same predictor variables, adding overturn, were used with driver injury (fatal, incapacitating, non-incapacitating, possible, none) as the dependent variable.

INTRODUCTION

Vehicle overturns, or rollovers, are associated with approximately 10,000 fatalities each year. In 1990, there were 9,565 fatalities in rollover accidents. The severity level of rollover is the second highest in the number of fatalities per registered vehicle, falling only behind frontal crashes (1). Rollovers are particularly complicated accidents since there are many possible contributing factors that have to be considered, some of which include road characteristics and conditions, roadside geometrics and features, vehicle characteristics, driver actions, and the dynamics of the collision.

It has been found that approximately 76 percent of rollovers occur outside the shoulder, i.e. in ran-off-road crashes (2, 3). Also in these studies by Viner, rollover was said to be the leading cause in ran-off-road fatalities. These accidents occurring off the roadway are not always well documented in police accident reports due to a lack of detail in the report forms. The results of this insufficient detail are accident data bases that lack specific accounts concerning off-road terrain. Therefore, when examining the data base in the event of an overturn on the roadside, it is not always possible to determine the factors causing the vehicle to rollover. More detail in the accident data about the roadside terrain would give more information as to the cause of overturn in off-road accidents (4, 5). Knowing that such a large percentage of overturns occurs off the road, it is important to investigate this area of the road as it pertains to rollover.

OBJECTIVE

In this analysis, the objective is to develop statistical models to determine the probability of rollover in ran-off-road crashes. The predictor variables to be considered for use in the models will be determined from the past literature and from univariate analyses. This is the preliminary step to a model for driver injury. By using the same predictor variables and forcing overturn into the model, models will be developed to determine the probability of fatal and serious injuries in the event of an overturn.

BACKGROUND LITERATURE

Past research has indicated that vehicle characteristics influence the propensity of a vehicle to overturn. Vining (6) pointed out that smaller cars are, in general, at a greater risk compared to larger cars when involved in accidents. The main problem with smaller cars, according to Council, et al. (6), is that they overturn more frequently at lower speeds. Griffin (7) stated that smaller cars have a greater tendency to overturn than larger cars on any class of road. Council, et al. (6) and Partyka (8) have reported that the number of driver fatalities increases with decreasing car mass. As vehicle mass and wheelbase length decrease, the tendency to overturn increases (2, 6, 7, 8, 9, 10, 11).

Not only do small passenger cars have a large tendency to overturn, but small utility vehicles and small
pickup trucks have been observed to frequently overturn, as well. Research has shown that overturn occurs more frequently in utility vehicles, pickup trucks, and small passenger cars. Both small pickup trucks and small utility vehicles have very high numbers of fatalities in single-vehicle rollover crashes per 10,000 registered vehicles. The smallest vehicles in these three vehicle classes - pickup trucks, utility vehicles, and passenger cars - have the highest death rates per 10,000 registered vehicles for both rollover and non-rollover crashes (11). In terms of overturn, for either fatal or incapacitating occupant injuries, rollovers are the most dangerous collision type for small vehicles (I). This is an important finding when considering the progressive downsizing of the automotive fleet, which could lead to severe results.

The roadway itself is a contributing factor to rollover in that one type of road may be more conducive to rollover than another. In other words, one roadway may be more functional to an errant vehicle such that the vehicle may regain control and continue unharmed. For example, several studies have indicated that rollover more frequently occurs on rural roads than on urban. Mengert, et al. (14) found urban/rural to be a very influential factor on rollover. Griffin found that passenger cars were more apt to overturn on county roads than on city streets (7). In the Western states, rural roads have a significantly higher percentage of rollover than urban roads (13). In other studies, when controlling for road type, the number of rollover vehicles varied widely among the urban and rural roads, suggesting this as an aspect of vehicle rollover that needs to be examined in greater detail (12, 15).

However, this type of urban/rural analysis needs to be more specific about the type of road, i.e. interstate or other roads. The reason for this is that urban and rural two lane roads differ greatly in quality, due to the higher speed limits, higher degree of curves and steeper grades, and the variability of road surfaces that occur. Rural interstates, however, are not of :nerior quality to urban interstates. They generally have wide clear zones. Therefore, if a vehicle leaves the roadway, unless the vehicle overturns, there are not many other factors that can damage the vehicle or driver. On urban interstates, there are many more roadside structures that can disable the vehicle when impacted. Because of these differences, some distinction must be made between the two in this type of analysis.

There are large differences in the tendency to rollover given the number of vehicles involved in the accident. The percentages of rollover in single vehicle crashes was found to be much higher than the percentage of rollover in multi-vehicle crashes (9). By distinguishing between the two possibilities, single or multi-vehicle crashes, a large difference in rollover tendency is found.

PROCEDURE

Data

The Highway Safety Information System (HSIS) is a data collection system in which accident data are collected into data files on a state basis. The data are not entered into the file directly from police accident reports or from inventory files. The states have implemented their own safety information systems with several data edits and quality checks. Therefore, the HSIS has more consistent and accurate data than those data bases that have not undergone this series of checks and edits.

There are five HSIS state data bases: Illinois, Michigan, Minnesota, Maine, and Utah. Illinois offers a set of variables indicating the series of events of an accident: FRST_INV, SND_INV, and THRD_INV, which correspond to the first, second, and third involvements that a vehicle encounters, respectively. Illinois is the only state to offer such variables with good descriptive detail on the nature of the event.

Related to the involvement variables in Illinois are the variables F_INVLOC, S_INVLOC, and T_INVLOC which are the first, second, and third involvement locations, respectively. These six involvement variables help to reveal the most probable sequence of events that occurred in the accident. These variables help to determine what each event was, where - on or around the roadway - it occurred, and in what order.

The Illinois state data base consists of four different files: Accident Data, Roadlog File, Bridge (Structure) File, and RR Grade Crossing File. The two that were used for this analysis are the Accident Data File and the Roadlog File. The Accident Data File is broken into three subfiles: the Accident, Vehicle, and Occupant Subfiles. For this analysis, the Accident and Vehicle Subfiles were merged with the Roadlog file. First, the Accident and Vehicle subfiles for the years 1985-1987 were combined, followed by the Accident and Vehicle subfiles for 1988-1989. These merged files were then combined with the Roadlog files for the corresponding years. To complete the merge, the Accident/Vehicle/ Roadlog file for 1985-1987 was combined with that for 1988-1989, resulting in a continuous file for the years 1985-1989.

Some of the accident reports were prepared by the driver or by other police officers who were on the scene. These driver and desk reports were deleted from the file to keep as much consistency in the data as possible. Only the reports that were submitted by state, county, or city officers who were on the scene were considered.
A vehicle was defined as a ran-off-road vehicle if F_INVLOC, S_INVLOC, or T_INVLOC was coded as the right or left of the roadway. Since this paper is concerned with rollovers in ran-off-road crashes, if the vehicle was not coded as a ran-off-road vehicle, that vehicle was deleted from the file. However, in some cases, the only information about a vehicle is that it ran off the road. There is no indication as to what might have caused the vehicle to leave the road or what happened to the vehicle once it did. These observations contained no helpful information, so they were also deleted. At this point in the process of creating the data subset, there were 117,742 vehicles in the file.

A new variable named ROADTYPE was created to distinguish between urban and rural roads, using the variables URB_AREA (urban area) and FUNC_CLS (functional class). The variable separates the road into six categories: interstate, other, and unknown for both rural and urban areas. Those vehicles that were coded as either urban or rural unknown roads were deleted from the file.

Both vehicles that overturned and those that did not were kept in the file for means of comparison. However, if the vehicle is a motorcycle, the accident will, in general, always result in a rollover. Thus, the motorcycle accidents are misleading since a very large majority of the motorcycle accidents are rollovers. If the vehicle was a motorcycle, the observation was deleted. Other vehicle type groupings that were deleted due to insignificant involvement frequencies were buses, other vehicles, and unstated vehicles. After this final deletion, the analysis file had 114,252 vehicles.

An accident was defined according to the number of vehicles involved in the accident. If only one vehicle was coded in the accident, the vehicle was coded as a single vehicle accident. All other accidents were coded as multi-vehicle accidents.

A vehicle was coded as an overturn accident if the vehicle rolled over in either the FRST_INV, SND_INV, or THRD_INV. In this analysis file, there were 16,369 vehicles coded as overturns.

After eliminating those vehicles that were not significant to the purpose of this study, the analysis file has 114,252 observations. In summary, the Illinois Rollover Analysis File contains only vehicles that:

- were not coded as motorcycles, buses, or other or unstated vehicles.

**Analysis Method**

Two sets of generalized logit models were developed during the course of this study. In the first set of models, vehicle overturn (yes, no) served as the dependent variable and vehicle type, location (urban, rural), highway type (interstate, other), and accident type (multi-vehicle, single vehicle) served as candidate predictor variables. In the second set of models, the dependent variable was driver injury (fatal, incapacitating, non-incapacitating, possible, none). Predictor variables included the previous four, plus overturn.

The generalized logit models were fit to the data by means of the CATMOD procedure in the Statistical Analysis System (SAS). By this procedure, maximum likelihood estimates of model parameters and estimates of cell probabilities were obtained.

The purposes of fitting logit models to the data were basically two fold: (1) to simplify and smooth the relationships between the dependent variables and the candidate predictor variables and (2) to allow for non-zero probability estimates of overturn (or driver injury) even if one or more combinations of the different levels of the candidate predictor variables in the data set produced no overturns (or injuries).

**Probability of Vehicle Overturn**

Two logit models predicting vehicle overturn were fit to the data: one model for passenger cars (large, small) and the other for trucks and vans (single unit trucks, tractors and semi-trailers, vans, pickups). All attempts to fit a single logit model to the data to predict vehicle overturn (by collapsing across all six types of vehicles) proved unsuccessful.

Since the dependent variable was binary, one logit was defined for each combination of all possible levels of the candidate predictor variables. The logits were calculated as the logarithms of the ratios of overturns to non-overturns.

The candidate predictor variables were defined as indicator variables. The variable highway, for example, had two levels: interstate (coded as a 1) and other (coded as a -1). Other predictor variables were coded in a similar fashion.

A backward stepwise regression procedure was used to develop the fitted models. Chi-square ($\chi^2$) tests served to test the goodness of fit for each model considered. The
simplest model that provided an adequate fit to the data (at a significance level of \( \alpha = 0.05 \)) while preserving the hierarchical nature of the terms included in the model was selected as the "best" model.

**Probability of Driver Injury**

Logit models to predict driver injury were developed for each of six different vehicle types: large cars, small cars, single unit trucks, tractors and semi-trailers, vans and pickups. Since the dependent variable used in each of these six models was multinomial with five levels of injury, four logits were defined for each combination of all possible levels of the candidate predictor variables. The first logit was calculated as the logarithm of the ratio of non-injuries to fatal injuries. The second logit was calculated as the logarithm of the ratio of incapacitating injuries to fatal injuries. These calculations continued similarly with the remaining injury levels.

The candidate predictor variables were defined as described in the previous section. Model selection was also carried out by the same procedure described in the previous section.

In order to determine the relative importance of overturn in the production of driver injury in each of six different vehicle types, additional logit models were developed. The best fitted model for a given vehicle type, as previously defined, was referred to as Model 1. Model 2 was derived from Model 1 by excluding all interactions that involved the variable overturn. Model 3 was derived from Model 2 by deleting overturn as a main effect. Model 4 was derived from Model 3 by deleting all remaining terms except the intercept. For the i-th model, a chi-square \((G_i^2)\) goodness of fit statistic was calculated. The proportion of the variance explained by the "best" fitted model was defined as follows:

\[
\rho_M^2 = \frac{G_4^2 - G_1^2}{G_4^2} \quad \text{(Eq. 1)}
\]

The information contained in this statistic was partitioned by defining three additional equations:

\[
\rho_I^2 = \frac{G_2^2 - G_1^2}{G_4^2} \quad \text{(Eq. 2)}
\]

\[
\rho_D^2 = \frac{G_3^2 - G_2^2}{G_4^2} \quad \text{(Eq. 3)}
\]

Equation 2 represents the proportion of the variation in driver injury in the full fitted model explained by the interaction of overturn and other effects; Equation 3 represents the proportion of the variation in driver injury in the full fitted model explained by overturn acting alone, i.e. as a main effect in the equation. The fourth equation explains all remaining variation in driver injury in the full fitted model that is not explained by Equations 2 and 3.

It should by noted that, by definition,

\[
\rho_M^2 = \rho_I^2 + \rho_D^2 + \rho_F^2 \quad \text{(Eq. 5)}
\]

Therefore, if \( \rho_D^2 \) is large relative to \( \rho_F^2 \), overturn explains a major portion of the variation in driver injury in the full fitted model.

**RESULTS**

**Probability of Vehicle Overturn**

Vehicle overturn in ran-off-road accidents was modeled for two classes of vehicles: (1) passenger cars (large cars, small cars) and (2) trucks and vans (single unit trucks, tractors and semi-trailers, vans, and pickups).

**Passenger Cars**

For passenger cars involved in ran-off-road accidents, a 10 degree of freedom (df) model was found to provide an adequate fit to the observed data \( (G_4^2 = 8.15; p = 0.2276) \). The main effects contained in the model were location (rural, urban), highway type (interstate, other), vehicle type (large car, small car), and number of vehicles involved in the accident (one, more than one). Four first order interactions of the main effects, and one second order interaction of the main effects, were also included in the full, fitted model.

The parameter estimates associated with the intercept, the main effects, the first-order interactions, and the second-order interaction are omitted to save space. The expected probabilities of passenger car overturn in ran-off-road accidents in each of the 16 combinations of location, highway, vehicle type, and number were calculated from these parameter estimates. These expected probabilities of overturn are plotted in Figure 1.
**Trucks and Vans**

The model to predict the probability of vehicle overturn for four types of trucks and vans is slightly more complex than the previous model for passenger car overturn. This model contains 23 degrees of freedom (an intercept term, four main effects, six first-order interactions, and two second-order interactions) and is seen to provide an adequate fit to the data ($G^2 = 12.27$; $pr = 0.1984$).

In Figure 2, the expected probabilities of truck and van overturns in run-off-road accidents for 32 combinations of location, highway, vehicle type, and number are provided. As before, these expected probabilities are based upon the parameter estimates for the model.

**Probability of Driver Injury**

In the analyses that follow, six logit models were developed to predict driver injury in run-off-road accidents -- one model for each of the six vehicle types considered in the previous section. The main purpose of developing these models was to determine if, and to what degree, vehicle overturn can explain driver injury. In other words, these models were to determine how much of the variability in driver injury can be explained by whether the vehicle overturned during the accident or remained upright.

**Large Cars**

The logit model to predict the severity of the injuries sustained by drivers of large cars involved in run-off-road accidents contains four main effects and four first-order interactions and provides an adequate fit to the data ($G^2 = 37.57$; $pr = 0.1069$). Figure 3 is a plot of the expected probabilities of fatal (K), incapacitating (A), non-incapacitating (B), and possible (C) injury for each of the 16 different combinations of run-off-road accidents. These probabilities were calculated from the estimated parameters for the fitted model.

The $\rho^2$ for the fitted model (which is analogous to $r^2$ in standard multiple regression) is 0.985. The $\rho^2$ for the fitted model can be partitioned into three component parts, as shown below:
FIGURE 2. PROBABILITY OF OVERTURN FOR TRUCKS AND VANS INVOLVED IN RAN-OFF-ROAD ACCIDENTS IN ILLINOIS (1985-89)

Or, 67.4 percent of the predictive power of the model results from the variable overturn used as a main effect. Another 0.6 percent of the predictive power of the model comes from the interactions between overturn and other variables in the model, while the remaining 32.0 percent comes from all remaining effects.

Small Cars

For run-off-road accidents, involving small cars, a logit model with 40 degrees of freedom was found to provide a reasonable fit to the observed data ($G_m^2 = 23.48$, $pr = 0.4917$). The four main effects in the model were again location, overturn, number, and highway. Four first-order interactions and one second-order interaction served as the other terms in the model, along with the intercept term. Expected probabilities for different severities of driver injury are provided in Figure 4.

The $\rho^2$ for the fitted model is 0.989. The $\rho^2$ for the variable overturn acting alone is 0.716. Therefore, 72.4 percent of the predictive power of the model is explained by overturn serving as the main effect in the model.

Single Unit Trucks

The logit model developed to predict driver injury for run-off-road accidents involving single unit trucks contains 44 degrees of freedom and provides adequate (though less good) fit to the data ($G_m^2 = 30.65$, $pr = 0.0600$). Figure 5 provides the expected probabilities of injury severity for drivers of single unit trucks involved in run-off-road accidents.

The $\rho^2$ for the fitted model is 0.924. The $\rho^2$ for the variable overturn acting alone is 0.368, i.e., only 39.8 percent of the predictive power of the model is explained by overturn serving as the main effect in the model.

Tractors and Semi-Trailers

The logit model developed to predict driver injury for tractors and semi-trailers contains 48 degrees of freedom and provides adequate fit to the data ($G_m^2 = 18.45$, $pr = 0.2981$) with four main effects, five first-order interactions, two second-order interactions and an intercept term. The $\rho^2$ for the fitted model is 0.989, of which 0.438 (44.3 percent) is accounted for by the main effect of overturn.
ACCIDENT-INVOLVED VEHICLES BY LOCATION, OVERTURN, ACCIDENT TYPE, HIGHWAY TYPE AND DRIVER INJURY LEVEL (N=54,674)

FIGURE 3. PROBABILITY OF DRIVER INJURY FOR LARGE CARS INVOLVED IN RAN-OFF-ROAD ACCIDENTS IN ILLINOIS (1985-89)
**ACCIDENT--INVOLVED VEHICLES BY LOCATION, OVERTURN, ACCIDENT TYPE, HIGHWAY TYPE AND DRIVER INJURY LEVEL (N=31,899)**

<table>
<thead>
<tr>
<th>Location</th>
<th>Overturnd</th>
<th>Single-Vehicle</th>
<th>Injury Level</th>
</tr>
</thead>
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<tr>
<td>Rural</td>
<td>Overturned</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rural</td>
<td>Up Right</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rural</td>
<td>Multi-Vehicle</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Urban</td>
<td>Overturned</td>
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<td>Urban</td>
<td>Up Right</td>
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<tr>
<td>Urban</td>
<td>Multi-Vehicle</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**FIGURE 4. PROBABILITY OF DRIVER INJURY FOR SMALL CARS INVOLVED IN RAN-OFF-ROAD ACCIDENTS IN ILLINOIS (1985-89)**
Figure 5. Probability of Driver Injury for Single Unit Trucks Involved in Run-Off-Road Accidents in Illinois (1965-80)
Figure 6 plots the expected probabilities of different driver injury severities based on the estimated parameters from the fitted model. It should be noted that in this regression model there were three cases (accident/severity combinations) for which there were no observed data, i.e., for which cell frequencies equalled zero:

Case 1 There were no fatally injured drivers reported in multi-vehicle overturn accidents on urban interstates.

Case 2 There were no drivers reported to have sustained possible injuries in multi-vehicle overturn accidents on urban interstates.

Case 3 There were no fatally injured drivers reported in single vehicle, non-overturn, non-interstate accidents in urban areas.

However, the fitted model allowed us to estimate that the probability of fatal injury in Case 1 was 0.0170, the probability of possible injury in Case 2 was 0.0804, and the probability of fatal injury in Case 3 was 0.0002. Note that although there were no recorded fatalities in Case 1 or Case 3, the estimated probability of fatal injury in Case 1 is many times more likely than in Case 3.

Vans

The logit model that was developed to predict the probability of injury severity for van drivers involved in run-off-road accidents was somewhat simpler than the others in this section. This regression model contains 20 degrees of freedom and provides an adequate fit to the data ($\chi^2_{20} = 13.21; \text{pr} = 0.3542$). For this accident subset of vans involved in run-off-road accidents, location is not a significant predictor variable for purposes of estimating driver injury. Figure 7 provides expected probabilities of driver injury. Therefore, the variable location (urban/rural) is not contained in this model.

This model has a $\rho^2$ of 0.833, and the $\rho^2$ for overturn as a main effect in this model is 0.557. Thus, overturn as a main effect accounts for 66.9 percent of the predictive power of the model.

**Pickup Trucks**

The last logit model developed during the course of this study included only those accidents involving pickup trucks that ran off the road during the accident. The fitted model contained 28 degrees of freedom and adequately depicted the observed data ($\chi^2_{28} = 37.93; \text{pr} = 0.3814$). The fitted model had a $\rho^2$ of 0.952 and a $\rho^2_{\text{overturn}}$ of 0.576. Thus, overturn acting as a main effect accounts for 60.5 percent of the predictive power of the model. Figure 8 provided expected probabilities of driver injury severities based upon the estimated parameters from the fitted model.

**DISCUSSION AND CONCLUSION**

As stated at the outset of this paper, and as confirmed by the analyses performed herein, rollover or overturn accidents are the result of many different factors acting singly and in concert. Nevertheless, in a probabilistic sense, vehicle overturn can be predicted with reasonable accuracy based upon a relatively few input variables and several of their interactions: class of vehicles (passenger cars vs. trucks and vans), location (urban/rural), highway type (interstate/other), number of vehicles involved in the accident (one/more than one), and vehicle type (large car/small car or single unit truck/tractor semi-trailer/van/pickup).

With the logit regression models developed in this study to predict driver injury, it was possible to derive expected (non-zero) probabilities of driver injury in different accident configurations (e.g., fatal injuries to drivers of tractors and semi-trailers involved in a single vehicle, overturn accident on a rural interstate) for which no injury data were available.

Finally, the analyses performed on all six vehicle types indicated that overturn is a powerful predictor of driver injury. (See Table 1.) For run-off-road accidents, vehicle overturn is probably the most salient in explaining the level of injury sustained to drivers of small cars, and somewhat less salient in explaining the level of injury sustained by drivers of single unit trucks.
FIGURE 6. PROBABILITY OF DRIVER INJURY FOR TRACTORS AND SEMI-TRAILERS INVOLVED IN RAN-OFF-ROAD ACCIDENTS IN ILLINOIS (1985-89)
ACCIDENT–INVOLVED VEHICLES BY OVERTURN, ACCIDENT TYPE, HIGHWAY TYPE AND DRIVER INJURY LEVEL (N=3,346)

FIGURE 7. PROBABILITY OF DRIVER INJURY FOR VANS INVOLVED IN RAN-OFF-ROAD ACCIDENTS IN ILLINOIS (1985-89)

ACCIDENT–INVOLVED VEHICLES BY LOCATION, OVERTURN, ACCIDENT TYPE, HIGHWAY TYPE AND DRIVER INJURY LEVEL (N=12,384)

FIGURE 8. PROBABILITY OF DRIVER INJURY FOR PICKUP TRUCKS INVOLVED IN RAN-OFF-ROAD ACCIDENTS IN ILLINOIS (1985-89)
ACCIDENT-INVOLVED VEHICLES BY LOCATION, OVERTURN, ACCIDENT TYPE, HIGHWAY TYPE AND DRIVER INJURY LEVEL (N=12,364)

FIGURE 8. (CONTINUED) PROBABILITY OF DRIVER INJURY FOR PICKUP TRUCKS INVOLVED IN RAN-OFF-ROAD ACCIDENTS IN ILLINOIS (1985-89)

TABLE 1. PROPORTION OF VARIABILITY IN DRIVER INJURY EXPLAINED BY FITTED LOGIT MODELS (AND FACTORS WITHIN THOSE MODELS) FOR SIX VEHICLE TYPES

<table>
<thead>
<tr>
<th>Proportion of Variability in Driver Injury Explained by the Fitted Model, and Factors within the Model</th>
<th>Vehicle Type</th>
</tr>
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<tr>
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<td>Single Tractors</td>
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<td>0.664</td>
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<td>Unit &amp; Semi-Cars</td>
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<td>Trailers</td>
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<td>Vans</td>
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<td>0.952</td>
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7. Lindsay I. Griffin, III. Probability of Overturn in Single Vehicle Accidents as a Function of Road Type and Passenger Car curb Weight. Texas Transportation Institute, November 1981.


The Effects of Light Rail Transit Vehicles On An Arterial Street System

GREGORY D. KRUEGER

Light Rail Transit (LRT) systems are becoming a very popular mass transit alternative. An effective model, however, is needed to determine the effects of adding an LRT to an existing network of arterials. The Federal Highway Administration's network computer model, TRAF-NETSIM, has been considered for this purpose. The objectives of this study were to determine the feasibility of using the 1988 release of TRAF-NETSIM to effectively model an LRT within a network of arterials, and to determine the impacts of adding the LRT. The results of this study show that TRAF-NETSIM is an effective tool only if the network of arterials is either very small or has very low v/c ratios. On large networks, it has been determined that the 1988 version of TRAF-NETSIM is an inadequate tool to model the effects of an LRT. Initial results on the effects of adding an LRT to a small network of arterials shows that a network will never fully recover from one train being introduced into the network, and that as train headways are decreased, delay within the network increases dramatically. Also included in the paper are suggestions for further uses and studies using TRAF-NETSIM for additional LRT applications.

INTRODUCTION

Background

Traffic congestion continues to grow in many major cities in the United States. Improvements to the traffic flow on arterials and freeways have become more and more difficult because of decreased revenue, a lack of available space in urban areas to expand or widen arterials, and a concern over the impact of automobiles on the surrounding environment. To provide an alternative to these automobiles, many city planners and transportation engineers are looking for new methods to improve mass transit in urban areas (1).

One of these alternatives are Light Rail Transit, or LRT Systems. LRT is defined as an urban transportation system that uses electrically powered light weight rail cars operating singly or in short trains on fixed duc-rail guideways, grade separated or at grade, loading passengers from low to medium height platforms (2).

Light Rail Transit is especially attractive because the rail systems need a minimal amount of space in which to operate, and these rail systems are less costly to install than heavy rail systems. LRT can be easily installed on an existing street, street median, bus lane, railroad line, or in new corridors exclusively used by LRT systems. Costs are minimized through the use of at-grade street crossings, unlike the heavy, or rapid, rail transit systems found in Chicago or New York City.

As LRT systems become increasingly popular, it is necessary to compare the impacts of increasing levels of service to the LRT against the impacts to automobile traffic and buses. Studies by Berry (1) and Cline et al. (3) have focused on the effects of light rail transit systems on a very microscopic basis, studying only the at-grade intersections. It has yet to be determined what effects the pre-emption of traffic signals caused by a crossing train will have on traffic progression and on the surrounding arterial network. To assist in this analysis, a tool or procedure must be developed that is both convenient to use and produces accurate results.

Study Objectives

Locating light rail crossings four hundred feet from the nearest intersection would be a nice luxury for most traffic engineers to work with. Most light rail systems, however, are being built into an existing network of arterials where there is limited space with which to work and build the system. New light rail systems are being built:

1. On existing roads to operate under mixed traffic flows;
2. In mass transit only roadways;
3. Within transit malls; and
4. On exclusive right-of-ways.
Regardless of where the tracks are installed, they must, at some point, cross a roadway. As Berry (1) pointed out, crossing times will vary depending on the type of protection at the intersection. It is therefore necessary to develop a method by which to analyze the effects of adding a light rail transit system to an existing network of arterials.

The objective of this research is two-fold. The first objective is to determine whether TRAF-NETSIM can be used effectively and efficiently to model a network of arterials with a light rail transit system. The second objective is to determine the effects of the light rail transit system in the existing network.

Related Research

The first step in analyzing the impacts of an at-grade light rail crossing is to research traffic engineering solutions that have been previously proposed. Building on this data to effectively model an arterial system in which to place a light rail transit system, it is possible to determine the effects of the LRT.

Richard Berry (1) has done extensive research on light rail transit systems which are currently in operation throughout the United States and Canada. The project was oriented towards isolated at-grade crossings with exclusive right-of-way for the light rail system. Through his research, he determined that there are four methods of crossing strategies:

1. No Protection (Passive warning systems);
2. Flashing light units only;
3. Flashing lights with gates; and
4. Traffic control signals.

All four methods have varying operation characteristics:

1. The advance warning time;
2. The actual train crossing time;
3. The clearance time;
4. The first vehicle lost time; and
5. The effective crossing blockage time.

As an example, the advance warning time for a crossing with flashing lights only will differ from a crossing with a gate. The Manual on Uniform Traffic Control Devices (MUTCD) (4), and the Manual for Railway Engineering (5) mandate that gates be lowered at least twenty seconds before the actual arrival of the train. It is possible for a crossing with passive warning systems or flashing lights only to have virtually no advance warning time because of the characteristics of certain drivers attempting to beat the train across the intersection.

Cline et al. (3) focused their study on the delay motorists faced as a result of light rail transit vehicles crossing the roadway and blocking traffic for a certain period of time. It was concluded that person-delay was the desired measure of effectiveness, and that this delay would be most easily understood if it was assigned a monetary value. In this study, TRAF-NETSIM, a computer traffic simulation program developed for the Federal Highway Administration (FHWA), was used.

Cline’s study focused on an isolated light rail train crossing. Using NETSIM, Cline determined that at 400 feet from the crossing, the effects of the light rail train crossing the arterial were minimized. Cline therefore determined that in order to minimize the effects of a light rail train crossing on traffic, the intersection of the LRT and the roadway should be placed at least 400 feet from the nearest intersection of crossing roadways.

METHODOLOGY

Simulating a Light Rail Transit System

The first task performed was to determine whether TRAF-NETSIM could effectively model a Light Rail Transit system. This was done by creating a simple arterial and then determining the best method of modeling the system. NETSIM does not explicitly model light rail transit systems. NETSIM does, however, allow for the use of buses and bus routes. NETSIM also allows the user to vary the size and operating characteristics of the buses.

The first attempt modeled two-lane, two-way links, allowing buses only on these links. NETSIM, however, requires at least one unrestricted lane exit between each link (6). The bus only model, therefore, could not be used. Likewise, creating two-lane links, with one lane restricted for the use of buses only, and the other closed, cannot be used for the same reason.

Three reasonably acceptable methods were then formulated and tried with success. The first involves using a two-lane link, with one lane restricted for bus use only, and the other open for through traffic. To model the train system, vehicle input volumes can be easily set to zero, and no vehicles are allowed to turn on to the tracks. Although this method can be used, the lane width must be taken into account as the distance vehicles must cross is now twice as large as the condition with only one lane.

The second method modeled the LRT system through the use of signals only and completely ignored the use of an arterial. This method can be used to model the system only in cases where automobiles have priority at
the intersection, or with pre-timed signals for the train. It should be noted, however, that a light rail transit system is considered on time if it arrives at the station within thirty seconds, either before or after, of its scheduled arrival time (1). Caution must therefore be used when giving the trains absolute priority with pre-timed signals.

The final, and most acceptable method, was to use a one-lane link. The lane must be unrestricted, that is to say any vehicle may use it. Volumes for automobile traffic, however, must be set to zero, and traffic must not be allowed to turn onto the lane. Buses using this model can be run and varying headways can therefore be used. The signals at the intersections with other arteries can either be pre-timed, with the same reservations given above, or actuated.

Building an Arterial Network within NETSIM

Because the scope of this study was mainly to determine the feasibility of using NETSIM to model a light rail transit system, it was decided to construct a network with a relatively high (greater than .70) v/c ratio, while maintaining the best level of service possible through optimizing signal offsets and timings.

The initial network consisted of a main arterial just over one half mile in length. This arterial carried traffic east and west, using three lanes in each direction, plus left turn pockets at every intersection as needed. Two hundred feet north and south of this arterial was a smaller parallel arterial, again carrying traffic east and west. As a smaller arterial, however, this arterial was modeled as two-lanes in each direction, with left turn pockets at each intersection as needed. The light rail tracks were located fifty feet south of the main east-west arterial. One main north-south arterial was modeled, bisecting the network. This arterial had the same characteristics as the main east-west arterial. Finally, six one-way streets were placed in alternating north-south directions, with three on either side of the main north-south arterial. These streets were located at four hundred foot intervals along the main east-west arterial. Traffic signals were placed at every intersection, with protected left-turns at locations where a vehicle was required to cross three lanes of opposing traffic. The remainder of the intersections were given permitted left turns only. All signal timing and offsets were optimized using TRANSYT-7F.

Due to the size of this network, and low g/c ratios, many traffic backups and delays occurred within the network. There were no spillbacks indicated by NETSIM within the network, however enough backups occurred to cause a somewhat linear increase in the number of vehicles in the network at one time. NETSIM, however, only allows a maximum of 1500 vehicles at any particular point in time in the network. The size of the network, and the ensuing delays caused the number of vehicles within the network to rapidly increase above that 1500 vehicle limit.

In an attempt to fix this problem, input volumes were reduced until the system could run for over twenty minutes without exceeding the 1500 vehicle limit. When this finally occurred, the volumes within the network were reduced to between 20 and 40 percent of capacity. When the train was simulated within the system no effects were noticed at any of the intersections. It was therefore decided that a very large network was not feasible to model with NETSIM, and that a smaller network must be chosen.

The final network, shown in Figure 1, was a much simplified version of the original network. The main street (Main) has three lanes in each direction, with left turn pockets as needed. A main arterial (Critical Ave.) bisects Main Street. Two one way streets cross main, one on either side, four hundred feet from Critical Ave. Signals were used at each of the intersections, with left turn protected phases included at those locations where a vehicle would have to turn left across three lanes of opposing traffic. Light rail transit tracks were modeled through the use of traffic signals. These signals were set up at both 50 feet and 200 feet south of Main Street.

Modeling the LRT to Check its Effects

For this study, two cases were examined, using the tracks fifty feet from Main Street. The first case involved running one train through the network, using pre-timed signals. The train entered the simulation six minutes after the run began, with the understanding that the first few time intervals in NETSIM cannot be used for analysis purposes because it takes about three cycles for the system to attain a steady-state condition. After the train passed, the network was allowed to run for one hour to determine the length of time required to return to a steady-state, pre-train condition, in which the effects of the train were mitigated.

The second case involved running two or three trains through the network with varying headways. Because of the ninety second cycle length recommended by Transyt, it was determined trains should be run with headways of multiples of ninety seconds. Therefore, trains were run through with headways varying from 90 seconds to ten minutes and thirty seconds.
FIGURE 1. DIAGRAM OF NETWORK USED.
RESULTS

Using TRAF-NETSIM to Model an LRT

The first objective of this study was to determine the feasibility of using TRAF-NETSIM to effectively model an LRT within a network of arterials. In performing this research, many deficiencies were noted which are inherent in the latest (1988) release of TRAF-NETSIM. The most noticeable deficiencies are as follows:

1. Age of TRAF-NETSIM;
2. Link length calculations;
3. Format of output; and
4. Ability to Expand.

Age of TRAF-NETSIM

The current version of TRAF-NETSIM was officially released on May 31, 1988. Most software packages are written at least one year prior to their release, which puts the last revisions of NETSIM to be in 1987, with most modifications happening earlier than that. In 1987 the most widely used computer processor was the Intel 80286, or the 286 based computer. These computers are very limited in their speed and the amount of calculations which they can perform. As a result, NETSIM has quite a few built in limitations. The most notable are:

1. Maximum of 150 Links;
2. Maximum of 75 Nodes;
3. Maximum of 18 Actuated Controllers; and
4. Maximum of 1500 vehicles within the network.

These four limitations within NETSIM make modeling an anything larger than a small downtown area virtually impossible.

Because of the size limitations, NETSIM cannot be used to model a large network. The limits of only 150 links and 75 nodes puts strains on the number of city blocks that can be modeled. NETSIM defines a node as an intersection, and a link as a street section. Each direction of an arterial between two nodes is considered one link. Through these two limits, the size of the network cannot very easily reach that of any metropolitan area in which it is desired to model a light rail transit system.

The second limitation is the number of actuated controllers allowed in the model. TRAF-NETSIM only allows 18 in any given network. This too will limit the size of the network that is being used. As stated earlier by Berry (7), a train is considered on time if it is either thirty seconds ahead of schedule, or thirty seconds behind schedule. If full train priority is required, actuated control is needed for the train system. Allowing only 18 actuated nodes limits the length of the LRT line to 18 or less arterial crossings. If the entire network that is to be studied is actuated, again only a small portion of the network may be studied.

The third limitation is the maximum number of vehicles allowed at any given time within the entire network. The first network modeled for this study contained 24 nodes, and about 140 links. Within this network, however, queues were long enough that automobile traffic accumulated to a point where there were over 1500 vehicles within the network at any given time. This limitation will either require the user to reduce the size of the network, or reduce the incoming traffic volumes. If traffic volumes are reduced, v/c ratios are decreased, and an accurate picture of the network cannot be determined. Therefore the size of the network must be reduced, again limiting what can be done with NETSIM.

Length of Link Calculations

NETSIM does not use clearly understandable methods to define link lengths, nor does NETSIM take the width of lanes into account when determining the width of a crossing. The NETSIM manual defines the length of the link as the distance from the stop line for the upstream link to the stop line of the subject link (6). As defined in Figure 2, the lengths are X and Y. Problems, however, occur in data entry when the widths of the arterials, W1 and W2 are taken into account. Logic says that if W1 is a two-lane arterial, with 12 foot lanes, then W1 should be 24 feet, while if W2 is a four-lane arterial with 12 foot lanes, then W2 should be 48 feet. NETSIM asks that the lengths X and Y be equal, as NETSIM does it's calculations on the intersections as if the node were a point. If X and Y are equal, then L1 and L2 cannot be equal if lane widths are taken into account. For the purposes of this study, link lengths were changed to reflect the differences in lane widths that occurred within the network.

Output Format

NETSIM produces output in a method that is neither convenient to work with, nor possible to discern what the impacts are at each requested time interval. The NETSIM output can be printed at user specified time intervals. These time intervals are usually the same as the cycle length used for a majority of the signals within the system. If NETSIM is run for a long period of time, and output is produced every cycle, NETSIM can easily produce a five hundred or more page output file. Going through this output is very time-consuming because for
FIGURE 2. LINK LENGTH DIAGRAM.
network-wide applications, only one or two lines of the output pertain to the entire network for each time interval.

NETSIM also provides data that is not accurate in describing the network over individual time periods. Figure 3 is a graph of the differences in delay times in vehicle-seconds from the control condition for varying train headways. NETSIM, in creating its output, averages it's output over the entire time the simulation has been running. In this instance, NETSIM produces point B by averaging points A and B. Likewise, point E is the average of points A, B, C, D, and E. Because of this averaging, it appears that the network never fully recovers from adding the train, when the outcome that one would expect would be to see a nearly sinusoidal curve for the train with the 10.5 minute headway. Figure 3, however, shows the effects of additional trains to be nearly a step function. Because of these average values, however, this graph, and the data from which it was obtained, cannot be used to determine exactly what happens over any given time interval.

The data that NETSIM does not average are all cumulative values, which must be altered to determine the effects over a certain time interval. In a program as sophisticated as NETSIM, the user should be able to use and graph the data without first altering it into a format that is easy to understand.

Ability to Expand

One of the advantages of Light Rail is that it may be grade separated at certain intersections if the delays to traffic are large. Interrupting the LRT tracks to grade separate a certain intersection is not possible with NETSIM either. If actuated controls are used to give the LRT priority at certain intersections, the train would have no way of crossing an arterial within NETSIM, except for an at-grade crossing. If the LRT track is interrupted, the vehicle being used to model the train would not be able to cross and continue on the other side of the arterial. Although for the purpose of most analysis, all crossings are at-grade, if the need arises for an intersection to be grade separated, NETSIM cannot be used.

The Effects of A Light Rail Transit Vehicle

The second objective of this study is to determine the impacts of adding a light rail transit vehicle to an existing network of arterials. All of these results were obtained from the total network output, using the network in Figure 1. The values were obtained by averaging three runs with different random number speeds for each run, but using the same random numbers for each case. For the purposes of this study, a control network was run to obtain a picture of the network before a train was added.

The Effects of One Train

The control case, or the "no train" line on Figure 4, showed no sudden changes in delay, as the slope remained constant throughout the time period being studied. This data is cumulative, therefore a straight line of constant slope is to be expected.

The second line in Figure 4 shows the effect of one train on the system at point A. At some point, it was expected that this line would become parallel to the control case, which would have meant that the network had recovered from the impacts of one light rail transit vehicle. At point D, however, the slope begins to once again increase. This little bump could easily be attributed to the variabilities associated with the random number seed. By only running the network three times, there is bound to be a statistically significant amount of error.

Effects of More than One Train

The third line depicts the effects of running three trains through the network with ten and a half minute headways. The first train arrives at point A, the second at point B, and the third at point C. There is a noticeable change in slope at both points A and B, however point C appears to have no change in slope. The difference in the effects of running each of the trains can be attributed to the initial shock that the network faces on the first train.

NETSIM allows the network to run until a steady-state condition is attained. When the first train arrives, it is a great shock to the system. The system is functioning smoothly until point A. At that point, traffic suddenly faces an unexpected stop that had not previously been seen. At point B, it again faces another shock, however this time the system responds less severely because traffic is already slightly backed up from the first run. By the third train, the negative impact is not as significant. It can be determined from this graph, then, that the first time a train arrives in the network, the shock is severe, however, after a certain number of trains pass through, they will no longer have an impact on traffic.

Figure 5 represents the effects of running two trains with a four and a half minute headway. Points A and B represent the train entering the network. As in Figure 4, the second train causes a second shock on the system, however this shock is greater than the second shock with the ten and a half minute headway. It can be concluded that the system needs more time than the four and a
FIGURE 3. CHANGES IN DELAY TIMES.

FIGURE 4. CUMULATIVE VEHICLE DELAY FOR ALL THREE CASES STUDIED.
FIGURE 5. CUMULATIVE VEHICLE DELAY. TWO TRAINS, 4.5 MINUTE HEADWAYS.
half minutes it now has to return to a steady-state condition.

CONCLUSIONS

Research on this topic is far from complete. It is important to remember that the main objective of this study is to determine if TRAF-NETSIM is an effective tool with which to model a Light Rail Transit system within a network of arterials. By using NETSIM and through coding various network configurations, it has been determined that for large urban arterial networks, this version of TRAF-NETSIM is an unacceptable tool. It can, however, be effectively used for smaller networks and networks with low v/c ratios.

Recommendations

Further testing does need to be done on TRAF-NETSIM. The following recommendations are given for this testing:

1. Wait for a New Version;
2. Use a Small Network;
3. Run Many Trains Thru the System;
4. Use many Random Number Seeds; and
5. Develop a Tool to Reduce Output.

Wait for a New Version

Since the last release of TRAF-NETSIM was in 1988, the FHWA is due to release an updated version relatively soon. This version may remove, or reduce many of the limitations in the current version. If waiting is not an option, it may be possible to obtain a copy of the Fortran source codes, and alterations may be made to the NETSIM source code by an experienced programmer.

Use a Small Network

There were some problems specific to the size of the network used in the original study which caused the backups to occur. As a result, there are size limitations that must be adhered to. A network must therefore be chosen that will not reach the inherent limits within NETSIM. It may also be an advantage to re-create an actual arterial network, rather than the theoretical network used in this study, within an urban area in which traffic volumes and signal timing patterns are known.

Run Many Trains

For this study, the effects of only two or three trains were evaluated. When three trains were run at a 10.5 minute headway, it appeared that the third train had little or no adverse impact on the arterial system. If this trend continues on the fourth and fifth train and beyond, the conclusion can be made that the level of service decrease for automobile traffic is insignificant compared to the increased level of service for LRT commutes.

For the purposes of this study, the trains were run using the Time Period function in NETSIM. Use of these time periods is only needed in the case where pre-timed signals are used and when train headways are greater than 120 seconds. If actuated control is used, a bus route may be programmed using the speed and stopping characteristics of an LRT.

Use Many Random Number Seeds

TRAF-NETSIM uses random numbers in the input stream to simulate the random elements of traffic flow. Among these random elements are bus dwell times at a stop, the traffic routing of an individual vehicle, and the characteristics of each driver/vehicle combination (4). For the purpose of this study, three random number seeds were used. By using only three sets of data, a very high variability was evident in each case.

For performing actual modeling in the future, a minimum of ten random number seeds should be used. Statistically, it is recommended that a minimum of thirty random number seeds be used, however, NETSIM has been used in the past with ten random number seeds performing accurate calculations.

Develop a Method to Reduce Data Quickly

The output netsim produces can become very tedious to wade through. Theoretically, NETSIM can easily produce a six hundred page output file, from which only thirty lines may be needed. Although it is possible to print out all six hundred pages and manually type the required lines into a spreadsheet or data analysis package, this uses a large amount of time and resources. For this study, two methods were used in an attempt to streamline this process. The first was using Word for Windows 2.0, opening up two documents, and performing cut and paste operations on both the output file, and a new file, created solely for the necessary data.

The second method involves writing a quick PC SAS routine to perform this same function. However PC SAS uses about 28 megabytes of space on a hard drive, and unless it is used on a regular basis, the costs involved with it are very great compared to the benefits received.

A tool needs to be created which can easily perform the task of reducing this output file. A simple program in C, Fortran, or Pascal could easily be written and
distributed that would reduce the time and resources used in this data reduction process.

REFERENCES


ACKNOWLEDGEMENTS

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An Evaluation of Traffic Operations At Exit Lane Drops

MARTY T. LANCE

INTRODUCTION

The traffic demand along a route varies from segment to segment according to the traffic exiting or entering the route. The overall efficiency of a route can be improved by providing lane capacity only where it is (or soon will be) required. Lane drops are used to reduce the number of through lanes on a freeway when traffic demands decrease. Lane drops can be subdivided into three classes: exit lane drops, lane splits, and lane terminations. The scope of this study was limited to the consideration of exit lane drops only. An exit lane drop refers to the location where one or more through lanes on a freeway become ramp lane(s) to a different destination.

Exit lane drops can cause difficulties for unaware motorists. In typical freeway situations drivers do not expect to preform a lane-change maneuver in order to continue in a through lane. Motorists unaware of their position in an exit lane may be forced to exit and become lost or delayed. At interchanges with poor route continuity, motorists may falsely assume that the major route continues on the through lanes, when in fact they must exit to continue along the major route.

A driver needs certain information in order to travel through an exit area smoothly. To negotiate a lane drop area successfully, motorists need the following: knowledge of the impending lane drop, location of the lane drop, choice of an appropriate maneuver, and time to execute that maneuver (1).

Background

In order to study traffic operations at lane drops, the different aspects of the freeway near and at the exit only ramp need to be examined. Factors such as geometric design, signing, and pavement markings influence how well the lane drop is communicated to the motorist. This section contains design principles for lane drops and also traces the development of traffic control devices currently used in lane drop areas.

Geometric Design

Exit lane drops can be classified based on their geometric design. A one-lane exit lane drop refers to the location on a freeway where the right most lane becomes an exit ramp that carries motorist to a different destination (Figure 1). Similarly, at two-lane exit lane drops motorists traveling in either of the two right most lanes will find themselves traveling in exit only lanes. Another case is the two-lane exit with an optional lane. This configuration differs from the two-lane exit because vehicles in the far right lane are required to exit, while motorist in the adjacent lane may either exit or continue with the through traffic (Figure 2).

Previous studies have found that accidents increased when interchange geometrics were poorly visible. Goodwin's study on the operational effects of geometric design suggested eight principles for freeway lane drops (2).

- Provide continuous visibility
- Minimize attention-dividing conditions
- Provide adequate transition cues
- Create lane drops on the right side of the freeway
- Coordinate visual and operational drop
- Provide adequate escape area
- Notify drivers that a lane is not continuous
- Use adequate traffic control devices

Signage

A motorist's success in the negotiation of a lane drop is partly based on their comprehension of traffic signs. In 1976, Roberts and Klipple researched driver expectancies at freeway lane drops in a study design to compare four exit panel messages (3). The signs included MUST EXIT, EXIT ONLY, ONLY, and EXIT LANE panels. The MUST EXIT and EXIT ONLY panels were found to be preferable with little difference in efficiency of the two messages. The MUST EXIT sign had a slight
FIGURE 2. TWO-LANE EXIT WITH AN OPTIONAL LANE
advantage in influencing a driver to make the correct decision to reach a certain destination or follow a certain route. The report suggested adoption of either panel because of the slight differences.

In 1976, the Michigan Department of Highways conducted a study to determine the effectiveness of a black-on-yellow EXIT ONLY panel as opposed to a silver-on-green panel (4). The researchers recommended the black-on-yellow panel. This study concluded that the black-on-yellow panel was better at attracting the driver's immediate attention than the silver-on-green color scheme; therefore, drivers reacted quicker to the message.

In 1976, Lunenfeld and Alexander directed a study to determine if devices in the current MUTCD were satisfactory based on driver expectancy (5). Their analysis indicated that down arrows should be used for advance guide signs and upward sloping arrows should be utilized on overhead exit direction signs. The study also recommended that diagrammatic signs be implemented at exit lane drops where the through route does not continue after the interchange. Diagrammatic signs were also suggested for exit lane drops with an optional lane while conventional signs were suggested at lane splits without an optional lane (see Figure 3). Their study also found that diagrammatic signs yielded quicker responses by drivers (5).

Pavement Markings

While a significant amount of research has been performed to evaluate the effectiveness of signs at lane drops, only a few studies considered pavement markings. The California Department of Transportation (Caltrans) developed pavement markings that would supplement the EXIT ONLY signs at exit lane drops. These markings consist of an 8-inch wide solid white strip that extends 300 feet in advance of the gore and 8-inch wide by 3-feet long white strips separated by 12-foot gaps that extend for an additional half-mile in advance of the gore (Figure 4) (6). In 1975 Caltrans recommended the adoption of the markings into the MUTCD (7). The 1984 MUTCD manual included the markings as an optional treatment.

Previous Evaluations

Previous research has analyzed several different types of data to evaluate the efficiency of lane drops. In Goodwin et al. study of lane drops, it was noted that the efficiency of traffic operations is commonly measured using the average travel time or the average overall speed (8). Although this type of measure is a good indicator of the flow of traffic, Goodwin et al. commented, it is not sufficient for evaluating lane drops. A more analytical description of the changes in speeds and flow patterns is necessary to evaluate the subject lane drop. Values of speed and flow for small sections of the lane drop area can be collected. By plotting the speed and flow in each of the smaller sections against the distance upstream of the site, one can trace the effects of lane drop geometry on traffic operations (8).

Lane change behavior and erratic maneuvers are commonly studied in lane drop situations. Erratic maneuvers can be used as an indicator of the effectiveness of lane drop communication. In a study of raised pavement markers at lane drops, Pigman grouped erratic maneuvers into seven categories: (1) cut across gore area, (2) crowded weave, (3) stopped, (4) slowed drastically, (5) swerved, (6) stopped and backed, and (7) multiple error (9). Speed changes, lane changes, and motorists in the gore area can create problems for both exiting and through traffic. As erratic maneuvers increase driver anxiety, hazardous conditions, or even accidents may result.

In Pigman and Agent's study of lane drops, brake light applications were recorded (9). Brake light applications were summarized for the median, middle, and shoulder lanes. The study reported a decrease in the number of erratic maneuvers with the implementation of raised pavement markers, while there was no significant change in brake light rates. This study concluded that brake light applications were not a useful indicator of the efficiency of lane drop communication.

Recent research by Brackett et al. of urban guide signs deficiencies in Texas included signs at exit lane drops (10). Although most deficiencies were related to specific locations, the report listed two frequent problems associated with the guide sign system: lack of adequate lane assignment and lack of advanced information. Lack of adequate lane assignment caused drivers to become confused about which lane they should be in to perform a desired movement. Lack of advanced information prevented drivers from having enough time to make an appropriate decision and a smooth transition through the lane drop area.

Ogden in 1989 studied the effects of double white stripes 300 feet prior to the gore area with short dashes preceding the stripes on driver lane change behavior at three exit ramps in Houston (11). The data collected was limited to the recording distance of the camera; 500 to 700 feet. The comparison of erratic lane changes revealed that the striping had a positive effect.
METHODOLOGY

Site Selection

This study evaluated traffic operations at a two-lane exit with an optional lane. One of the preliminary tasks was site identification. The site selection was based upon the following: geometric design, accessibility, traffic volumes, current signing, and pavement markings. Ideally, structures in the lane drop area should be located such that cameras placed on these signs and bridges can record the motorist approach up to a half-mile ahead of the gore.

The lane drop selected for this evaluation was a two-lane exit with an optional lane located in Houston. This exit is located on the I-45 Northbound freeway, at the interchange with I-610 Eastbound, south of downtown. Traffic control devices at the site include the standard EXIT ONLY panel and a white stripe separating the exiting and optional lanes for approximately 600 feet prior to the gore.

Data Collection

The site contained a pedestrian bridge on which video cameras were placed to record data for 600 feet prior to the gore. Filming occurred in different time intervals throughout the day on three consecutive weekdays: a Tuesday (1:00 p.m. to 3:00 p.m.), a Wednesday (8:00 a.m. to 6:00 p.m.), and a Thursday (7:00 a.m. to 1:00 p.m.). Information collected on videotape included the following: lane changes and their locations, erratic maneuvers, and volumes of exiting vehicles in the three right lanes.

Data Reduction

In order to reduce the data, locations or zones within the lane drop area were identified (see Figure 5). The 600 feet prior to the gore were divided into 100-foot zones. Rumble strips that appeared every 100 feet on the right shoulder, were used to delineate the zones. The sixth zone contained the 20 feet prior to the painted gore and the gore area, while the other zones contained approximately 100 feet of freeway each. Each lane was labeled beginning with the right most lane, or the exit only lane as lane one, and the optional lane as lane two. However, approximately 200 feet from the gore the optional lane becomes wide enough to hold two vehicles. Therefore, it was necessary to distinguish between the exiting portion (2E) and the through portion (2T) of this lane.

Lane changes that involved entering or exiting either the optional or the through lanes were recorded. For each lane change, the lane that the change began and ended in was noted. The zone the change originated in was also recorded. A single lane change occurred when a motorist moved out of one lane and into an adjacent lane. Erratic lane changes involved crossing the intermediate lane in order to reach a third lane. By recording the beginning and ending lanes for each change, it could be determined whether or not the movement was erratic.

The location of erratic maneuvers were determined with the same criteria as lane changes. In addition to location, the type of erratic maneuver was also designated with the following number code.

1. Erratic lane change
2. In/out of shoulder
3. Cut across the gore (moving from a through to an exiting lane)
4. Cut across the gore (moving from an exiting to a through lane)
5. Suddenly slowing (indicated by other cars having to pass)
6. Slowing/stopping to merge
7. Straddling white line

Traffic volumes were also obtained from the videotapes. The volumes were recorded for the exit only lane (lane 1), the optional exit lane (lane 2E), and the optional through lane (lane 2T). These counts were made for fifteen minutes at the beginning of each hour. The cars were counted as they passed the point of the painted gore.

Once the information was extracted from the video tapes the data were entered into a spreadsheet. Each entry in the spreadsheet indicated either a traffic volume, lane change count, or an erratic maneuver count for that particular fifteen-minute time interval. The entry's position in the spreadsheet specified the lane location, and zone that the activity occurred in within the lane drop area. An additional column contained the code classifying each erratic maneuver that occurred in that zone. The data were then summarized into total counts according to day, hour, lane, zone, and type of activity.

Data Analysis

The data were analyzed by comparing traffic volumes, lane change counts, and erratic maneuver counts by time of day, lane placement, or zone location. The various counts were compared in numerous ways to investigate the possibility of a relationship between two
I-45NB to I-610 EB
Two-Lane Exit with an Optional Lane

Sign Structure

Lane 3  Lane 2  Lane 1  Shoulder

Zone
6
5
4
3
2
1

FIGURE 5. SCHEMATIC OF SITE
or more factors. The volumes counted for the exit only lane, the optional exit lane, and the optional through lane were also used to calculate the percent of exiting vehicles for each lane.

FINDINGS

Percentages of Exiting Vehicles

The fifteen-minute traffic volume counts for the exit only, optional exit, and optional through lanes were reviewed to determine if the percentages of vehicles in each lane varied throughout the day. The volume was composed of approximately a 50-20-30 split between the optional through, optional exit, and exit only lanes respectfully. Figure 6 illustrates how this composition varied only slightly throughout the day with an increase in the percent of vehicles in the optional through lane during the morning and evening peak periods.

Classification of Erratic Maneuvers

Erratic maneuvers were classified using the seven codes listed in the data reduction section of this report. The number of erratic maneuvers were summarized by the number of vehicles making an erratic maneuver and the total number of erratic maneuvers made. Of the total number of erratic maneuvers, those multiple errors committed by a single motorist were included in each appropriate category. Figure 7, a pie chart classifying erratic maneuvers, demonstrates that erratic lane changes, or multiple lane changes, have the greatest percentage of erratic maneuvers (74 percent), while gore related errors constituted approximately 20 percent. The number of vehicles making an erratic maneuver was used in all subsequent analyses.

Variations in Zones

Figure 8, which shows the rate of erratic maneuvers and lane changes at different distances from the gore as a function of non-peak hourly volume, shows that the rate of lane changes varies slightly in comparison to the rate of erratic maneuvers. The decrease in the rate of lane changes in the final zone is expected because this zone only contains 20 feet of normal freeway lanes and the painted gore, as opposed to the previous 100-foot zones.

The rate of erratic maneuvers increases drastically as the motorists approach the gore. The widening of the optional exit lane contributes to the drastic increase that occurs at 200 feet from the gore. Again the rate decreased in the final zone because of the size of the zone.

Comparisons

After comparing traffic operations in different zones it was necessary to examine variations in hourly counts of lane changes and erratic maneuvers throughout the day. In Figure 9 it appears that the number of lane changes remained fairly constant while the erratic maneuver counts seemed to peak during the 7:00 a.m. peak morning hour, during the noon hour, and again during the 5:00 p.m. peak evening hour.

The next step was to take the frequency of erratic maneuvers and lane changes and convert them into a rate by dividing each hourly count by the non-peak hourly volume; Figure 10 illustrates these rates. The rate of erratic maneuvers does not demonstrate the same peaking trends shown in Figure 9 (frequency of erratic maneuvers and lane changes). Both erratic maneuvers and lane changes rates were greatest during the hour from 12:00 to 1:00 p.m.

After conducting various comparisons regarding erratic maneuvers and lane changes, erratic lane changes were considered specifically. Erratic lane changes were divided into changes moving from exiting to through lanes and changes moving from through to exiting lanes. The composition of erratic lane changes varies throughout the day, as shown in Figure 11. During the peak morning and evening hours changes from exiting to through lanes were a larger portion of the total erratic lane changes, while during the non-peak periods changes from the through to exiting lanes constituted a larger portion of the total.

Fifteen-minute counts of traffic volumes and hourly counts of erratic lane changes traveling from the exiting to the through lanes were compared from 7:00 a.m. to 6:00 p.m. (Figure 12). Erratic maneuvers seemed to follow the same trend as traffic volumes during the peak morning and evening periods. However, it is apparent that the trend does not continue during the noon lunch hour. This discrepancy may be due to the fact that the traffic volumes were only fifteen minute counts done at the beginning of each hour. By just counting traffic volumes from 12:00 to 12:15 p.m., the larger portion of the peak lunch hour traffic may not have been included. For a more accurate comparison hourly traffic volume counts would be needed.

CONCLUSIONS

This study examined traffic volumes and traffic operation at a two-lane exit with an optional lane in order to gain an understanding of the motorists behavior in these lane drop areas. Erratic maneuvers, lane changes, and traffic volumes were compared throughout the day and as
FIGURE 6. COMPOSITION OF TRAFFIC VOLUMES

FIGURE 7. ERRATIC MANEUVER CLASSIFICATION
FIGURE 8. RATES OF ERRATIC MANEUVERS AND LANE CHANGES VERSUS DISTANCE FROM THE GORE

FIGURE 9. FREQUENCY OF ERRATIC MANEUVERS AND LANE CHANGES VERSUS TIME OF DAY
FIGURE 10. RATES OF ERRATIC MANEUVERS AND LANE CHANGES VERSUS TIME OF DAY

FIGURE 11. COMPOSITION OF ERRATIC LANE CHANGES VERSUS TIME OF DAY
FIGURE 12. ERRATIC LANE CHANGES AND VOLUMES VERSUS TIME OF DAY

the motorists approached the gore in order to investigate possible relationships that may exist between these different factors.

Overall the I-45 site appeared to be fairly active; many lane changes and erratic maneuver were observed. The most apparent relationship in this study was the comparison of erratic lane changes and traffic volumes as opposed to time of day (Figure 11). As traffic volumes increased during the morning and evening peak periods it appears that the number of vehicles moving from the exiting to the through lane erratically also increases. This trend could be explained by examining the type of motorist using the facility. During the peak periods a majority of the drivers are commuters traveling to and from work. As traffic volumes increased the through lanes approached capacity and a queue formed in the optional through lane. In order to bypass this queue commuters are continuing in the exiting lanes and then attempt to merge into the through lanes.

RECOMMENDATIONS

Future research needs to collect additional data at other sites and data on driver behavior following the installation of selected pavement markings. In the additional studies it is recommended that the peak hour data be separated from the non-peak hour data. This suggestion is based on the driver behavior observed in this study. The commuters traveling through the lane drop area during peak periods are familiar with the interchange and rarely needs additional traffic control devices to negotiate the lane drop situation. Commuters trying to "beat the queue" will probably not be affected by additional pavement markings. Motorists traveling in the non peak periods, however, may be affected by the implementation of additional pavement markings.

REFERENCES


Comparison of Trip Length Frequency Distributions As Observed in the Household Surveys Vs. Theoretical Estimates

MARK LUSZCZ

A study was undertaken to evaluate the accuracy of two Trip Length Frequency Distribution (TLFD) models, the Improved Trip Length Frequency Distribution Model (ITLFDM) and the One Parameter Model (OPM). Results from the 1990 Household Travel Survey were compared to the estimations predicted by the models to determine the level of accuracy in three areas: the error in the average trip length, the shape of the curve, and the error in the tails of the curve.

No difference was found in the TLFD's produced by either linked or unlinked trips. Results indicated that both models produced adequate to excellent estimations of the TLFD's observed in the survey data. There is no pressing need to improve upon these models at the current time, and either can be used with equal confidence.

However, to address two minor deficiencies in the models, an adjustment method is suggested based on a correlation between the average household income of an urban area and the percent error in the tails of the OPM. This adjustment causes a significant increase in the accuracy of the tails of the distribution, but often decreases the accuracy of the average trip length.

INTRODUCTION

Transportation facilities in today's urban areas are extremely important, both economically and socially. The future planning of these facilities must be done as carefully and accurately as possible. To assist in planning for future needs, an urban travel demand forecasting process has been developed that attempts to quantify the amount of travel on a given transportation system. This process is vital to today's growing urban areas. The process consists of four major steps: trip generation, trip distribution, mode usage and trip assignment.

Trip generation is the process by which urban activity is translated into a specific number of trip ends. Trip distribution determines the destination of these trip ends. Mode usage finds the way these trips were made (i.e. automobile, transit, etc.) and trip assignment assigns each trip to a specific route. Each of these four steps has different data requirements.

Trip distribution is based primarily on Newton's Law of Gravity. The model works on the premise that the number of trips produced in zone A and attracted to zone B is directly proportional to the number of trips produced in A, the number of trips attracted to B, and inversely proportional to the separation of A and B. Separation, in this case, refers to the travel time between the two zones (I). One of the inputs to trip distribution is Trip Length Frequency Distributions (TLFD's) for each trip purpose under analysis.

TLFD's describe the percentage of trips that occur at each travel time in an urban area. In the past, only extensive origin-destination surveys could accurately describe these distributions (2). To eliminate the need for these expensive and time consuming surveys, two models (the One Parameter Model, OPM, and the Improved Trip Length Frequency Distribution Model, ITLFDM) were developed to estimate TLFD's.

The purpose of this study was to determine the accuracy of the two models in estimating observed TLFD's. A secondary purpose was to determine if a need existed to improve the models.

This report will first discuss current modeling techniques before reviewing previous research on TLFD's. This will be followed by the problem statement and study design of this research. The results will be described next, concluding with this study's findings.
CURRENT PRACTICE

For transportation planning uses, a "network" is created for an urban area. This network is an abstract representation of the urban area, and can be readily analyzed using a computer. For the network, the urban area is divided into "zones." Zones vary in size, with smaller zones in areas of dense development and large zones in areas of sparse development.

Each zone has a center of activity, called the centroid (similar to the center of mass of an object). The connection of centroids from a given urban area make up the network. Within the network, all trips originate and end at the centroids of the zones. To determine the travel time between two zones, the distance between their centroids is divided by an estimated speed between the two zones.

The trip distribution process is designed to distribute trips between zone pairs in accordance with the percentage of trips estimated to occur at each travel time, as input via the TLFD (1).

PREVIOUS RESEARCH

In an extensive study conducted by Voorhees and Associates, which dealt mainly with the factors affecting trip length, a method for estimating TLFD’s was developed utilizing the Gamma Distribution. This study indicated that the Gamma Distribution achieved better results than three other distributions: the Poisson, Chi-Squared and Pearson Type III distributions, which all have a shape similar to TLFD’s (3). Pearson, Stover and Benson (2) of TTI (Texas Transportation Institute) determined that the Weibull Distribution gave results similar to the Gamma Distribution; however, the Gamma Distribution was more stable over different urban areas.

The functional form of the Gamma distribution is:

\[ g(t) = \frac{b^a}{\Gamma(a)} t^{(a-1)} e^{-bt} \]  

(1)

\[ g(t) : \text{relative density of trips taking time } t \]
\[ a : \text{shape parameter} \]
\[ b : \text{scale parameter} \]
\[ t : \text{time} \]
\[ e : \text{base of natural logarithms (2.71828...)} \]
\[ \Gamma(a) : \text{Gamma function, } (a - 1)! \]

The TTI study developed a method of estimating TLFD’s using the Gamma Distribution. Unlike the Voorhees study, which used the maximum likelihood method to obtain the parameter values of the distribution (3), the TTI study used a direct approach. The model was calibrated on nine urban areas. This was done by non-dimensionalizing each area by dividing each separation by the average trip length. Then the parameter values were found using non-linear least squares regression routine. The only inputs needed to use this model are:

1. The average travel time, which can be accurately estimated by as little as 600 surveys (unfortunately, an accurate TLFD cannot be produced from these few surveys) (4), and
2. The maximum separation as determined by the network of the urban area.

The TTI study decided that the model gave the best results when the two parameters, \( a \) and \( b \), where equal (see Equation 1). Therefore, the model was calibrated finding only the shape parameter and setting the scale parameter to that. Hence the name: the One Parameter Model. This study also found a linear relationship between the maximum separation of trips that occurred, and the maximum possible separation of the network (2).

The model gave, overall, adequate to excellent results. However, two deficiencies were found in this model: the consistent underestimation of trips at longer separation (the right tail, defined as greater than 60% of the maximum separation) and at the shortest separations (the left tail, defined as less than four minutes travel time), especially in larger cities. To address these problems, an Improved Trip Length Frequency Distribution Model (ITLFDM) was developed in 1979 (5).

This model also utilized the Gamma Distribution (equation 1). However, the calibration process was different. Instead of using set parameter values as the OPM does, the ITLFDM determines new parameter values each time it is used. Like the Voorhees study, the maximum likelihood method is used. This method requires the geometric mean trip length, which the report related to the arithmetic mean (average) trip length, for each trip purpose.

This model apparently shows much better results in the two problem areas of the One Parameter Model, while sacrificing less than 3 percent accuracy in the estimation of the average travel time, which was considered acceptable in that report. These improvements were found mainly in the large cities of the study, while the ITLFDM gave essentially equivalent estimates to the OPM when used in smaller cities (5).
PROBLEM STATEMENT

The major issue addressed by this study is the accuracy of the two models, the OPM and the ITLFDM, in estimating TLFD's. By finding the accuracy of these current models, it will then be determined whether or not they need to be modified, recalibrated, or completely revised.

Additionally, there was a question as to whether there is a difference between the TLFD's produced by either linked or unlinked trips (see Table 1); this was also addressed.

STUDY DESIGN

Description of Data

The data used in this research was obtained from household travel surveys conducted in five Texas urban areas in 1990 (see Table 2). These surveys were conducted by several consulting firms for the Texas Department of Transportation. The households were selected from a stratified random sample. Each member of each household was asked to keep a diary of their trips for one day.

It was decided to eliminate any intrazonal trips (any trip that originates and ends in the same zone) from the data set. This was done for two reasons. First, neither model was designed for use with intrazonal trips. While the rest of the distribution creates a relatively smooth curve, the inclusion of intrazonal trips would create a sharp drop between the one and two minute time intervals. Secondly, intrazonal trips are not important to the trip distribution analysis. Because of the zonal manner in which the analysis is used, any intrazonal trip would originate and end at the same point, producing zero miles traveled.

Analysis Approach

The TLFD's were compiled from the 1990 travel survey by city and trip purpose. The trip purposes are defined as: Home Based Work (HBW) trips, a trip from home to work or from work to home; Home Based Non Work (HBNW) trips, any non work trip that either originates or ends at home; and Non Home Based (NHB) trips, any trip that does not originate or end at home. Next, the average trip lengths found from this data were input into each model, along with the maximum possible separation, determined by the network of each urban area. The output of the model was compared to the survey data. Three areas were evaluated:

1. The average trip lengths
2. The shape of the curves
3. The tails of the curves

The average trip length and the tails of the curves were measured by looking at the percent errors, defined as the model value minus survey value, divided by the survey value, multiplied by 100. Two statistical measures were employed in measuring the accuracy of the shape of the curves:

1. Coefficient of Determination, R². This is a value ranging from zero (no correlation) to one (perfectly correlated, the model and survey data matching exactly). Negative values down to negative one can also be found when dealing with inversely correlated values.

2. Kolmogorov-Smirnov (K-S) Goodness of Fit Test. This test compares the cumulative distributions of the models and the surveys. The difference between the survey and model is checked at each point along the distribution. The largest difference found is the measure used in this test. If it is less than a critical value (based on the number of observations) for a given significance level (in this case, 80 percent significance level), then the model is said to "pass" the test (0).

Objectives

The following specific objectives were decided upon for this research:

1. To determine if there is a difference in the TLFD's produced by using either linked or unlinked trips from the 1990 travel survey.
2. To determine how well each model estimates the TLFD's observed in the 1990 travel survey.
3. To determine whether the models are effective as they are or if they need improvement, based on the findings of objective 2. If such improvements were warranted, suggestions for this improvement would be given.

RESULTS

The complete statistical results of this research are lengthy and often very similar within certain categories. Therefore, only selected results are shown (in tables and figures) that exemplify the remaining results.

Linked vs. Unlinked

Table 3 shows the R² value produced by both models on all five cities (for HBW trips) when the survey data
### TABLE 1. DEFINITION, LINKED VS. UNLINKED TRIPS

<table>
<thead>
<tr>
<th>Trip Type</th>
<th>Defined as</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linked Trips</td>
<td>Combining two or more trips into one to account for its true purpose. For example, if after leaving work, a person stopped at the video store and then continued home, it can be hypothesized that the real purpose of the trip was to go from work to home. Therefore, the trips would be &quot;linked&quot; together to form one Home Based Work trip.</td>
</tr>
<tr>
<td>Unlinked Trips</td>
<td>The true trips that occurred. In the above example, there was one Non Home Based unlinked trip (from work to the video store) and one Home Based Non Work unlinked trip (from the video store to home).</td>
</tr>
</tbody>
</table>

### TABLE 2. CITIES USED IN THE 1990 HOUSEHOLD TRAVEL SURVEY

<table>
<thead>
<tr>
<th>City</th>
<th>Population</th>
<th>Households Surveyed</th>
<th>Average Trip Length (min.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>HBW</td>
</tr>
<tr>
<td>Amarillo</td>
<td>187,547</td>
<td>2554</td>
<td>9.33</td>
</tr>
<tr>
<td>Brownsville</td>
<td>260,120</td>
<td>1367</td>
<td>6.05</td>
</tr>
<tr>
<td>San Antonio</td>
<td>1,185,394</td>
<td>2453</td>
<td>15.49</td>
</tr>
<tr>
<td>Sherman-Denison</td>
<td>95,021</td>
<td>2349</td>
<td>9.25</td>
</tr>
<tr>
<td>Tyler</td>
<td>151,309</td>
<td>2025</td>
<td>6.99</td>
</tr>
</tbody>
</table>

*Counts for entire counties in which these cities are located

*Unlinked trips, intrazonal trips NOT included
used was both linked and unlinked. In some places linked trips are better, in others unlinked are better. The differences seen are very small, the largest being only 0.0075. Similar results are seen in the average trip length error, the K-S value and the error in the left and right tails, for all three trip purposes. Sometimes linked worked better, sometimes unlinked, but not in any consistent manner (such as linked always working better with the OPM or unlinked always working better in one city), and never by a significant amount. (See Figure 1)

It was concluded, therefore, that there was no significant difference between using linked and unlinked trips when dealing with TLFD's. An analyst may use either one, depending on his/her preference or on other parts of the trip distribution analysis, which may be affected by whether the trips are linked or not. In this survey data, the total number of HBW trips increased and HBNW and NHB trips decreased by as much as 13 percent due to trip linking. This will probably have a significant effect on the trip distribution analysis, but determining such effects was not the aim of this research.

**ITLFDM vs. OPM**

The second objective of the research was to determine how well each model estimated the TLFD's of the survey data. The major statistical measures that are important in determining how well the models are working are the percent error in the average trip length, the R² value and the K-S Goodness of Fit Test. If the models do well in these areas, then they are doing at least a good job of estimating the true TLFD, if not excellent. Both models, in all five cities and for all three trip purposes, passed the K-S Goodness of Fit test at the 80 percent significance level. The R² value and the error in the average trip length were very good, as shown in Table 4. So, in general, the models are doing a very good job.

The other two measures that must be noted, the errors in the left and right tails, however, are not nearly as good. There are two reasons why these errors are so high. First, many of the results are much lower than the average shown, but one or two are much, much higher, bringing the average up. For example, in San Antonio HBW trips, the left tail error of the ITLFDM was 138.78 percent, while every other value was under 30 percent, and all but two under 15 percent. The fact that the mode (defined as the middle value when the values are placed in numerical order) of each is much lower than the average shows this.

Secondly, both tails represent a very small percentage of the overall distribution. This, in turn, has two implications. First, a very small difference in percent trips can lead to a very large percent error. For example: if the survey showed that one percent of the trips were in the right tail, and the model predicted two percent, a one percent discrepancy between the survey and model is produced, but a 100 percent error is also produced. Secondly, when dealing with small portions of the distribution, random fluctuations in the survey data tend to vastly increase the percent error. These errors, therefore, are not as bad as they seem, but improving the tails would still be beneficial.

Determining the specific accuracy of each model requires a closer examination than simply the average values. Table 5 shows the statistical results for Amarillo (Figure 2 shows HBNW trips graphically). Note that the average trip length error is no longer shown, since it did not show any difference between the two models.

At first glance, it appears the ITLFDM is the better model for Amarillo. Indeed, the ITLFDM has a better value in every category. However, most of the differences are very small. Similar results were found in Brownsville, Sherman-Denison and Tyler. Occasionally, in some categories, the OPM did better, but overall, the ITLFDM did a slightly better job on these cities.

Table 6 shows the statistical results for San Antonio (Figure 3 shows HNB trips graphically). This was, by far, the largest urban area in the survey. According to previous research (5), the ITLFDM should do much better in estimating this city’s tails. As Table 6 shows, this was not the case. In fact, the OPM was significantly better than the ITLFDM, although the ITLFDM’s results were certainly adequate. Table 7 shows an overview of these statistical findings for all five cities.

These results show that both models provide a very good to excellent estimation of the TLFD’s found in the 1990 travel surveys. Therefore, the analyst is recommended to continue using whichever model he/she is currently using. Any difference in the accuracy of the estimations of the two models was small compared to the overall accuracy achieved.

**Possible Improvements**

The previous results indicate that there is no pressing need to improve or adjust the models in any way. However, even though both models show good results, improvements would be beneficial. Recalibration of the models, using current data, may increase the accuracy of the models. However, both models are estimating just as well as when they were originally calibrated, and are exhibiting the same minor deficiencies. Therefore, recalibration may not help.
### TABLE 3. COEFFICIENT OF DETERMINATION, R², LINKED VS. UNLINKED

<table>
<thead>
<tr>
<th>City</th>
<th>Linked</th>
<th>Unlinked</th>
<th>Best Fit</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Amarillo</td>
<td>0.9765</td>
<td>0.9757</td>
<td>Linked</td>
<td>0.0008</td>
</tr>
<tr>
<td>ITLFDM</td>
<td>0.9621</td>
<td>0.9647</td>
<td>Unlinked</td>
<td>0.0026</td>
</tr>
<tr>
<td>OPM</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Brownsville</td>
<td>0.9745</td>
<td>0.9714</td>
<td>Linked</td>
<td>0.0031</td>
</tr>
<tr>
<td>ITLFDM</td>
<td>0.9755</td>
<td>0.9745</td>
<td>Linked</td>
<td>0.0010</td>
</tr>
<tr>
<td>OPM</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>San Antonio</td>
<td>0.9060</td>
<td>0.9135</td>
<td>Unlinked</td>
<td>0.0075</td>
</tr>
<tr>
<td>ITLFDM</td>
<td>0.9511</td>
<td>0.9451</td>
<td>Unlinked</td>
<td>0.0000</td>
</tr>
<tr>
<td>OPM</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sherman-Denison</td>
<td>0.9743</td>
<td>0.9784</td>
<td>Unlinked</td>
<td>0.0041</td>
</tr>
<tr>
<td>ITLFDM</td>
<td>0.9511</td>
<td>0.9563</td>
<td>Unlinked</td>
<td>0.0052</td>
</tr>
<tr>
<td>OPM</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tyler</td>
<td>0.9617</td>
<td>0.9643</td>
<td>Unlinked</td>
<td>0.0026</td>
</tr>
<tr>
<td>ITLFDM</td>
<td>0.9613</td>
<td>0.9641</td>
<td>Unlinked</td>
<td>0.0028</td>
</tr>
<tr>
<td>OPM</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*Home Based Work trips*

![Travel Time (minutes)](image)

**FIGURE 1. LINKED VS. UNLINKED, SAN ANTONIO TLFD, HOME BASED WORK**
### TABLE 4. STATISTICAL MEASURES, ITLFD & OPM COMBINED

<table>
<thead>
<tr>
<th></th>
<th>HBW</th>
<th>HBNW</th>
<th>NHB</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average R²</td>
<td>0.9608</td>
<td>0.9416</td>
<td>0.9624</td>
</tr>
<tr>
<td>Average Trip Length Error</td>
<td>0.45%</td>
<td>0.26%</td>
<td>0.30%</td>
</tr>
<tr>
<td><strong>Left Tail Error:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>20.60%</td>
<td>16.50%</td>
<td>20.11%</td>
</tr>
<tr>
<td>Mode</td>
<td>15.26%</td>
<td>12.63%</td>
<td>11.72%</td>
</tr>
<tr>
<td><strong>Right Tail Error:</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>30.94</td>
<td>32.65%</td>
<td>37.80%</td>
</tr>
<tr>
<td>Mode</td>
<td>19.40</td>
<td>25.68%</td>
<td>28.68%</td>
</tr>
</tbody>
</table>

### TABLE 5. STATISTICAL MEASURES, AMARILLO, ITLFD & OPM

<table>
<thead>
<tr>
<th></th>
<th>ITLFD</th>
<th>OPM</th>
<th>BEST MODEL</th>
<th>DIFFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>HBW</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>R²</td>
<td>0.9757</td>
<td>0.9647</td>
<td>ITLFD</td>
<td>0.0110</td>
</tr>
<tr>
<td>K-S Value</td>
<td>0.0190</td>
<td>0.0460</td>
<td>ITLFD</td>
<td>0.0270</td>
</tr>
<tr>
<td>Left Tail Error</td>
<td>8.29%</td>
<td>28.49%</td>
<td>ITLFD</td>
<td>20.02%</td>
</tr>
<tr>
<td>Right Tail Error</td>
<td>41.04%</td>
<td>56.62%</td>
<td>ITLFD</td>
<td>15.58%</td>
</tr>
<tr>
<td><strong>HBNW</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>R²</td>
<td>0.9125</td>
<td>0.8917</td>
<td>ITLFD</td>
<td>0.0208</td>
</tr>
<tr>
<td>K-S Value</td>
<td>0.0695</td>
<td>0.0798</td>
<td>ITLFD</td>
<td>0.0103</td>
</tr>
<tr>
<td>Left Tail Error</td>
<td>20.12%</td>
<td>23.23%</td>
<td>ITLFD</td>
<td>3.11%</td>
</tr>
<tr>
<td>Right Tail Error</td>
<td>62.90%</td>
<td>70.16%</td>
<td>ITLFD</td>
<td>7.26%</td>
</tr>
<tr>
<td><strong>NHB</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>R²</td>
<td>0.9890</td>
<td>0.9742</td>
<td>ITLFD</td>
<td>0.0148</td>
</tr>
<tr>
<td>K-S Value</td>
<td>0.0269</td>
<td>0.0431</td>
<td>ITLFD</td>
<td>0.0162</td>
</tr>
<tr>
<td>Left Tail Error</td>
<td>8.64%</td>
<td>13.85%</td>
<td>ITLFD</td>
<td>5.21%</td>
</tr>
<tr>
<td>Right Tail Error</td>
<td>35.40%</td>
<td>50.93%</td>
<td>ITLFD</td>
<td>15.53%</td>
</tr>
</tbody>
</table>

![Figure 2. Amarillo TLFD, Home Based Non Work](image)

FIGURE 2. AMARILLO TLFD, HOME BASED NON WORK
**TABLE 6. STATISTICAL MEASURES, SAN ANTONIO, ITLFDM VS. OPM**

<table>
<thead>
<tr>
<th></th>
<th>ITLFDM</th>
<th>OPM</th>
<th>BEST MODEL</th>
<th>DIFFERENCE</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>HBW</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( R^2 )</td>
<td>0.9135</td>
<td>0.9451</td>
<td>OPM</td>
<td>0.0316</td>
</tr>
<tr>
<td>K-S Value</td>
<td>0.0639</td>
<td>0.0364</td>
<td>OPM</td>
<td>0.0275</td>
</tr>
<tr>
<td>Left Tail Error</td>
<td>138.78%</td>
<td>30.80%</td>
<td>OPM</td>
<td>107.98%</td>
</tr>
<tr>
<td>Right Tail Error</td>
<td>96.46%</td>
<td>5.99%</td>
<td>OPM</td>
<td>90.47%</td>
</tr>
<tr>
<td><strong>HBNW</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( R^2 )</td>
<td>0.8948</td>
<td>0.8954</td>
<td>OPM</td>
<td>0.0006</td>
</tr>
<tr>
<td>K-S Value</td>
<td>0.0652</td>
<td>0.0850</td>
<td>ITLFDM</td>
<td>0.0198</td>
</tr>
<tr>
<td>Left Tail Error</td>
<td>63.55%</td>
<td>15.59%</td>
<td>OPM</td>
<td>47.96%</td>
</tr>
<tr>
<td>Right Tail Error</td>
<td>8.72%</td>
<td>58.14%</td>
<td>ITLFDM</td>
<td>49.42%</td>
</tr>
<tr>
<td><strong>NHB</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( R^2 )</td>
<td>0.8783</td>
<td>0.9628</td>
<td>OPM</td>
<td>0.0845</td>
</tr>
<tr>
<td>K-S Value</td>
<td>0.0885</td>
<td>0.0391</td>
<td>OPM</td>
<td>0.0494</td>
</tr>
<tr>
<td>Left Tail Error</td>
<td>110.90%</td>
<td>12.41%</td>
<td>OPM</td>
<td>98.49%</td>
</tr>
<tr>
<td>Right Tail Error</td>
<td>65.14%</td>
<td>17.43%</td>
<td>OPM</td>
<td>47.71%</td>
</tr>
</tbody>
</table>

**FIGURE 3. SAN ANTONIO TLFD, NON HOME BASED**
TABLE 7. STATISTICAL MEASURES, ITLFDM VS. OPM

<table>
<thead>
<tr>
<th></th>
<th>Amarillo</th>
<th>Browns.</th>
<th>San Ant.</th>
<th>Sher.-Den.</th>
<th>Tyler</th>
</tr>
</thead>
<tbody>
<tr>
<td>HBW</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>R²</td>
<td></td>
<td></td>
<td>OPM</td>
<td>ITLFDM</td>
<td>-</td>
</tr>
<tr>
<td>K-S Value</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Left Tail Error</td>
<td>ITLFDM</td>
<td></td>
<td>OPM</td>
<td>ITLFDM</td>
<td>-</td>
</tr>
<tr>
<td>Right Tail Error</td>
<td>ITLFDM</td>
<td></td>
<td>OPM</td>
<td>ITLFDM</td>
<td>-</td>
</tr>
<tr>
<td>HBNW</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>R²</td>
<td>ITLFDM</td>
<td></td>
<td></td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>K-S Value</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Left Tail Error</td>
<td></td>
<td></td>
<td>OPM</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>Right Tail Error</td>
<td></td>
<td></td>
<td>ITLFDM</td>
<td>ITLFDM</td>
<td>OPM</td>
</tr>
<tr>
<td>NHB</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>R²</td>
<td></td>
<td></td>
<td>OPM</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>K-S Value</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Left Tail Error</td>
<td></td>
<td></td>
<td>OPM</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>Right Tail Error</td>
<td>ITLFDM</td>
<td></td>
<td>ITLFDM</td>
<td>ITLFDM</td>
<td>OPM</td>
</tr>
</tbody>
</table>

*Model name in matrix indicates that it was significantly better in that category. - indicates the models were essentially equivalent

*Significant when absolute difference > 0.0200

*Significant when absolute difference > 0.0400

*Significant when difference in errors > 10.00%

A second improvement method would be to adjust the output of the models in some fashion to "fix" the tails. It was decided to attempt this method. After evaluating both models, the following strategy for finding an adjustment method was decided upon:

1. Find a significant correlation utilizing the left and right tails and some other available independent variable. From the tails, three measures could be used: the percentage determined from the survey (the "real" percent of trips in the tails), the absolute error in the models or the percent error in the models. Other available variables included: average trip length, maximum separation, number of zones, population, average household income and average income per person.

2. If a correlation was found, "fix" the tails and adjust the middle separations to compensate. For example, if a correlation indicated five percent of the trips should be in the left tail, and a model estimated three percent, two percent of the trips would be moved from the "middle" separations (between the two tails) to the left tail. This would significantly improve the tails, and, hopefully, not significantly decrease the overall accuracy of the model.

3. Statistically analyze this "Adjusted Mode" to determine if any improvements were actually made.

Correlations

Each of the tail measures were plotted against each of the other available variables. For the most part, no simple correlations were evident, and probably did not exist at all, except for the following two pairs:

Average Trip Length vs. Left Tail (from survey)

When these variables were plotted against each other, a clear decreasing function was found for all three trip purposes (see Figure 4 for HBW trips). However, this correlation posed several problems. First, it was not linear, but seemed to be a decaying function that would approach an asymptote. When a linear line was fitted to these points (HBNW and NHB look more linear than HBW, but also exhibit possible non-linearity), the results of the model usually were closer to the survey data than the line was. Adjusting the model based on this would not improve results. Secondly, this correlation was found very near the deadline of the research. Because of these two problems, any adjustments using this correlation fell beyond the scope of this research. It should also be noted that no correlation between the average trip length and the right tail was found.
Percent Error in Tails vs. Average Household Income

When the percent error in the right tail of the OPM was plotted against the average household income of each urban area, a clear decreasing function was seen (see Figure 5). Of course, with only five cities, this correlation is questionable at best, yet it can be rationalized as follows: in cities of lower income, the OPM will overestimate the percent of trips at longer separations, and in higher income cities, it will underestimate these same trips. This makes sense, especially in HDW trips. Lower income employment can probably be found closer to home, while a person would be willing to travel farther for a higher income job.

However, a similar decreasing function was also found when comparing the percent error in the left tail of the OPM against average household income. This can not be as readily rationalized, but was seen in all three trip purposes. Another indication that this correlation may not hold true for other cities is that it was only found in the OPM, not the ITLFDM. The ITLFDM did show a more linearly decreasing function when the city of San Antonio was left out. With the previous findings on San Antonio, it may make sense to do this, but only four data points are then left. Therefore, it was chosen not to do this.

Instead, it was decided to use the correlation in the OPM and adjust the model accordingly. A line was fitted to the points using linear least squares regression, for each trip purpose and each tail. The resulting equations are:

**HBW**
- Left Tail: \( E = -8.375I + 24.71 \)
- Right Tail: \( E = -34.78I + 171.29 \)

**HBNW**
- Left Tail: \( E = -11.11I + 51.11 \)
- Right Tail: \( E = -33.90I + 153.91 \)

**NHB**
- Left Tail: \( E = -17.50I + 95.25 \)
- Right Tail: \( E = -75.00I + 412.50 \)

The equations are in the form \( E = \text{percent error in tail} \) and \( I = \text{average household income} \).
Adjustments

After performing several adjustments, it was noted that the average trip length was affected (by as much as 10%) when all the separations between the tails were adjusted to compensate for the "fixing" of the tails. Therefore, only a smaller portion of that middle section was used (from four minutes to 35% of the maximum separation). The resulting adjustments continued to affect the average trip length, but to a lesser degree (only up to 5%).

Visually, the adjusted model does not look much different from the non-adjusted original (see Figure 6). Statistically, however, the adjustments are clearly evident. Table 8 shows the results for Amarillo. The adjusted output significantly improves the accuracy of the tails as well as improves the $R^2$ value. Unfortunately, the accuracy of the average trip length is decreased, significantly in the case of HBNW. Similar results were seen in all five cities, in all three trip purposes.

This adjusted output significantly increases the accuracy of the tails of the OPM, however, it often decreases the accuracy of the average trip length, it is based on the questionable correlation between percent error in the tails and average household income, and it does not work on the ITLFDM. The procedure for making the adjustments has proven effective, though, and if different correlations are found in the future, a similar procedure could be used to modify the output of the models.

FINDINGS

In conclusion, the main findings of this research are:

1. There is no significant difference between the TLFD's produced by using either linked or unlinked trips.

2. Both models' estimations of the TLFD's seen in the 1990 household travel survey were considered very good to excellent. Future research may determine whether there is an advantage in using one model over the other in specific instances.

3. There is no pressing need to modify the current models in any way. Either model can be used with equal confidence.

4. The procedure used to find a method of adjusting the output of the models proved effective. This procedure can be used with future survey data to find more appropriate adjustments.
FIGURE 6. OPM VS. ADJUSTED OPM, SHERMAN-DENISON, HOME BASED WORK

TABLE 8. STATISTICAL MEASURES, AMARILLO, OPM VS. ADJUSTED OPM

<table>
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<tr>
<th></th>
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<th>ADJUSTED</th>
<th>BEST MODEL</th>
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<tr>
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*aAverage trip length
5. The adjusted output obtained significantly improved the tails of the OPM, but often decreased the accuracy of the average trip length. If an analyst is sufficiently dissatisfied with the tails produced by his/her model, then this alternative could be applied. Future research would be helpful in three areas:
   a. Determining whether the correlation used in the adjustment process holds true on other cities.
   b. Evaluating the correlation between average trip length and percent trips in the Left Tail, and fitting an exponential or other decaying curve to this data.
   c. Creating an adjustment method that does not affect the average trip length.

REFERENCES

Lime Stabilization of Subbase

RONALD L. NOWLIN

The current Texas Flexible Pavement Design System (TFPS) does not allow designing with cement or lime stabilized bases and subbases. This is a problem because these stabilizers are used extensively in Texas as a result of the poor subgrade soils. A project (Study Number 0-1287) was developed to address the problem.

This study is a portion of project 1287. It deals with studying the structural stability that lime provides for subbases. The study involves backcalculating the layer moduli for twelve different pavement sections with lime stabilized subbases. This report shows that lime stabilization, if performed correctly, will improve structural stability in a pavement system.

INTRODUCTION

This report contains a study on the stiffness of lime stabilized subbases for twelve different highways around the state of Texas as shown in Table 1. All locations are Strategic Highway Research Program (SHRP) sites. Each pavement test section contains the following four layers: surface, base, lime treated subbase, and subgrade. Layer thicknesses, soil materials, and amounts of lime used is subbase stabilization vary among the sections and are shown in Table 1.

Chemical soil stabilization is often used in Texas to reduce shrink and swell potential in subgrade soils. Lime and Portland cement are the commonly used stabilization agents in Texas because of the limestone rich regions located within the state.

By adding a small percentage of lime to a soil, the resilient modulus, or stiffness, of the soil can be increased through a cementitious reaction between clay minerals and the lime. Increasing the stiffness of the subbase course can increase the structural stability of the pavement system. "Layered elastic analysis demonstrates that stress and strain distribution within the layered pavement system is governed (primarily) by modular ratios between the various layers" (1). By increasing the subbase stiffness, the surface to subbase modular ratio can be decreased. This will increase resistance to surface rutting, alligator cracking, and deflections in the pavement layers.

The in situ resilient moduli of pavement layers can be calculated from the deflection data obtained with a Falling Weight Deflectometer (FWD). The FWD is a nondestructive testing device that simulates the deflection produced by a moving truck wheel. A range of loads can be applied to the pavement and several sensors then measure the deflection of the pavement at selected distances. From this deflection data, the layer moduli can be backcalculated using an elastic layer based program (1).

PROBLEM STATEMENT

This study addresses lime stabilization of pavement subbase layers. It is part of a larger project entitled "Identify Structural Benefits of Stabilization and Updating FPS to Accommodate Stabilized Layers" (Study Number 0-1287) being conducted for the Texas Department of Transportation (TxDOT).

FPS or TFPS (Texas Flexible Pavement Design System) is a program developed for TxDOT for characterizing and designing pavement layers. The program is based on elastic layered analysis. It requires material properties and stiffness values to characterize pavement layers. The current TFPS system does not allow designing with stabilized bases and subbases. These stabilizers are used extensively in Texas because of the poor subgrade soils and lack of aggregates in Eastern Texas. To address this problem, the TFPS program is being modified to account for the structural contribution of the stabilized pavement layers. Study 1287 was developed to modify the TFPS program to include lime, Portland cement and fly ash stabilized layers (D.N. Little, unpublished data).

OBJECTIVES

This study was conducted to determine the impact of lime stabilization on subbase stiffnesses for several
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pavement structures throughout the state of Texas. The results will be used to help modify the current TFPS program to include stabilized pavement layers.

BACKGROUND

Soil-Lime Reactions

When lime (CaO or Ca(OH)₂) is added to a soil with an appreciable amount of clay minerals in the presence of water, several reactions may take place. These reactions include cation exchange, flocculation/agglomeration, pozzolanic reaction, and carbonation (2).

Clay minerals tend to adsorb cations (like sodium) that attract water. This increases the double water layer between the soil particles causing dispersion of the particles and swelling of the soil. Lime provides a source of free calcium which replaces these cations through cation exchange decreasing the double water layer. This also promotes flocculation/agglomeration which causes the soil particles to clump together into larger particles (2).

A pozzolanic reaction begins when lime is added to a soil creating a high pH environment (approximately 12.4). This greatly increases the solubilities of silica and alumina. The pozzolans (SiO₂ and Al₂O₃) react with the free calcium from lime, in the presence of water, to form a cementitious material. This reaction, which binds the soil particles together, can increase the soil shear strength, resilient modulus, and flexural modulus, while lowering the plasticity (1).

Lime works best with clay-like soils that provide natural pozzolans. Other soils, such as sands and silts, do not provide the pozzolans. Therefore, adding lime to these soils will not produce a pozzolanic reaction.

Carbonation occurs when lime reacts with carbon dioxide to form a carbonate. This inhibits the pozzolanic reaction by using up the free calcium. Therefore, this reaction should be avoided. (2).

Measuring Resilient Modulus

The resilient modulus of a pavement layer can be determined in two ways. The first is performed in the laboratory. It involves preparing a sample to simulate field conditions, applying a repetitive load, and measuring the resilient modulus. The second method involves in situ testing with an FWD (Falling Weight Deflectometer.) This is believed to be more representative than laboratory testing because the pavement section tested is undisturbed (3). An FWD is a nondestructive, in situ deflection measuring device. It allows a range of loads to be dropped on a pavement and measures the deflection of the pavement at several locations. This process may be carried out several different times along a pavement section, changing the location or load each time. A series of deflection bowls is collected. This data can be entered into a backcalculation program and the moduli values for each pavement layer calculated.

MODULUS Program

MODULUS is an elastic layer based program that backcalculates stiffness values of pavement layers from FWD deflection data. The program was developed by the Texas Transportation Institute (TTI) for TxDOT (4).

The deflection data, layer thicknesses, assumed Poisson ratios and layer moduli ranges are entered into the MODULUS program. Poisson ratios are assumed constant and remain the same throughout execution of the program. An initial assumption of moduli ranges for each pavement layer is made, and a seed modulus is assumed for the subgrade. However, these values may change if reasonable results are not obtained.

The MODULUS program then carries out a series of operations. First, the entered moduli ranges are used to calculate a set of deflections for each layer which are stored in a data base. Then the program searches the data base and tries to match the measured deflections with the calculated deflections stored in the data base. It finds the closest match, and the moduli values corresponding to the calculated deflections are taken as the solution. Absolute percent errors between the calculated deflections and the actual deflections are computed by the program. These percent errors should be kept as small as possible (4).

The MODULUS program also determines the depth to bedrock, or an apparent rigid layer, at each deflection bowl. Before this operation was included, backcalculation procedures assumed the subgrade thickness to be uniformly stiff and infinitely thick. In general, the subgrade stiffness varies with depth, and the thickness is finite. If a rigid layer exists at a shallow depth and is ignored, the stiffness of the subgrade may be overpredicted. This could result in an unconservative pavement design since the pavement thickness is dependent of the stiffness of supporting subgrade. Adding this procedure to the program improves the backcalculated layer moduli (3).

If the results seem unreasonable for any reason, then several changes to the original entered data can be made and the program run again. This process may be repeated a number of times. If reasonable results are not obtained, then this data set is removed from further analysis. The reasonable results are collected and average layer moduli for the pavement can be acquired.
STUDY DESIGN

Data Collection

Pavement deflection data were collected with Falling Weight Deflectometers on several SHRP sites in the state of Texas. Deflection data sets (twenty-one for each test site) were collected in both the wheel path and the center of the driving lane at each site. The data for pavements with lime stabilized subbases were used for this study. The following information was also collected from each SHRP site: pavement layer thicknesses, material types, percent lime used in stabilizing the subbase, location of SHRP sites, and pavement surface conditions. Lab tests (liquid limit, plastic limit, density, and moisture content) were conducted on the stabilized pavement layers for some of the sites.

Enter Data Into MODULUS

After the deflection and pavement layer data for each site were collected, they were entered into the MODULUS program. First, the deflection data from each test set (one from wheel path and one from center of driving lane) for each site were entered. Next, the pavement layer thicknesses, assumed Poisson ratios, layer moduli ranges, and subgrade seed modulus were entered.

Run MODULUS Program

When all of the pavement data had been entered, the MODULUS program was executed. After the program had finished running, the results were printed.

Analyze Results

Once the results were obtained, they were analyzed and classified as either reasonable or unreasonable.

The results were reasonable if the following criteria were met:

1. the depth to apparent rigid layer for each deflection data bowl was reasonably close (usually within 50 inches);
2. the absolute percent error per sensor was small (less than three percent was deemed acceptable);
3. the base modulus was greater than the subgrade modulus;
4. the moduli of the surface, base, and subbase were within the selected ranges; and
5. the subgrade modulus was close to the subgrade seed modulus.

If each of these requirements were met, the results were classified as reasonable and an average modulus was calculated for each pavement layer. If any of the requirements were not met, the results were classified as unreasonable.

When unreasonable results were obtained, the following changes were made:

1. if the depths to apparent rigid layer for each deflection bowl were not reasonably close, then they were divided into groups with ranges of around 50 inches;
2. if the calculated moduli for the surface, base, or subbase equaled either limit at the selected range, then that range was changed;
3. if the subgrade modulus was not close to the subgrade seed modulus, the seed modulus was set equal to the calculated subgrade modulus.

After these changes were made the modulus program was run again. This process was repeated until reasonable results were obtained. If reasonable results could not be acquired, then that particular deflection set was discarded from further analysis.

Conclusions

After the final results were obtained, they were analyzed and compared to the SHRP data collected to determine if they were acceptable. Conclusions were then made about the lime stabilized subbases at each site.

Documentation and Reporting

The last step in the study was to document the research conducted and report the final results and conclusions about the lime stabilized subbases.

RESULTS

Table 1 shows the percent lime used in stabilizing each of the 12 SHRP sections. Most of the sections were stabilized with 3 to 4 percent lime. The percent lime used for section 5 was not available.

FWD deflection data were collected at each SHRP site, every 25 feet over a 500 foot section for a total of 21 FWD tests at each pavement section. The deflection data were taken from both the wheel path and the center of the lane at each site. These data were used to backcalculate layer moduli.

The ranges for moduli of average granular bases is usually from about 15 kips per square inch to a little over 100 kips per square inch (5). Figure 1 shows the moduli values backcalculated for the base, subbase and subgrade layers for the wheel paths of the SHRP sites. The
backcalculated layer moduli from the center of the lane is shown in Figure 2. The range of moduli for the lime treated subbase is less than 20 to approximately 1000 kips per square inch for both the wheel path and center of lane. Some of the values are questionable, such as the subbase modulus for section 6 which is close to 1000 kips per square inch for both locations.

There is also a large variation in the backcalculated subbase moduli from the two locations, especially for section 8. It was first hypothesized that the moduli for the wheel path would be lower than that for the center of the lane because the wheel path would receive higher stresses and therefore undergo more deterioration. Figures 1 and 2 demonstrate that this was not always the case. The backcalculated subbase moduli for the wheel path was higher than the moduli for the center of the lane in seven of the twelve sections.

Figures 3 and 4 show the backcalculated moduli and standard deviations of the lime treated subbases for the wheel path and center of lane, respectively. The standard deviations were very high, compared to the average values, for many of the sections such as sections four and twelve for the center of lane.

Because of these unreasonable results, it was concluded that the backcalculated moduli for the lime treated subbase may not have been accurate. Another reason for this conclusion was due to the inconsistencies in the subbase moduli backcalculated from the MODULUS program. When the set layer moduli ranges were changed during the execution of the program, the backcalculated subbase modulus increased or decreased dramatically, in most cases.

Therefore, the program was run until the absolute percent errors between the calculated deflections and the actual deflections were at a minimum. The resulting absolute percent errors are presented in Table 2. In most cases, the percent error was below three percent. This was considered very reasonable.

There are several other factors that could have contributed to the variation between the lime treated subbase moduli of the 12 sections and the large standard deviation within each section. They include the following: alligator cracking in the pavement surface which could have led to inaccurate deflection test results; uneven distribution of the lime in the subbase layer; inadequate mixing of the lime and soil; and the type of soil that was lime stabilized, along with others.

Only clay-like soils provide the pozzolans needed for a pozzolanic reaction to occur between the soil and the lime. Therefore the less clay is soil contains, the less effect lime stabilization will have. This can be observed by comparing section 10, which had a natural subgrade of sand and a backcalculated modulus less than 20 kips per square inch in the wheel path, to any of the other sections which had more clay-like subgrades and much higher moduli.

The plasticity index was obtained for some of the SHRP sites. A percent increase of modulus between the natural subgrade and the lime treated subgrade was calculated by assuming that the backcalculated natural subgrade moduli equals the subbase moduli values without stabilization.

By plotting the percent increase of modulus for the wheel path versus the plasticity index as seen in Figure 5, it is observed that as the plasticity index increases, so does the modulus. This seemed reasonable because the higher the plasticity of a soil, the greater the pozzolanic reaction and therefore the greater the increased stiffness in the soil.

Even though the backcalculated moduli for the lime treated subbase may not have entirely been accurate, in most cases, the subbase moduli usually showed a considerable increase over the moduli of the natural subgrades. Therefore, it was concluded that lime stabilization, if performed correctly, does provide an increased stiffness for a pavement system.

REFERENCES


FIGURE 1. AMOUNT OF LIME USED IN SOIL STABILIZATION

FIGURE 2. BACKCALCULATED MODULI FOR WHEEL PATH
FIGURE 3. BACKCALCULATED MODULI FOR CENTER OF LANE

FIGURE 4. SUBBASE MODULI AND STANDARD DEVIATION FOR WHEEL PATH
TABLE 2. AVERAGE ABSOLUTE PERCENT ERROR PER SENSOR

<table>
<thead>
<tr>
<th>Section</th>
<th>Wheel Path</th>
<th>Center of Lane</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.19</td>
<td>1.34</td>
</tr>
<tr>
<td>2</td>
<td>2.56</td>
<td>1.88</td>
</tr>
<tr>
<td>3</td>
<td>2.26</td>
<td>0.65</td>
</tr>
<tr>
<td>4</td>
<td>1.76</td>
<td>3.19</td>
</tr>
<tr>
<td>5</td>
<td>1.12</td>
<td>1.31</td>
</tr>
<tr>
<td>6</td>
<td>0.81</td>
<td>1.08</td>
</tr>
<tr>
<td>7</td>
<td>0.65</td>
<td>0.66</td>
</tr>
<tr>
<td>8</td>
<td>2.02</td>
<td>2.56</td>
</tr>
<tr>
<td>9</td>
<td>1.25</td>
<td>1.01</td>
</tr>
<tr>
<td>10</td>
<td>2.07</td>
<td>1.18</td>
</tr>
<tr>
<td>11</td>
<td>0.81</td>
<td>0.97</td>
</tr>
<tr>
<td>12</td>
<td>1.26</td>
<td>1.38</td>
</tr>
</tbody>
</table>

FIGURE 5. SUBBASE MODULI AND STANDARD DEVIATION FOR CENTER OF LANE
FIGURE 6. PLASTICITY VERSUS PERCENT INCREASE OF MODULUS OF WHEEL PATH
Determination of Driver Capability in the Detection and Recognition of an Object

DALE L. PICHIA

This study determines the capability of a driver in a vehicle to detect and recognize various size objects that are intentionally placed in the roadway. The study was conducted to meet concerns about the visual capabilities of drivers. A driver must have adequate reaction and braking time to stop the vehicle in a hazardous situation, and this depends largely on the visual capabilities of the driver. This study uses six different objects of varying contrasts to determine if adequate visual capabilities coincide with actual reaction and braking distances necessary to completely stop the vehicle. The results of the study indicate that drivers are able to detect a high-contrast, 6-inch object and still have adequate stopping sight distance (SSD). A low-contrast, 6-inch object was not detectable and neither of the objects were recognizable above the minimum SSD. It is recommended that future studies be conducted to determine detection and recognition distances at night, in different weather conditions, and incorporating a vertical and/or horizontal curve into the study.

INTRODUCTION

The visual requirement of a driver is to see the road ahead, and more importantly objects in the road ahead. This is one of the most important aspects in roadway design. This visual requirement is referred to as sight distance, or the length of roadway ahead visible to the driver (7).

There are three types of sight distances used in roadway design: 1) stopping sight distance; 2) passing sight distance; and 3) decision sight distance. Stopping sight distance (SSD) is the concern of this study simply because these are the minimum design values. It is defined as the "minimum sight distance required for driver to stop a vehicle after seeing an object in the vehicle's path without hitting that object" (2). These minimum values should provide an adequate distance to allow a below average driver travelling near the design speed to react and stop a vehicle in time before reaching a stationary object in the road.

The American Association of State Highway Officials (AASHO) established a SSD model in 1940 that is still in use today for design purposes. The SSD model methodology is based on many factors, namely velocity of the vehicle, tire-to-pavement friction, driver eye-height above the surface of the road, object height above the surface of the road, perception-reaction time (PRT), and the algebraic difference in grades (1,3). Extensive research through the years has recommended values for each of these factors based on human, vehicle, highway, and environmental considerations (4). The values, however, remain questionable, namely the object height, even though it is one of the least sensitive factors in the SSD model (4).

PROBLEM STATEMENT

The object height has had various interpretations through the years. The height was initially the same as the driver eye height at 5.5 feet in 1921 (5). It was reduced considerably to 4 inches in the 1940 AASHO stopping sight distance model, and was redefined again in 1965 to the present standard of 6 inches (6). Although it would be ideal to have a 0 (zero) inch object so the driver could see every point on the road, construction costs can be considerably decreased if a 6-inch object height is used for design. A reasonable compromise between cost and driver visibility is 6 inches (7). This height was rationally chosen because it is the smallest object that can create a hazardous situation (i.e., sudden change in speed and/or direction) but can still be perceived as hazardous by the driver in time to stop before reaching it (7).

The problem remains, however, whether or not a driver can actually detect and recognize a 6-inch object within the required stopping sight distance provided by the American Association of State Highway and Transportation Officials (AASHTO) design policy. While
it may not be necessary to fully recognize an object in the vehicle's path, the driver must detect the object and recognize that it does create a hazard. According to the AASHTO’s *A Policy on Geometric Design of Highways and Streets*, a driver travelling between approximate speeds of 48 MPH to 55 mph requires 432 to 538 feet (rounded to 450 to 550 feet for design purposes) of visible roadway to stop the vehicle for an object in its path ($I$). This is applicable on all highway classifications, based on a tire-to-pavement friction value of 0.30 and a PRT value of 2.5 seconds. These values apply to a wet pavement and a below average vehicle or operator ($I$).

**RESEARCH OBJECTIVE**

The objective of this research was to determine, with a controlled test, if drivers can actually detect and recognize objects within the minimum stopping sight distance provided by AASHTO. Six-inch objects with different color contrasts were a particular item of interest. The velocity of the vehicle was not an important parameter, but rather the distances at which objects were detected and recognized. The results from the study were compared to values based on the current AASHTO Stopping Sight Distance model.

**STUDY DESIGN**

The drivers’ responses were quantitatively analyzed to meet the objectives of this study. This study measured the ability of the driver to detect and recognize objects in the road ahead. Included in the study design are the objects used in the study, the measure of driver performance used, a controlled testing procedure, a layout of the course, and the analysis method that was used.

**Objects To Be Used**

The objects that were used in this study are as follows:

1) 2 ~ 2"x4" wooden boards
2) a 6-inch black dog
3) a 6-inch white dog
4) an 8-inch tire tread
5) a 12-inch tree limb
6) an 18-inch bale of hay

The objects and their color and contrast relative to a concrete pavement are listed in Table 1. These objects represent some of the more common objects found on the road today (8). This was determined in Kahl’s research by a study of accident reports and accident databases in two regions of the United States which determine objects that are probable causes of accidents on various highway classifications.

All objects used remained stationary in the roadway during the study. A particular item of study are the two 6-inch objects of different contrasts. The 6-inch, stationary object is of interest because this is the criterion used in SSD calculations. The objects were evaluated on their ability to be detected and recognized.

**Measure Of Driver Performance**

With the aid of a distance-measuring instrument (DMI), a device installed in the vehicle, distance, time, and velocity were observed at points of object detection and object recognition. With this information the distance to the object can be calculated to determine if this object is detected and recognized within the minimum sight distance values. The subject’s braking and steering behaviors were also observed for each object. This observation showed whether or not the subject showed extreme caution to a particular object; namely to show a contrast between representative animate objects and inanimate objects, size of the objects, and the perceived hazard of the objects.

**Controlled Testing Procedure**

**Subject Pool**

From a desired subject pool of previous study participants, 45 licensed drivers, 15 in each of three different age groups ( <25, 25-55, and >55 ), were contacted and scheduled to drive the course. Each subject was asked personal questions, including their age, the average number of miles they drive in a year, how long they have been legally driving a vehicle, whether or not they had a corrective lens restriction on their driver’s license, and if so, if they were wearing their corrective lenses. This would ensure that all subjects met the legal requirement of a least 20/40 static visual acuity, which is required in Texas. Demographic information also included gender and ethnic background.

**Instruction to the Subject**

The instructions given to the subject must be clearly understood. Each subject, prior to beginning the course, was instructed to get comfortable with the Texas Transportation Institute vehicle, a 1991 Ford Crown Victoria with an automatic transmission.

By familiarizing the subject with the mirrors, brakes, air-conditioning, seating, and the overall experience of driving an unfamiliar vehicle, he/she would be more relaxed and feel the vehicle was a personal vehicle being driven.
TABLE 1 OBJECTS USED IN STUDY

<table>
<thead>
<tr>
<th>Object</th>
<th>Height (inches)</th>
<th>Color</th>
<th>Contrast</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 X 4 Wood</td>
<td>4</td>
<td>Lt. Brown</td>
<td>Low</td>
</tr>
<tr>
<td>Dog</td>
<td>6</td>
<td>Black</td>
<td>High</td>
</tr>
<tr>
<td>Dog</td>
<td>6</td>
<td>White</td>
<td>Low</td>
</tr>
<tr>
<td>Tire Tread</td>
<td>8</td>
<td>Black</td>
<td>High</td>
</tr>
<tr>
<td>Tree Limb</td>
<td>12</td>
<td>Green</td>
<td>Low</td>
</tr>
<tr>
<td>Bale of Hay</td>
<td>18</td>
<td>Lt. Brown</td>
<td>Low</td>
</tr>
</tbody>
</table>

The subject was then briefed on what the study entailed and why the study was being conducted. Each subject was given the following instructions: "the course is setup to represent a two-lane rural highway, follow the signs as you normally would on this type of road, you will be the only driver on the course at the time of the testing, and the objects will be placed in a location that will not create a hazard for your driving." Each subject was also asked to drive in a safe and usual manner and even though a 55 MPH speed limit was posted, the subject was asked to drive at the speed he/she felt comfortable driving.

Subject Participation

Subject participation was very important for this study. Once instructed, the subject began driving the course. Each subject was asked to indicate when an object was first detectable by immediately saying "NOW" or in some way indicating that there was something in the road that should not be there. After detecting the object, or if the object was not detected at all, the subject was to then identify the object by verbally indicating what the object is, when it was recognizable.

Each subject drove through the course seven times with an observer in the car. While driving through the course each time, one of the objects would be introduced. This was accomplished by an assistant who would place the object at a predetermined station before the subject would begin the course each time. As the object was passed by the subject in the vehicle, the assistant remained out of view.

Distance, time, and speed were measured with the DMI by the observer at points of detection and recognition for each object. Distance and time were measured relative to a beginning reference point on the course. Also, even though the subject was instructed that the object would not create a hazard, any change in speed or direction of the vehicle was noted when an object was encountered.

Object Use and Placement

There were four different random orders that the objects would appear in, and are represented in this study as Group 1, Group 2, Group 3, and Group 4. Each subject was placed into one of these four groups. The groups and their order of object appearance is shown in Table 2. The objects also changed position and location relative to the center-line stripe and the solid-white stripe on the right side shoulder. The objects' locations were at fixed stations so determining detection and recognition distances would be possible. Four consecutive subjects not only saw a different order of objects but different locations at which the objects were encountered. The only objects that remained in the same location for every subject was the tire tread and the bale of hay. These two objects were relatively large and heavy and too difficult to relocate to another station.

Follow Up Questions

After driving through the course seven times, the subject was asked additional questions by the observer in the vehicle concerning objects encountered in the road. Each subject was asked if a particular object had ever been encountered while driving a vehicle that created a hazardous situation. A hazardous situation was explained as a "sudden change in speed or direction of the vehicle being operated." If such a situation had occurred, the subject was asked what that object was. Each subject was also asked what the smallest object that he/she considered to be hazardous.

After the observer recorded the verbal responses, the subject was then shown a series of photographs taken on a two-lane, rural highway of objects in the road. Table 3 lists these objects used in the photographs. Based on
TABLE 2 ORDER OF OBJECT APPEARANCE

<table>
<thead>
<tr>
<th>Test Run</th>
<th>Group 1</th>
<th>Group 2</th>
<th>Group 3</th>
<th>Group 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>2 ~ 1x4’s</td>
<td>Bale of Hay</td>
<td>Tree Limb</td>
<td>Tire Tread</td>
</tr>
<tr>
<td>2</td>
<td>Black Lab</td>
<td>2 ~ 1x4’s</td>
<td>Nothing</td>
<td>White Poodle</td>
</tr>
<tr>
<td>3</td>
<td>Tire Tread</td>
<td>Black Lab</td>
<td>Bale of Hay</td>
<td>Tree Limb</td>
</tr>
<tr>
<td>4</td>
<td>White Poodle</td>
<td>Tire Tread</td>
<td>2 ~ 1x4’s</td>
<td>2 ~ 1x4’s</td>
</tr>
<tr>
<td>5</td>
<td>Tree Limb</td>
<td>White Poodle</td>
<td>Tire Tread</td>
<td>Bale of Hay</td>
</tr>
<tr>
<td>6</td>
<td>Nothing</td>
<td>Tree Limb</td>
<td>Black Lab</td>
<td>Nothing</td>
</tr>
<tr>
<td>7</td>
<td>Bale of Hay</td>
<td>Nothing</td>
<td>White Poodle</td>
<td>Black Lab</td>
</tr>
</tbody>
</table>

The environment and how the object looked in the photograph, the subject was asked to rate each object as either a low, moderate, or extreme hazard. A low hazard is explained as "being able to pass over or around the object in the vehicle with very little braking or steering"; a moderate hazard is explained as "being able to pass over or around the object with some degree of difficulty"; and an extreme hazard is explained as "a sudden and drastic change in speed and direction of the vehicle." Each subject was then instructed that the testing was complete. Any questions asked were answered and any comments made pertaining to the study were recorded.

The study was conducted only during the daytime and only between the hours of 10:00 A.M. and 5:00 P.M. in order to achieve maximum sunlight conditions. Dry pavement conditions were also a requirement. There were no horizontal or vertical curves involved with the detection and recognition of the objects. Each object was placed only on the tangent section of the course. Weather conditions were noted for each day, including temperature, humidity, and cloud conditions.

These conditions eliminated some of the variables in the study, such as horizontal or vertical alignment, a wet pavement, and night-vision requirements. The conditions presented here are ideal and represent a best-case scenario. Any conditions other than these would only result in shorter distances of object detection and recognition.

**Physical Layout Of The Course**

The entire length of the concrete pavement course is approximately 5000 feet in length. The course, representative of a two-lane rural highway, begins with the longest tangent running north to south being 3000 feet in length, followed by a horizontal curve to the right and another tangent section about 1100 feet in length. This is followed by another horizontal curve to the right and a 500 foot tangent section. A layout of the course is shown in Figure 1. This sight was chosen because adequate distance was provided on the 3000 foot straight-away to introduce objects to the subject. This distance allowed the reduction of driver workload which is required on horizontal curves and allowed the subject to reach a constant speed in the vehicle on the straight-away before the object could be detected.

To simulate a two-lane, rural highway, six-inch yellow and white lines were painted and 4 standard signs were installed on the course, including two Speed Limit 55 signs (R2-1) and two Yellow Curve Warning signs (W1-2R) with 35 MPH Advisory Speed Plates (W13-1). Pavement markings and sign use and placement meet specifications stated in the Manual on Uniform Traffic Control Devices (MUTCD) (9). Sign use remained the same for all subjects, regardless of what group number the subject was in.

The beginning of the course is at the northern most segment of the course. For odd-numbered test runs (1,3,5,7) the subject drove the course in the south-bound lane. The subject would turn around after each of these test runs at the end of the course and drive in the north-bound lane for the even-numbered test runs (2,4,6). Once to the beginning, the subject would again turn around and continue in the south-bound lane again.

**Analysis Method**

The analyses of the data for this paper include distances of detection and recognition for each object and an analysis of the hazard rating for each object shown to the driver in the photographs. Demographic and personal data was also analyzed to determine a difference or correlation in age and driving experience between the drivers.
### TABLE 3 OBJECTS USED IN PHOTOGRAPHS

<table>
<thead>
<tr>
<th>Photograph</th>
<th>Object</th>
<th>Height (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Armadillo</td>
<td>6</td>
</tr>
<tr>
<td>2</td>
<td>2 ~ 1 x 4's</td>
<td>4</td>
</tr>
<tr>
<td>3</td>
<td>2 ~ 1 x 6's</td>
<td>2</td>
</tr>
<tr>
<td>4</td>
<td>Wooden Pallet</td>
<td>5</td>
</tr>
<tr>
<td>5</td>
<td>Tire Tread</td>
<td>8</td>
</tr>
<tr>
<td>6</td>
<td>Tree Limb</td>
<td>12</td>
</tr>
<tr>
<td>7</td>
<td>Bale of Hay</td>
<td>18</td>
</tr>
</tbody>
</table>

### FIGURE 1. COURSE LAYOUT
Distances

The distance of detection and recognition for each object was recorded with the DMI by the test administrator in the car. With each object at a known station, the distance is subtracted from the known station to obtain the detection and recognition distances. The driver must be able to detect an object before determining whether it creates a hazard or not, but does not have to recognize it, only recognize the fact that the particular object creates a hazard. Although not accurate, a mean value between detection and recognition could represent stopping sight distance. The analysis of the data will determine a range of distances between detection and recognition for different size objects and determine if it is adequate stopping sight distance for the driver.

Hazard Rating

An analysis of the driver response to the photographs of the objects shown was also conducted. Objects with different size, depth, and contrast, as well as animate and inanimate objects, were evaluated. Each object was classified by the driver as either a low, moderate, or extreme hazard. With this information, correlations can be shown between a particular object and its degree of hazard. Three of the objects, photographs 2, 3, and 4, are of different sizes of wooden objects in the road. The 1x4's are small, the 1x5's are small but span the entire lane, and the pallet not only has a height dimension but a depth dimension. The study analyzed the difference in responses to these three objects in particular.

Personal and Demographic Data

The personal and demographic data analyzed for this study include age and driving experience. The subjects were divided into three age groups to account for the differences in driving experience and visual ability. Younger (less than 25) drivers lack driving experience, average age drivers (25 to 55) have experience and good physical abilities, and older (greater than 55) drivers have much experience but might lack physical abilities, namely good eyesight. Gender and ethnic background were not controlling variables in this study.

RESULTS

The data that was primarily addressed in this study was the distances of detection and recognition for each object, the hazard rating for the objects shown in the photographs, and a breakdown of the demographic data. The field data is reduced and the distances of detection and recognition and the hazard ratings are calculated.

Distances

Results of drivers in all three age groups are presented in Table 4 and in Figures 2-7. Represented in Table 4 are the 15th and 50th percentile values of detection and recognition distances for the six objects studied. The 15th percentile values for detection and recognition of the six-inch black dog was 593 feet and 18 feet, respectively, and the 15th percentile values for detection and recognition of the six-inch white dog was 231 feet and 2 feet, respectively. In Figures 2-7, a cumulative distribution illustrates the difference between detection and recognition of each object.

The Green Book states that an object must be perceived as a hazard before reacting to it. This would indicate that stopping sight distance is somewhere in the range between detection and recognition. The driver definitely has to detect an object but does not necessarily have to recognize it to make a decision to stop. The 15th percentile driver detected the objects in this study, except for the four-inch wooden object and the six-inch white dog, at or above 450 feet. This distance is the minimum or optimum, required stopping sight distance for a vehicle travelling between 48 to 55 miles per hour and also a rounded design value designated in the Green Book. The values for the six-inch objects differ because of contrast. The black dog was a high contrast object on the concrete pavement and was more easily detectable than a low-contrast white object. Both 15th Percentile values, however, for recognition were nearly identical for the 6-inch objects: 18 feet for the black dog and 2 feet for the white dog.

The values obtained in this study can be compared to rounded design values that are used in different countries (10, 11) based on optimum and desired speed ranges designated in the Green Book. These design values are presented in Table 5, listing values for six different countries: Australia, France, Germany, Switzerland, Sweden, and the United States. The difference in the values are due to varying factors such as perception-reaction time and pavement friction factors, which varies in each country’s SSD model based on speed of the vehicle. The four-inch wooden object and the six-inch, low contrast object again fall below each country’s optimum stopping sight distance. Even Australia and France, which have the two lowest values at 330 feet, are still 100 feet greater than the value for the low contrast, six-inch object.

For the photographs shown to the driver, mean responses are depicted in Figure 8. These responses are similar to what was obtained in a similar study (12) which
### TABLE 4 DISTANCES OF DETECTION AND RECOGNITION*

<table>
<thead>
<tr>
<th>Object</th>
<th>15&lt;sup&gt;th&lt;/sup&gt; Percentile</th>
<th>50&lt;sup&gt;th&lt;/sup&gt; Percentile</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Detect</td>
<td>Recognize</td>
</tr>
<tr>
<td>2 ~ 1x4's</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Black Lab</td>
<td>593</td>
<td>18</td>
</tr>
<tr>
<td>White Poodle</td>
<td>231</td>
<td>2</td>
</tr>
<tr>
<td>Tire Tread</td>
<td>903</td>
<td>132</td>
</tr>
<tr>
<td>Tree Limb</td>
<td>503</td>
<td>72</td>
</tr>
<tr>
<td>Bale of Hay</td>
<td>834</td>
<td>134</td>
</tr>
</tbody>
</table>

*All distances given in feet.

---

**FIGURE 2. DETECTION AND RECOGNITION DISTANCES FOR 4-INCH WOODEN BOARDS**
FIGURE 3. DETECTION AND RECOGNITION DISTANCES FOR 18-INCH BALE OF HAY

FIGURE 4. DETECTION AND RECOGNITION DISTANCES FOR 6-INCH BLACK LAB
FIGURE 5. DETECTION AND RECOGNITION DISTANCES FOR 6-INCH WHITE DOG

FIGURE 6. DETECTION AND RECOGNITION DISTANCES FOR 8-INCH TIRE TREAD
**FIGURE 7. DETECTION AND RECOGNITION DISTANCES FOR 12-INCH TREE LIMB**

**TABLE 5 REQUIRED STOPPING SIGHT DISTANCES IN DIFFERENT COUNTRIES**

<table>
<thead>
<tr>
<th>Country</th>
<th>Required SSD Speed = 48-55 MPH</th>
</tr>
</thead>
<tbody>
<tr>
<td>Australia</td>
<td>330 - 410</td>
</tr>
<tr>
<td>France</td>
<td>330 - 420</td>
</tr>
<tr>
<td>Germany</td>
<td>360 - 490</td>
</tr>
<tr>
<td>Switzerland</td>
<td>370 - 490</td>
</tr>
<tr>
<td>Sweden</td>
<td>440 - 530</td>
</tr>
<tr>
<td>United States</td>
<td>450 - 550</td>
</tr>
</tbody>
</table>

*All distances given in feet (10.11).*
showed a general increase in hazard rating as a function of object size. These objects, however, were not evaluated solely on size, but on appearance, depth, and whether it represented an animate or inanimate object. The 12-inch tree limb had the lowest rating of the seven objects. Reasons for this might be that the drivers felt that the tree limb would pose no danger to them or would not damage the vehicle if the object was struck. A tree limb of a different size probably would make a difference in the responses. The 6-inch armadillo shows the second lowest hazard rating, but this is a result of lack of driver sympathy towards an animal such as an armadillo. A different representation of an animate object might show more sympathetic responses, meaning more evasive action required in the vehicle to avoid the animal, increasing the hazard rating. The wooden objects show a general increase in hazard rating as a function of size and depth. This can be attributed to the driver uncertainty of the vehicle being able to straddle the object. The bale of hay received the highest rating. No other option is available to the driver but to go around this object or come to a complete stop if a collision is to be avoided.

Drivers in the older age group showed a higher response to "extreme hazard" for four of the seven objects shown to the subjects. Older drivers also showed a lower 85th percentile running speed. This indicates that despite the lack of statistical evidence to prove that older drivers lack visual capabilities, older drivers were found to be more cautious.

Demographic and Personal Data

The demographic and personal data addressed in this study included age, gender, ethnic background, corrective lens requirements, and the average number of miles driven in a year. Ethnic background information was collected solely for informative purposes and is not presented here. A listing of pertinent information is presented in Table 6. Of the total 43 subjects participating, 16 were less than the age of 25, 15 were between the ages of 25 and 55, and 12 were over the age of 55. All subjects required by law to wear corrective lenses did in fact wear them. A larger percentage (50 percent), however, of subjects over the age of 55 wore corrective lenses, versus 31 percent and 27 percent for less than 25 years of age and between 25 and 55 years of age, respectively. Subjects in the middle age group, on the average, drove 15,700 miles a year. This was the highest of the three age groups. Subjects in the younger age group drove an average of 13,800 miles a year and subjects in the older age group drove an average of 10,400 miles a year. These averages include both genders. Table 6 provides a breakdown of average miles driven per year for each age group and gender.
TABLE 6 Demographic and Personal Data of Participating Subjects

<table>
<thead>
<tr>
<th>Age</th>
<th>Gender</th>
<th># Drivers</th>
<th>Corr. Lenses</th>
<th>Average Miles Driven Per Year</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>&lt;10,000</td>
</tr>
<tr>
<td>&lt;25</td>
<td>Male</td>
<td>10</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Female</td>
<td>6</td>
<td>3</td>
<td>3</td>
</tr>
<tr>
<td>25-55</td>
<td>Male</td>
<td>6</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>Female</td>
<td>9</td>
<td>1</td>
<td>3</td>
</tr>
<tr>
<td>&gt;55</td>
<td>Male</td>
<td>4</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td></td>
<td>Female</td>
<td>8</td>
<td>5</td>
<td>4</td>
</tr>
<tr>
<td>Totals</td>
<td></td>
<td>43</td>
<td>15</td>
<td>15</td>
</tr>
</tbody>
</table>

A visual capability comparison between the three age groups was studied. No statistical evidence supports any difference between the age groups. This is primarily due to the sample size of each age group and the high sample standard deviations calculated. There is evidence, however, that older drivers are more cautious. For the objects shown in the photographs, the older age group showed a higher response to "extreme hazard" for four of the seven objects. The older age group also had a lower 85th Percentile running speed at points of detection and recognition. The speeds showed a general decrease with increasing age. The speed data is listed in Table 7. These values were calculated by combining speeds from all six objects studied.

CONCLUSIONS AND RECOMMENDATIONS

Conclusions indicate that drivers can detect a high contrast 6-inch object within the minimum parameters established by AASHTO for a driver travelling at 48 to 55 miles per hour. The same can be said for objects that are greater than 6-inches in height, regardless of contrast. Further evidence concludes that drivers do not have the visual capabilities to recognize an object of any contrast and reasonable size within AASHTO parameters. This is not totally necessary for stopping sight distance, but the driver must be able to recognize the object as a hazard.

The hazard rating responses for the objects indicate that there is a general increase in hazard rating as a function of the object size. This, however, is not always the ideal case. The analysis of the driver responses shows that small, wooden objects with depth or length dimensions are perceived just as hazardous as objects that are twice the size in height. These results conclude that some objects have to be considered on a case-by-case study, as with the three-dimensional objects, and not compared with other objects that appear as two-dimensional.

This study presented an ideal condition for the drivers: a flat roadway, a dry pavement, and good lighting conditions. The drivers were also alert and anticipating an object in their path. A previous study on perception and reaction time (13) indicates that there is a factor of 1.35 from an anticipation condition to a surprise condition. A surprise condition in this study would only decrease the values obtained for detection and recognition. A driver travelling at 55 mph may not have adequate stopping sight distance even with adequate visual capabilities.

The design of crest vertical curves incorporates the stopping sight distance and the 6-inch object for determining the length, as well as the driver eye-height and the algebraic difference in grades. The AASHTO model assumes the driver will immediately react and brake to an object as soon as the top of it becomes visible over a crest vertical curve. The model does not provide a factor for object recognition or for object contrast. Consideration should be given to designing for low-contrast objects and for providing for a visibility angle. This angle will allow drivers to detect a portion of the object before making a decision to apply the brakes.

Further research should be continued in this area. This study presented an ideal condition for the drivers involved. Research is recommended for real conditions: night time study, adding an element of surprise, testing in different weather conditions (rain or fog), and conducting the study on an actual road where a horizontal or crest-vertical curve might be incorporated into the study.
TABLE 7  AVERAGE AND 85TH PERCENTILE RUNNING SPEEDS (MPH)

<table>
<thead>
<tr>
<th>Percentile</th>
<th>Detection 50th</th>
<th>Detection 85th</th>
<th>Recognition 50th</th>
<th>Recognition 85th</th>
</tr>
</thead>
<tbody>
<tr>
<td>Younger (&lt;25)</td>
<td>39.1</td>
<td>47.5</td>
<td>41.3</td>
<td>47.3</td>
</tr>
<tr>
<td>Middle (25-55)</td>
<td>38.4</td>
<td>45.6</td>
<td>41.1</td>
<td>46.6</td>
</tr>
<tr>
<td>Older (&gt;55)</td>
<td>35.5</td>
<td>42.7</td>
<td>39.1</td>
<td>46.4</td>
</tr>
</tbody>
</table>

REFERENCES


ACKNOWLEDGEMENTS

The author wishes to thank a number of individuals who helped contribute to this research. The author acknowledges Dr. Daniel B. Fambro and Karen B. Kahl of the Texas Transportation Institute and Texas A&M University for their invaluable assistance and advice that made this study possible. Robert Odstrcil of the Texas Department of Transportation and Ronald L. Nowlin of the Texas Transportation Institute are also acknowledged for their contributions to the study, as are the many drivers who participated in this study. Their time is gratefully appreciated. Lastly, the author wishes to thank Dharmesh Shah who assisted in setting up the course and collecting the data.
An Investigation of the Relationship Between Congestion and Air Quality

JANET RICCI

The purpose of this study was to determine if there is a relationship between traffic congestion and air quality and the magnitude of any relationship. This report examines the air quality and traffic characteristics of 50 urban areas. Air quality levels were estimated according to the concentrations and classifications of the Environmental Protection Agency. The air quality was then compared to several traffic and population characteristics of each area.

It was determined from this study that traffic congestion and urban characteristics explain less than 30 percent of the variation in air pollution levels. Other factors such as topography, industry, climate and meteorology have a great impact on a region's air quality. Three pairs of cities with similar congestion levels and different air pollution levels were examined to analyze the impact of these other factors. Each area must be examined separately in order to determine what is responsible for its air pollution.

INTRODUCTION

Traffic congestion is increasing with each passing year. More and more vehicles are being squeezed onto roadways with inefficient capacity. To commuters, congestion means aggravation and longer travel time. Of course these are two of the many impacts of traffic congestion, but what about the impact on the environment? What about air pollution? The automobile does emit hazardous pollutants into the atmosphere but, can air quality be blamed on transportation alone?

Air pollution is the introduction of natural and artificial gaseous and particulate contaminants into the atmosphere(1). It has become a major controversial issue in today's society. In the past few years, the United States has been striving to improve air quality by passing the Clean Air Act Amendments of 1990 (CAAAA) and the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991.

Traffic Factors

Roadway congestion, by reducing travel speed, increases the number of vehicle hours traveled on the roadway system during peak travel hours. This tends to increase the level of emissions from mobile sources. Figure 1 illustrates that automobiles traveling at 25 mph or less emit twice as many pollutants as one traveling at 55 mph. Emissions also increase at speeds greater than 55 mph.

Other causes of higher emissions are acceleration and deceleration patterns which occur during congested traffic conditions, and emissions related to starting and turning off a vehicle. The latter are known as cold starts (starting a vehicle that has not been running within four hours) and hot soaks (emissions that occur as a vehicle cools down). Figure 2 illustrates that these two factors are relatively constant and do not vary according to the distance traveled.

Natural Factors of Air Quality

There are factors besides human factors which influence air quality. Natural disasters such as forest fires or active volcanoes may increase the pollution in an area for a period of time. Meteorological and topographical factors also have an impact on the air quality of a particular region. Since these factors vary throughout the country, so does the amount and effect on air pollution. For example, mountains may trap pollutants in a region by limiting air circulation and therefore concentrations of pollutants will increase. An area with high winds may have lower pollutant concentrations due to better circulation (2).

The Past Twenty Years

According to the EPA's Air Quality Atlas (3), although the population has grown 23 percent since 1970 and highway travel has increased 92 percent, there have
FIGURE 1. HYDROCARBON EMISSIONS BY AVERAGE SPEED ON 10 MILE TRIP

FIGURE 2. HYDROCARBON EMISSIONS BY TRIP
been some impressive improvements in pollutant emissions. Lead has decreased 97 percent, total particulates decreased 59 percent, sulfur oxides 25 percent, carbon monoxide 41 percent and volatile organic compounds 31 percent. The only increase was in nitrogen oxide emissions which increased six percent (3). Since congestion has increased a great deal in the past twenty years and pollutant emissions have decreased, the impact of congestion on air quality does not appear to be well defined.

Pollutants from Transportation Sources

Scientists believe that there is a warming trend in global climate and that it is a direct result of the accumulation of certain gases in the atmosphere such as carbon monoxide and ozone. It has been estimated that automobiles emit 80 percent of the carbon monoxide found in the atmosphere. The other two pollutants emitted by automobiles, hydrocarbons and nitrogen oxides, combine to form ozone. It is estimated that cars are responsible for 40 to 60 percent of the ozone problem in most major U.S. cities (4).

Carbon monoxide (CO) is a colorless, odorless, poisonous gas emitted mainly through the exhaust system of the automobile (3). It is a primary pollutant whose concentration in the atmosphere is partly a function of the rate of emissions from nearby sources. The highest levels of carbon monoxide are found in urbanized areas near congested roadways during peak travel times. Large doses of CO can be deadly to humans. Levels of over 50 percent in the bloodstream can cause coma and death due to the binding of CO with blood hemoglobin. Even at levels above five percent losses of alertness, weakness and drowsiness will occur (5).

Long term, lower level exposures to CO can also have serious effects on human health. There is some adaptation, including the increased concentration of red blood cells, to increase oxygen carrying capacity. People with heart disease are particularly susceptible to the risks of CO exposure. Other groups are identified by occupation and include people working in the transportation system who are continuously exposed to vehicles in operation such as toll collectors, garage workers and traffic police, as well as those in the steel industry and petroleum and chemical industry (5).

Ozone is a secondary pollutant which is formed in the atmosphere by the interaction of hydrocarbons with nitrogen oxides; both primary pollutants are emitted by vehicles. However, a direct relationship does not exist between the amount of these pollutants emitted and the amount of ozone which is formed. There are other factors which influence the formation of ozone such as wind speed and direction, climate, atmospheric stability, and solar radiation. Formation may take several hours or days and cannot be caused by individual hydrocarbon or nitrogen oxide emission sources. Short term health effects of ozone include irritation of the eyes, and an increase in susceptibility to infectious diseases that are contracted through the lungs. Long term effects include increased susceptibility to respiratory infection, permanent damage to lung tissue and impaired breathing capacity (5).

There are two types of ozone, ground-level or tropospheric, and stratospheric. Ozone in the stratosphere is beneficial since it provides a screen from the sun’s ultraviolet rays. Ozone at ground level, however, is a health hazard, an environmental concern and the primary ingredient of smog (3).

Clean Air Act Amendments and the National Ambient Air Quality Standards

In 1990, the Clean Air Act Amendments (CAAA) were passed to instruct the Environmental Protection Agency (EPA) "to implement stronger environmental policies and regulations to ensure better air quality" (6). The main goal of the CAAA is "to achieve and maintain a healthy environment while supporting strong and sustainable economic growth and a sound energy policy" (6). The amendment will require implementation and enforcement actions by both public agencies and private companies.

The Clean Air Act defines pollutant emissions from several different sources. These sources include:

- Point or stationary source — a large concentrated source at a fixed location such as a coal-fired power plant
- Area source — a collection of smaller, dispersed emissions sources such as residential and commercial space heaters and city street systems
- Line source — emissions that are generated uniformly along a line such as an urban highway
- Mobile source or transportation-related source — emissions from highway or off-highway vehicles, aircraft, railroads, or marine vessels

This report will focus on emissions from highway-related mobile sources (7).

Focus of the Study

Air quality measures are based on the National Ambient Air Quality Standards (NAAQS). Standards have been defined for six pollutants, carbon monoxide (CO), lead (Pb), nitrogen dioxide (NO₂), ozone (O₃), particulates and sulfur dioxide (SO₂). If the standards are
not met in a specific area, which may be a metro or urban area or something larger or smaller, the EPA classifies the area as one of several nonattainment groups defined by the severity of pollution. Since the two major pollutants associated with transportation are carbon monoxide and ozone, those will be the focus of this study. An area is a nonattainment area if ozone emissions exceed 0.12 ppm or carbon monoxide emissions exceed 9.1 ppm. A region may be nonattainment if it exceeds either or both of these standards. The NAAQ Standards are displayed in Table 1.

Based on these standards the EPA developed a system of classifying U.S. cities which are areas of nonattainment. The classifications are found in Table 2. These areas have to come into compliance in a given time period according to the severity of its air pollution. If a region fails to meet the requirements by the deadline, it is automatically placed in a classification which is more severe. This results in less government funding and more stringent regulations.

METHODOLOGY

The main objective of this study was to determine the relationship between traffic congestion and air quality. The study encompassed 50 large and medium United States urban areas. Considering the EPA classifications and the congestion index and characteristics of each city, it was determined to what degree mobile sources could be held responsible for the air quality of the area.

TII Database

The Texas Transportation Institute has established a database which includes congestion and vehicle travel statistics for 50 United States urban areas between 1982 and 1990 (9). Several statistics used in this study were obtained from the database including the following:

- **TRAVEL**
  - Daily Vehicle-Miles Traveled (VMT)
  - Transit Trips
  - Passenger-Miles Traveled
- **SUPPLY**
  - Lane-Miles
  - Transit Revenue Miles
- **URBAN AREA CHARACTERISTICS**
  - Population
  - Population Density
  - Registered Vehicles
  - Registered Vehicles per Square Mile
  - Registered Vehicle per Capita
- **TRAVEL-SUPPLY**
  - VMT per Lane-Mile
- **TRAVEL-URBAN AREA CHARACTERISTICS**

VMT per Square Mile
VMT per Registered Vehicle
VMT per Capita
Transit Trips per Capita
Passenger-Miles per Capita
- **SUPPLY-URBAN AREA CHARACTERISTICS**
  - Lane-Miles per Capita
  - Lane-Miles per Square Mile
  - Registered Vehicles per Lane-Mile
  - Revenue Miles per Capita

Combinations of these indicators were also used in the analysis.

All the roadway indicators are the totals for freeways and principal arterial streets. The transit indicators include buses, and heavy, light and commuter rails (10).

Roadway Congestion Index

A roadway congestion index (RCI) is a measure of the severity of a city's traffic congestion. It was developed at TTI in order to study the mobility trends of urban areas. The RCI is calculated using a combination of congestion indicators and the number of vehicles per lane per day for both freeways and principal arterial streets. The equation for RCI is shown in Equation 1 (10).

Table 3 lists the urban areas contained in the TTI database with their RCIs and pollutant concentrations. This data was plotted in order to investigate the relationship between severity of traffic congestion and air pollution. As is shown in the table, 36 of the 50 areas are ozone nonattainment areas. However, only 19 are carbon monoxide nonattainment areas. This can be attributed to the fact that ozone is a much more complex pollutant than carbon monoxide and is more difficult to break up.

Statistical Analysis System

The relationship between the pollutant concentrations in Table 3 and congestion characteristics was investigated using the Statistical Analysis System (SAS). The first analysis involved the coefficient of determination, $R^2$, to relate pollutant concentrations and a single congestion characteristic. $R^2$ is a number between zero and 1.0 which represents the relationship between two variables. An $R^2$ value close to 1.0 indicates a closer relationship between the best fit line and the individual data points than an $R^2$ value near zero. For example, an $R^2$ of 0.50 indicates that 50 percent of the variability in the dependent variable can be explained by the independent variable being tested.
### TABLE 1. NATIONAL AMBIENT AIR QUALITY STANDARDS

<table>
<thead>
<tr>
<th>Pollutant</th>
<th>Averaging Time Period</th>
<th>Standard (ppm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbon Monoxide (CO)</td>
<td>8-hour*, 1-hour*</td>
<td>9.1, 35.0</td>
</tr>
<tr>
<td>Ozone (O₃)</td>
<td>1-hour*</td>
<td>0.12</td>
</tr>
</tbody>
</table>

* not to be exceeded more than once per year.

*Source: Reference (3).*

### TABLE 2. 1990 CLEAN AIR ACT AMENDMENTS CLASSIFICATIONS

<table>
<thead>
<tr>
<th>Pollutant</th>
<th>Classification</th>
<th>Concentration (ppm)¹</th>
<th>Time Period</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ozone (O₃)</td>
<td>Marginal</td>
<td>0.121 to 0.138</td>
<td>3 years</td>
</tr>
<tr>
<td></td>
<td>Moderate</td>
<td>0.138 to 0.160</td>
<td>6 years</td>
</tr>
<tr>
<td></td>
<td>Serious</td>
<td>0.160 to 0.180</td>
<td>9 years</td>
</tr>
<tr>
<td></td>
<td>Severe 1</td>
<td>0.180 to 0.190</td>
<td>15 years</td>
</tr>
<tr>
<td></td>
<td>Severe 2</td>
<td>0.190 to 0.280</td>
<td>17 years</td>
</tr>
<tr>
<td></td>
<td>Extreme</td>
<td>0.280 and above</td>
<td>20 years</td>
</tr>
<tr>
<td>Carbon Monoxide (CO)</td>
<td>Moderate &lt; = 12.7</td>
<td>9.1 to 12.7</td>
<td>5 years</td>
</tr>
<tr>
<td></td>
<td>Moderate &gt; 12.7</td>
<td>12.7 to 16.4</td>
<td>5 years</td>
</tr>
<tr>
<td></td>
<td>Serious</td>
<td>16.5 and above</td>
<td>10 years</td>
</tr>
</tbody>
</table>

¹ Parts per million

*Note: EPA may grant two one-year extensions of attainment date.*

*Source: Reference (9).*
<table>
<thead>
<tr>
<th>Urban Area</th>
<th>1990 RCI</th>
<th>Ozone Concentrations (ppm)</th>
<th>Carbon Monoxide Concentrations (ppm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Los Angeles CA</td>
<td>1.55</td>
<td>0.330</td>
<td>23.4</td>
</tr>
<tr>
<td>Washington DC</td>
<td>1.37</td>
<td>0.165</td>
<td>11.4</td>
</tr>
<tr>
<td>San Fran-Oak CA</td>
<td>1.35</td>
<td>0.140</td>
<td>11.8</td>
</tr>
<tr>
<td>Miami FL</td>
<td>1.26</td>
<td>0.138</td>
<td>---</td>
</tr>
<tr>
<td>Chicago IL</td>
<td>1.25</td>
<td>0.190</td>
<td>---</td>
</tr>
<tr>
<td>San Diego CA</td>
<td>1.22</td>
<td>0.190</td>
<td>9.9</td>
</tr>
<tr>
<td>Seattle Everett WA</td>
<td>1.20</td>
<td>0.131</td>
<td>---</td>
</tr>
<tr>
<td>San Bernardino-Riverside CA</td>
<td>1.19</td>
<td>NA</td>
<td>NA</td>
</tr>
<tr>
<td>New York NY</td>
<td>1.14</td>
<td>0.201</td>
<td>13.5</td>
</tr>
<tr>
<td>Houston TX</td>
<td>1.12</td>
<td>0.220</td>
<td>---</td>
</tr>
<tr>
<td>New Orleans LA</td>
<td>1.12</td>
<td>NA</td>
<td>---</td>
</tr>
<tr>
<td>Honolulu HI</td>
<td>1.11</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Atlanta GA</td>
<td>1.11</td>
<td>0.162</td>
<td>---</td>
</tr>
<tr>
<td>Detroit MI</td>
<td>1.09</td>
<td>0.144</td>
<td>---</td>
</tr>
<tr>
<td>Portland OR</td>
<td>1.07</td>
<td>0.128</td>
<td>10.0</td>
</tr>
<tr>
<td>Boston MA</td>
<td>1.06</td>
<td>0.165</td>
<td>9.8</td>
</tr>
<tr>
<td>Philadelphia PA</td>
<td>1.05</td>
<td>0.187</td>
<td>11.6</td>
</tr>
<tr>
<td>Tampa FL</td>
<td>1.05</td>
<td>0.129</td>
<td>---</td>
</tr>
<tr>
<td>Dallas TX</td>
<td>1.05</td>
<td>0.140</td>
<td>---</td>
</tr>
<tr>
<td>San Jose CA</td>
<td>1.04</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Denver CO</td>
<td>1.03</td>
<td>NA</td>
<td>---</td>
</tr>
<tr>
<td>Phoenix AZ</td>
<td>1.03</td>
<td>0.141</td>
<td>12.6</td>
</tr>
<tr>
<td>Sacramento CA</td>
<td>1.02</td>
<td>0.160</td>
<td>12.6</td>
</tr>
<tr>
<td>Baltimore MD</td>
<td>1.01</td>
<td>0.194</td>
<td>9.5</td>
</tr>
<tr>
<td>Milwaukee WI</td>
<td>0.99</td>
<td>0.183</td>
<td>---</td>
</tr>
<tr>
<td>St. Louis MO</td>
<td>0.99</td>
<td>0.156</td>
<td>---</td>
</tr>
<tr>
<td>Cleveland OH</td>
<td>0.97</td>
<td>0.157</td>
<td>10.1</td>
</tr>
<tr>
<td>Cincinnati OH</td>
<td>0.96</td>
<td>0.157</td>
<td>---</td>
</tr>
<tr>
<td>Norfolk VA</td>
<td>0.96</td>
<td>0.130</td>
<td>---</td>
</tr>
<tr>
<td>Ft. Lauderdale FL</td>
<td>0.94</td>
<td>0.138</td>
<td>---</td>
</tr>
<tr>
<td>Jacksonville FL</td>
<td>0.94</td>
<td>NA</td>
<td>---</td>
</tr>
<tr>
<td>Austin TX</td>
<td>0.94</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Minn-St. Paul MN</td>
<td>0.93</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Albuquerque NM</td>
<td>0.93</td>
<td>---</td>
<td>11.1</td>
</tr>
<tr>
<td>Memphis TN</td>
<td>0.91</td>
<td>0.140</td>
<td>9.6</td>
</tr>
<tr>
<td>Fort Worth TX</td>
<td>0.90</td>
<td>0.140</td>
<td>---</td>
</tr>
<tr>
<td>Nashville TN</td>
<td>0.89</td>
<td>0.138</td>
<td>---</td>
</tr>
<tr>
<td>Hartford CT</td>
<td>0.89</td>
<td>NA</td>
<td>10.2</td>
</tr>
<tr>
<td>San Antonio TX</td>
<td>0.88</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Louisville KY</td>
<td>0.86</td>
<td>0.149</td>
<td>---</td>
</tr>
<tr>
<td>Salt Lake City UT</td>
<td>0.85</td>
<td>0.143</td>
<td>---</td>
</tr>
<tr>
<td>Columbus OH</td>
<td>0.83</td>
<td>0.131</td>
<td>---</td>
</tr>
<tr>
<td>Indianapolis IN</td>
<td>0.83</td>
<td>0.121</td>
<td>---</td>
</tr>
<tr>
<td>Pittsburgh PA</td>
<td>0.82</td>
<td>0.149</td>
<td>---</td>
</tr>
<tr>
<td>Oklahoma City OK</td>
<td>0.79</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Charlotte NC</td>
<td>0.78</td>
<td>0.158</td>
<td>---</td>
</tr>
<tr>
<td>El Paso TX</td>
<td>0.74</td>
<td>---</td>
<td>12.6</td>
</tr>
<tr>
<td>Kansas City MO</td>
<td>0.74</td>
<td>0.120</td>
<td>---</td>
</tr>
<tr>
<td>Corpus Christi TX</td>
<td>0.72</td>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>Orlando FL</td>
<td>0.72</td>
<td>---</td>
<td>---</td>
</tr>
</tbody>
</table>

NA - implies area is nonattainment but concentrations were not available. --- means that the area is not nonattainment.
The initial analysis of the relationship between pollution and a single variable did not find significant $R^2$ values. A second analysis using more than one independent variable was performed to identify higher $R^2$ values. To determine whether an acceptable relationship between the variables existed, three factors were considered: $R^2$, variance inflation and probability. The variance inflation factor is a measure of the relationship between the independent variables, (in this case the two congestion indicators). A variance inflation factor greater than 10 indicates a close relationship between the independent variables (rather than between the independent and dependent variables) and is undesirable.

The T-test probability indicates the usefulness of the variables, and ranges from zero to one. The higher the probability, the closer the variable is to being considered statistically irrelevant in that model.

EXAMINING THE RELATIONSHIP BETWEEN TRAFFIC CONGESTION AND AIR POLLUTION

The statistical relationship between pollution and traffic congestion was investigated using several congestion indicators. To further illustrate the variation in the causes of air pollution, the congestion characteristics and urban characteristics of pairs of cities were investigated.

Ozone Pollution Levels

Of the 50 cities included in the TTI database, 36 are ozone nonattainment areas. Plotting the ozone concentrations versus the roadway congestion index for each of these areas yields the plot in Figure 3. $R^2$, which, in this case, is a measure of the relationship between ozone concentration and the roadway congestion level (RCI), is 0.31. This means that 31 percent of the variance in ozone air pollution can be attributed to traffic congestion and the other 69 percent must be a result of something else. As a general rule, an $R^2$ of 0.50 or higher is needed in order for an acceptable relationship to exist.

The Los Angeles values for both congestion and ozone pollution exceed all other areas by a substantial amount. The statistical analysis was repeated without Los Angeles in order to identify the effect of this potential outlier. The $R^2$ value for this modified data set was 0.11, indicating a significant and undesirable effect from the Los Angeles data. Without the outlier, a more realistic relationship between the other, more similar data, can be determined. The higher $R^2$ appears to be the result of the Los Angeles data point providing a focus to an otherwise fairly random data set.

Since congestion levels explain only 11 percent of the variation in ozone concentrations, other factors were analyzed. These factors were examined using the Statistical Analysis System program (SAS) described in the Methodology section of this report, in comparison to ozone concentrations. Table 4 includes the indicators which yielded the five highest $R^2$ values. The Los Angeles values are presented for illustration only, and are not used in further analysis.

Combinations of the indicators from Table 4 were examined in relation to the ozone concentrations to identify better relationships between air pollution and congestion characteristics. Listed in Table 5 are the combinations considered, the $R$ squared value, the probability and the variance inflation factor. The variance inflation, as discussed in the Methodology section of this report, is a measure of the relationship between the independent variables. Since models three, four and six have variances near or greater than 10, a close relationship between the independent variables exists and these models are not suitable for use in predicting ozone levels. Models one through four all include vehicle miles traveled (VMT), and it is observed that adding another variable to VMT will not increase the usefulness of the VMT factor alone ($R^2 = 0.27$) to predict ozone levels. This would indicate that using a combination of two variables does not increase the amount of air pollution explained by vehicle travel.

There is also no substantial improvement in $R^2$ values for either of the lane-mile models included in Table 5 (models five and six).

Carbon Monoxide Pollution Levels

The relationship of carbon monoxide concentration and the congestion level in the 19 carbon monoxide
FIGURE 3. CONGESTION LEVEL VS. OZONE POLLUTION

TABLE 4. SINGLE INDICATORS FOR OZONE LEVEL

<table>
<thead>
<tr>
<th>Indicator</th>
<th>Coefficient of Determination ($R^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Without L.A.</td>
</tr>
<tr>
<td>Daily Vehicle Miles Traveled (VMT)</td>
<td>0.27</td>
</tr>
<tr>
<td>Lane-Miles</td>
<td>0.26</td>
</tr>
<tr>
<td>Urban Area</td>
<td>0.26</td>
</tr>
<tr>
<td>Population</td>
<td>0.25</td>
</tr>
<tr>
<td>Registered Vehicles</td>
<td>0.25</td>
</tr>
</tbody>
</table>

Note: The difference between the $R^2$ values for the analysis with and without Los Angeles indicates that analyzing the data set which includes Los Angeles would not produce accurate results.
TABLE 5. COMBINATIONS OF INDICATORS FOR OZONE LEVEL

<table>
<thead>
<tr>
<th>Model #</th>
<th>Factors</th>
<th>$R^2$</th>
<th>Probability $&gt; T$</th>
<th>Variance Inflation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>VMT Population Density</td>
<td>0.27</td>
<td>0.0507</td>
<td>1.95</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.5550</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>VMT Lane-Miles per Sq. Mile</td>
<td>0.27</td>
<td>0.0032</td>
<td>1.07</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.6726</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>VMT Lane-Miles</td>
<td>0.27</td>
<td>0.7160</td>
<td>35.11</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.8345</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>VMT Reg. Vehicles</td>
<td>0.27</td>
<td>0.3696</td>
<td>11.37</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.9178</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Lane-Miles VMT per Sq. Mile</td>
<td>0.27</td>
<td>0.0058</td>
<td>1.21</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.7584</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Lane-Miles Registered Vehicles</td>
<td>0.26</td>
<td>0.3871</td>
<td>9.96</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.8371</td>
<td></td>
</tr>
</tbody>
</table>

Note: Probability $> T$ indicates the probability that the factor is statistically irrelevant to the model (the higher the probability, the less useful the factor is in the model). Variance Inflation indicates the relationship between the factors.

nonattainment areas (Figure 4) has an $R^2$ of 0.30 and an $R^2$ of 0.11 if Los Angeles is omitted. As was the case for ozone, Los Angeles was also determined to be an outlier for the carbon monoxide analysis.

The travel, supply and urban characteristic indicators were analyzed against the carbon monoxide concentrations. The indicators with the five highest $R^2$ values (Table 6) are slightly lower for carbon monoxide than for ozone indicating less of a relationship between carbon monoxide concentrations and these indicators. This analysis also identified a significant drop in $R^2$ values if the Los Angeles data were omitted.

Table 7 illustrates the indicator combinations which were compared to the carbon monoxide concentrations. The indicators in model 1 did significantly increase the $R^2$ value from that of the single variable VMT. The probability values for model 1, however, are not as low as desired. They indicate a 6 to 13 percent possibility that the factors are irrelevant. The changes in $R^2$ in all the other models, however, were not significant when compared to the single factor models.

ILLUSTRATIONS OF THE CAUSES OF AIR POLLUTION

To further analyze the relationship between traffic congestion and air quality three individual sets of urban areas were examined. Each set consisted of two of the 50 urban areas which were relatively the same size and having the same or relatively the same congestion index and yet had different EPA classifications. The purpose of this analysis was to use the urban areas as illustrations of the relationship between traffic and population statistics and air pollution. From this analysis the other causes of air pollution could then be determined.

Summary of Non-Transportation Related Factors

There are many factors which contribute to air quality besides transportation and traffic congestion. The following are those factors which are discussed in this study.

Meteorology

Meteorology includes speed and direction of wind currents and atmospheric stability. Wind plays an important role in the dispersion of pollutants, especially carbon monoxide. A stable atmosphere allows the buildup and formation of pollutants, in the case of transportation ozone is the pollutant most affected.

Topography

Mountains are a major topographical feature which play an important role in air pollution. They form a
FIGURE 4. CONGESTION LEVEL VS. CARBON MONOXIDE POLLUTION

TABLE 6. SINGLE INDICATORS FOR CARBON MONOXIDE LEVEL

<table>
<thead>
<tr>
<th>Indicator</th>
<th>Coefficient of Determination ($R^2$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Without L.A.</td>
</tr>
<tr>
<td>Urban Area</td>
<td>0.11</td>
</tr>
<tr>
<td>Registered Vehicles</td>
<td>0.09</td>
</tr>
<tr>
<td>Lane-Miles</td>
<td>0.09</td>
</tr>
<tr>
<td>Revenue Milts</td>
<td>0.08</td>
</tr>
<tr>
<td>Daily Vehicle Miles Traveled (VMT)</td>
<td>0.07</td>
</tr>
</tbody>
</table>

Note: Much like the $R^2$ values for ozone, the values in Table 6 are greatly reduced when the analysis is performed without Los Angeles data.
### TABLE 7. COMBINATIONS OF INDICATORS FOR CARBON MONOXIDE LEVEL

<table>
<thead>
<tr>
<th>Model</th>
<th>Factor</th>
<th>R Squared</th>
<th>Probability &gt; T</th>
<th>Variance Inflation</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>VMT</td>
<td>0.21</td>
<td>0.0639</td>
<td>3.12</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>0.1279</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Lane-Miles</td>
<td>0.10</td>
<td>0.2078</td>
<td>1.15</td>
</tr>
<tr>
<td></td>
<td>VMT per Sq. Mile</td>
<td></td>
<td>0.6155</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>VMT</td>
<td>0.10</td>
<td>0.6485</td>
<td>33.15</td>
</tr>
<tr>
<td></td>
<td>Lane-Miles</td>
<td></td>
<td>0.5139</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>VMT</td>
<td>0.10</td>
<td>0.7358</td>
<td>14.74</td>
</tr>
<tr>
<td></td>
<td>Registered Vehicles</td>
<td></td>
<td>0.5217</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Lane-Miles</td>
<td>0.09</td>
<td>0.9526</td>
<td>12.59</td>
</tr>
<tr>
<td></td>
<td>Registered Vehicles</td>
<td></td>
<td>0.7748</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>DVMT</td>
<td>0.07</td>
<td>0.2943</td>
<td>1.06</td>
</tr>
<tr>
<td></td>
<td>Lane-Miles per Sq. Miles</td>
<td></td>
<td>0.9295</td>
<td></td>
</tr>
</tbody>
</table>

**Note:** Probability > T indicates the probability that the factor is statistically irrelevant to the model (the higher the probability, the less useful the factor is in the model). Variance Inflation indicates the relationship between the factors.

Barrier, which the pollutants cannot penetrate, trapping the air and reducing circulation.

**Climate**

Climate includes both sunlight and rainfall. Sunlight radiation is a major component in the formation of ozone. The more sunlight, the higher the chance of ozone formation. Rainfall, however, works in the opposite way. The more rainfall, the less sunlight radiation, and the smaller the chances for ozone formation. Rainfall also washes other pollutants from the atmosphere.

**Industry**

Unlike the factors already discussed, industry is a direct source of pollutant emissions. It emits pollutants such as carbon monoxide, hydrocarbons and nitrogen oxides into the atmosphere. The amount and type of industry will also contribute to an urban area’s air pollution.

**Location**

Sometimes an urban area is located in the vicinity of another area which is highly polluted. Because of wind currents it is sometimes possible for one area to receive pollutants from other areas.

**Houston and Dallas**

The first study was of Houston and Dallas. Both are urban areas within the state of Texas. Houston is a slightly more congested area with an RCI of 1.12. From the classifications and roadway congestion indexes in Table 8, it seems that the air quality worsens with an increasing congestion index. The percentages of hydrocarbon emissions from mobile sources, however, is 65 percent in Dallas and 31 percent in Houston. Hydrocarbons (which are not shown in Table 8) are primary pollutants emitted directly from automobiles and are precursors of ozone.

Another factor in air quality is rainfall. Rain reduces pollutant concentrations in the atmosphere. In this case, however, Houston has a greater average annual rainfall and greater rainfall during the hotter summer months (1), but still has a poorer classification. Most other climate, topographic and meteorological features are essentially the same in both areas.

Since there was no evidence that congestion, climate, topography, or meteorology were the causes for the poorer air quality in Houston, other factors had to be explored.

One factor that may be responsible for the difference is the higher industrial emissions in Houston. Much of
the hydrocarbon emissions are produced in Houston's petrochemical operations, which are not present in Dallas. The contribution of these stationary sources would help explain the higher pollutant levels in Houston despite the lower mobile source percentage.

Seattle and San Bernardino

The second case study was Seattle, Washington and San Bernardino, California. Both areas are west coast cities with similar population densities. San Bernardino is considered part of the Los Angeles area and falls into the extreme category, while Seattle is classified as a marginal nonattainment area. Three factors were identified that might explain the variation in the air quality (Table 9) between the two cities. The first is climate. Sunlight plays an important role in the formation of ozone. San Bernardino has a greater number of annual average sunny days (73) than Seattle (45). Seattle has more rain and less sunshine which does not allow a build up of sunlight radiation. Without this build up, ozone formation is hindered.

Other factors contributing to the air quality difference are meteorology and topography. San Bernardino is located between mountains and the Pacific Ocean. The wind direction is inland off the coast towards the mountains which pushes the Los Angeles area pollutants to San Bernardino-Riverside and also reduces air circulation in San Bernardino-Riverside. While Seattle is also near a mountain range, the pollution is only generated in the Seattle area. Wind speed also plays a role in air quality, and the average wind speed is greater in Seattle.

Milwaukee and St. Louis

The third study was Milwaukee, Wisconsin and St. Louis, Missouri which have the same congestion index (Table 10). The percentage of hydrocarbon emissions from mobile sources (42%) is also the same as obtained from the East-West Gateway Coordinating Council, St. Louis, MO and the Department of Natural Resources, WI. St. Louis has a slightly warmer climate and Milwaukee has higher wind speeds. However, Milwaukee's air quality is much worse than St. Louis'.

This might be attributed to several factors. Over the past few years, St. Louis has had fewer sunny days than normal. Less sunlight results in lower ozone concentration. St. Louis has also experienced greater wind currents, yielding better air circulation and less pollution. (Source: East-West Gateway Coordinating Council) Milwaukee, while an industrial area with local pollution sources, is also near Chicago. As with San Bernardino-Riverside, some air pollution from Chicago (a severe-17 area) is carried to Milwaukee.

CONCLUSIONS

Traffic congestion appears to be a significant, but not overwhelming contributing factor to air quality. A direct relationship between congestion and air quality does not exist, and less than 15 percent of the variability in air pollution levels can be explained by traffic congestion levels. This percentage is, however, an average and varies for individual areas. It was determined that more of a relationship exists between air pollution and the magnitude of transportation and population factors than the intensity of activity or ratios of those factors. The best relationships were found between air pollution and the following factors:

- Daily Vehicle Miles of Travel (VMT)
- Lane-Miles
- Registered Vehicles
- Revenue Miles
- Urban Area
- Population

Besides congestion and population there are several other factors which influence air quality in particular regions. These factors included:

- Meteorology
- Topography
- Climate
- Industry
- Location
TABLE 9. CONGESTION AND POLLUTION STATISTICS FOR SEATTLE AND SAN BERNADINO

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Seattle-Everett, WA</td>
<td>1,730</td>
<td>2,390</td>
<td>1.20</td>
<td>Marginal</td>
</tr>
<tr>
<td>San Bernardino-Riv., CA</td>
<td>1,170</td>
<td>2,390</td>
<td>1.19</td>
<td>Extreme</td>
</tr>
</tbody>
</table>

TABLE 10. CONGESTION AND POLLUTION STATISTICS FOR MILWAUKEE AND ST. LOUIS

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Milwaukee, WI</td>
<td>1,230</td>
<td>2,240</td>
<td>0.99</td>
<td>Severe-17</td>
</tr>
<tr>
<td>St. Louis, MO</td>
<td>1,960</td>
<td>2,690</td>
<td>0.99</td>
<td>Moderate</td>
</tr>
</tbody>
</table>

The effects of these factors were illustrated by examining pairs of urban areas. It was found, however, that in most cases a combination of these factors existed, indicating that many factors are responsible for air pollution.

What was not determined in this analysis was the exact combination of these factors which produce poor air quality. Transportation systems and traffic congestion contribute to air quality problems, but this study noted that many other factors are involved in predicting air quality. Transportation or traffic congestion, unlike most natural factors however, can be affected by the actions of governmental agencies. In order to properly determine the actions needed to improve air quality it is necessary to analyze the characteristics and factors in individual areas.

REFERENCES

Analysis of the Houston Metro Electronic Information System

TRICIA A. THOMASON

A number of transit systems around the country have tested the use of a variety of advanced public transit systems technologies. Many of these focus on improved customer information and customer interface. One such test, implemented by Houston METRO is an electronic information map to provide enhanced customer information. Currently, three "Digiplan" maps are located in downtown Houston. This research project examined the use of transit customer information systems and the specific application of electronic maps and other new technologies. Preliminary information on the use of the METRO Digiplan maps was examined and possible improvements and upgrades to the existing Digiplan maps were researched and reviewed.

The results of this analysis indicate that the Digiplan maps were limited by several important factors including site selection, technology capabilities/availability, and data evaluation. These factors are summarized in order to help METRO and other agencies interested in developing similar systems. Thus, the results of this analysis should be of interest to METRO and other transit and transportation agencies.

INTRODUCTION

A number of transit systems around the country have been implementing advanced technologies to improve the overall efficiency of their systems. The main purpose of these improvements is to encourage transit use, increase the number of passengers, and improve operations and management. To help promote and develop these new technologies the Federal Transit Administration (FTA) initiated the Advanced Public Transit Systems (APTS) program (1). The purpose of this program is to focus and coordinate funding and programs related to public transit. The Metropolitan Transit Authority of Harris County (Houston METRO) is using one application of APTS technology to provide enhanced customer information. METRO is currently testing three "Digiplan" maps in downtown Houston. This technology is intended to increase both the riders and non-riders knowledge of the transit system by improving customer information and customer interface capabilities.

This research study was undertaken to examine the experience to date with the use of the Digiplan maps and to identify potential areas for improvement. To accomplish this, the study includes an overview of transit customer information systems and the specific use of electronic maps and other advanced technologies, documents the development and implementation of the Digiplan maps in Houston, analyzes the experience to date with the Digiplan maps, and identifies possible improvements in the maps. The suggestions for potential improvements should be of benefit to both METRO and other agencies interested in developing similar systems.

The Importance of Public Transit Information Systems

The availability of accurate information is an essential part of transit operations. For passengers to use a transit system, they must have an understanding of how to get to their desired destination, fare levels, and schedules. In the past, transit systems have used pamphlets, time schedules and simple maps to inform passengers. Research, however, suggests that this type of information is not sufficient (2). For example, one study found that only nine percent of a sample group was able to successfully determine from a printed schedule when the next bus would pass their home, and only 23 percent and 69 percent of the respondents were unable to use a transit route guide to plan an actual transit trip (3).

The main objectives of providing passenger information is to encourage the use of transit and to increase the number of passengers. The direct benefits to the transit agency are simple; increased ridership. There are also direct benefits to society that include an increase in overall productivity, efficiency, and safety of the transportation system while reducing travel time,
pollution, and congestion. These improvements are especially important today since new social and environmental restrictions are being placed on the transportation system.

Many transit systems in the United States are implementing a variety of new services and programs to increase ridership. One approach is to increase the public's knowledge about the transit system. Simply said by Fruin,

"Transit trips are encouraged where there is knowledge and confidence about connections to a prospective destination, frequency of services, and trip costs. Conversely, the lack of this information discourages trips by public transit, particularly by non-peak-period passengers, areas visitors, and other types of infrequent or new users" (4).

As technology develops, so do the capabilities of information systems. In the past, information was transferred visually by printed brochures or fixed signs, but now changeable electronic screens are easily available and relatively feasible. Customer interface provides the passengers with specific information they want or need. The programming capabilities of the computer allows more information to be accessed and used. This information can include timetables, travel distances, fares, geography, real-time information, traffic conditions, local weather, points of interest and emergency facilities.

This report begins with an introduction that describes the importance of information systems to public transit. The background provides a history and general facts on the Digiplan map and reviews some similar applications in use in other areas. This is followed by the analysis, which describes the technical data, the methodology, results and a brief discussion on some of Digiplan's limitations. Finally, the paper concludes with a summary which includes a short explanation of some limitations of the system and possible future improvements.

BACKGROUND

Advanced Public Transit Systems (APTS) are just one element of the Intelligent Vehicle Highway System (IVHS) which is intended to make trips faster, easier and safer for the public. IVHS can be defined as the application of advance technologies and communications to improve the overall efficiency and effectiveness of the transportation system. The goal of IVHS is to make significant improvements in mobility, safety and productivity by building transportation systems that utilize advanced technologies, communications, and computer software (5). It is important to realize that IVHS is not a single static technology, but the continuing development of new technology. The five general categories that have been defined and accepted are: advanced traffic management systems (ATMS), advanced public transit systems (APTS), advanced traveler information systems (ATIS), advanced vehicle control systems (AVCS), and commercial vehicle operations (CVO). The Digiplan maps are an example of both ATIS and APTS technologies.

The Digiplan Map

The Digiplan map is an electronic information system device which gives necessary information at your fingertips on such things as bus numbers and local interest points. Individuals can access the system by touching their origin and their desired destination. The Digiplan computer then provides information on bus routes, schedules, and other information between the two pairs. By simply pushing a button, the passenger can receive a printed copy of their suggested route. These "directions" include such things as bus numbers, departure times, transfer connections, and general information on that location.

According to a METRO staff member, METRO first showed interest in Digiplan when the general manager viewed a similar system in France (6). After being approached by the manufacturer, METRO agreed to take the Digiplan maps as a demonstration project only. In 1990, the cost of three machines totaled $96,000. The machines were purchased with a buy back option after an evaluation in December of 1991, in case METRO decided to stop the demonstration.

Testing of the first machine was initiated in June of 1990, followed by two more in September of the same year. After the systems were first evaluated, it was decided to continue testing for another year, to provide additional time to examine the use and benefits of the maps. Currently, METRO, with the assistance of the Texas Transportation Institute, has applied for FTA/FHWA demonstration funding to expand and enhance the capabilities of the maps. The next evaluation of the maps by METRO is scheduled for December of 1992.

The three locations of Digiplan maps are shown (Figure 1) and briefly described below.

- Louisiana and Polk - This location is inside the receptionist office of Houston Metro on the second floor. The machine is only available during the regular business hours of Monday through Friday from 8:00 a.m. to 5:00 p.m.
- Brown Convention Center - This machine is located on the second floor where registration
takes place. It is only in operation during group registration.

- Capital and Fannin - This machine is located inside the METRO ridestore, which already has an information assistant. It is only open during the business hours of Monday through Friday from 8:00 a.m. to 5:00 p.m.

Other Maps in Use

The Houston Digiplan map is just one application of an electronic information system that uses customer interface. The technology and the machine originated in France where it is used extensively. Currently, Digiplan’s route-planning is based only on the Houston bus routes because the system was designed for Houston METRO. Digiplan is just one approach to this type of technology, but there are several different applications that are being tested. Two cities that use similar applications of this technology are Baltimore, Maryland and Orlando, Florida. The major elements of these systems are briefly summarized to provide a comparison to the Digiplan system.

In Baltimore, a kiosk system was placed at the new baseball stadium for the Orioles. This system uses the easy touchscreen monitor to provide information to the fans on bus and rail services. The project uses the Westinghouse technology and was developed by the Maryland Mass Transit Administration to help encourage use of the light rail system by people attending the new baseball stadium. Passengers can obtain information on schedules, maps, local interest points, and a user’s guide (7).

Another interesting application in Orlando is called the “TravelerMatch Express.” This system, which was developed by the American Automobile Association (AAA) and Navigation Technologies (NavTech) was created for automobile travel. The kiosks were scheduled for implementation by late summer 1992 and will be tested for six months. The twenty-five machines will be installed at the airport, and selected hotels and car rental counters. They will feature an easy touchscreen color monitor, 40 megabyte hard disk and 4 megabyte of RAM. Information will include the local area map, hotel and restaurant locations and descriptions, and travel information (8).

ANALYSIS

This study examined the current experience with the use of the Digiplan maps through the examination of data and information from METRO and personal observation of the use of the maps by the researcher. Possible improvements and upgrades to the existing Digiplan maps were researched and analyzed. The results of this study will assist in determining the current experience with the maps and the identification of potential improvements that can be incorporated into Digiplan to increase the effectiveness of the system. In particular, factors that appear important in starting and implementing a successful electronic information system are identified. These should be of benefit for improving the Houston system and to other agencies interested in developing similar systems.

Technical Data

The Digiplan machine was created, designed and manufactured in France by HII (CGA Camp Transport, Alcatel Alsthom Group) with technical cooperation from les Transports en Commun Lyonnais (9). It consists of two sections: the upper module and the lower module. The general shape and dimensions of the system are illustrated in Figure 2. The upper module consists of a large capacitive keyboard that has a symbolized map of Houston superimposed on it. Digiplan is characterized by the large, colorful map face made from epoxy PCB glass. The keyboard consists of 2279 individual keys that are organized into a simple matrix of 43 rows and 52 columns. Therefore, the general location in Houston can easily be identified by the row and column number. Each individual key can be divided into 12 sub-keys, that allow a particular destination to be found within that general location. In dense areas of the city, this is important because the map is limited by the small scale necessary to fit the whole Houston area on the map.

The system is automatically activated when one of the capacitive keys are touched. The presses’ key generates a change in the capacity, which creates an imbalance that is detected by the decoding board. After detecting the imbalance, the decoding board defines the line and column coordinates of the key. The coordinates are then transmitted to the processor. For Digiplan to work effectively, it is important that the location selected by the customer is detected quickly and accurately.

The computer system receives information from the decoding board, processes the information, and then displays the detailed information on the video screen. The printer makes it possible to print this information for the customer to take with them. The system is a 286 IBM compatible computer with 20M hard drive, 640K memory, monochrome display, printer and a 5.25” floppy disk drive. The French used Pascal to develop a computer software program for Digiplan. The program is designed to calculate the best-route available using the average times of scheduled bus timetables.
There are some other features and options that are available to Digiplan that are self-explanatory. The regular features are a heater, power supply unit and power protection device. The optional features include: air-conditioning device, color monitor, ticket machine and coin-operation device.

Analysis of Data

Since the first machine was implemented in June of 1990, Houston Metro has been collecting data. However, to date METRO had not undertaken any detailed analysis of these data. Most of the data were collected and organized according to the key numbers. METRO provided the researcher with examples of this original data in printed form. From this, the location of the request was identified by the key number and plotted on a citywide map of Houston. A general count and date from the printouts provided an approximate number of daily users.

Recent information from the Louisiana and Polk machine was received from METRO on floppy diskette. The files were in ASCII format and transferred into spreadsheets for analysis. There were four different formats of the same data information. Two of these formats had statistical counts, Digiplan key numbers, and location names. These two files were sorted twice, once according to frequency of use and once alphabetically. The keys with the largest inquiries were noted while the ones with no inquiries were erased. The top locations, general information and languages were then calculated from these sorted files.

The locations or street names that were requested the most were determined and are shown in Table 1. The most popular general information that was accessed on the map face are shown in Table 2. Even though English is the default language, there are four other languages available. The top three languages are shown in Table 3.

The extent of the analysis was limited by the availability of the data from the maps. The available data were arranged with a general count of how many times each key was pressed. Currently, there is no daily counting device or method of calculating the daily use accurately. Another problem is distinguishing between the passengers "using" Digiplan compared to how many are playing with the machine. A ridestore employee provided a rough approximation of their daily users to be about five percent of the visitors to the ridestore (10). There is also no way of determining if people find the information of benefit and actually use it for trip planning purposes. A survey of users would be needed to accurately determine this. While this was outside the scope of this analysis it may be appropriate for further study.

In Europe, public transportation agencies commonly use electronic computer displays to inform passengers of arrival times, cancellations, and delays. Europe has found these types of information systems to be very successful. One reason for the success is the fact that Europe's transportation systems are centered on mixed rail and trolley operations, which supply high passenger volumes in relatively centralized locations. Another reason for the success of these systems is due to the wide variety of languages European travelers speak. The demand and need of supplying travelers with several languages is very common and important. Basically, the European culture and transportation system demands the use of information systems that supply customer interface.

The American culture and transit systems are different from Europe because they generally do not have language problems and they lack centralized locations. Problems with buses also occur because bus routes are frequently changed. They are also dispersed throughout the city which makes it hard to establish locations with high traffic volumes. With just a few modifications, transit agencies should find similar information systems will improve operations.

CONCLUSIONS

The analysis of the Digiplan maps conducted in this research study identified a number of potential improvements and enhancements that could be made to expand the capabilities, and, thus, the use of the system. These suggestions, which are outlined below, should be of use to METRO in examining ways to improve the existing maps and the areas that may be interested in implementing similar systems.

- location of maps
  Currently, the three maps are located in areas with relatively low traffic volumes. Ideally, these types of information devices should be located in high traffic volume areas where tourists, visitors, and non-riders can easily access the system. However, security is an issue and the maps need to be situated where they can be protected from possible vandalism and abuse.
- technical capabilities
  Currently, only major bus route information is available on the Digiplan maps. This has limited the route planning capabilities of the system. Further, the maps are capable of handling additional information on a variety of subjects. Some of these include transit real-time information, weather
### TABLE 1

<table>
<thead>
<tr>
<th>The Ten Most Requested Locations</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. University of St. Thomas</td>
</tr>
<tr>
<td>2. Astroworld/Astroworld</td>
</tr>
<tr>
<td>3. Sugarland</td>
</tr>
<tr>
<td>4. Main &amp; Richmond</td>
</tr>
<tr>
<td>5. Main &amp; Polk</td>
</tr>
<tr>
<td>6. Texas Southern University</td>
</tr>
<tr>
<td>7. Memorial Park</td>
</tr>
<tr>
<td>8. Johnson Space Center</td>
</tr>
<tr>
<td>9. Addicks Reservoir</td>
</tr>
<tr>
<td>10. Rice University</td>
</tr>
</tbody>
</table>

### TABLE 2

<table>
<thead>
<tr>
<th>Top Five Information Inquires</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. METRO</td>
</tr>
<tr>
<td>2. Libraries</td>
</tr>
<tr>
<td>3. Shopping</td>
</tr>
<tr>
<td>4. Golf</td>
</tr>
<tr>
<td>5. Universities</td>
</tr>
</tbody>
</table>

### TABLE 3

<table>
<thead>
<tr>
<th>Top Three Languages Used</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. English</td>
</tr>
<tr>
<td>2. Spanish</td>
</tr>
<tr>
<td>3. French</td>
</tr>
</tbody>
</table>
conditions, traffic conditions, and general information on Houston. Expanding the system to include this type of information would enhance its potential use.

- data collection/evaluation
  
  As noted earlier, a restricting factor in this analysis was the limited amount of available data on the use of the system and the mixing formats of current data. Developing a more structured data collection and evaluating program would assist in future evaluations. Major elements to be considered in such a process should include monitoring daily use, monitoring major destinations requests, and conducting a survey of users to determine their reactions and use of the system.

  
  In conclusion, this analysis indicated that while the Digiplan maps appear to provide the capability for enhanced customer information and interface, improvements could be made that would increase their use and benefits. These improvements, which were summarized above, would benefit current riders, potential passengers, visitors, and tourist, and METRO. Further, consideration of these improvements should be of benefit to other agencies interested in developing similar systems.

REFERENCES


The Impact of a Newly Added Traffic Signal On An Urban Arterial Signal System

JOE VAN ARENDONK

This paper is an impact assessment study based on a hypothetical scenario. A shopping mall or large complex is to be constructed on both sides of Valley Boulevard, an actual existing artery in Alhambra, California. The effect of the new complex will be an increase in traffic volumes, from approximately 80 to 230 vehicles per hour, on 5th Street, a two-way cross street. Due to the increase in traffic, a study must be made to first verify the need for a traffic signal at the intersection of 5th Street and Valley Boulevard, and then to analyze the effects of the traffic signal and increased traffic volume on the performance of the artery. In order to study these effects, two computer models will be used, namely TRANSYT-7F and PASSER-II. Part of the study's objective is to introduce these models to the first time user and assess their capabilities.

The findings were as follows: a signal was needed at the intersection due to excessive delays and dangerous conditions; as a result of the signal, the system wide measurements of effectiveness (MOEs) or performance indicators did not change considerably, whereas the MOEs at the intersection in question changed considerably; in regard to the computer models, TRANSYT-7F was seen to be insensitive to geometric changes and to actuated control; when PASSER-II was used in combination with TRANSYT-7F to optimize values, the MOE's improved for the system whereas they declined for the intersection.

INTRODUCTION

Problem Statement

A large complex, such as a shopping mall, is to be constructed on either side of Valley Boulevard in Alhambra, Los Angeles, at 5th Street, a cross street of the artery (see Figure 1). The traffic volumes will increase throughout the artery; especially on 5th Street, which currently has low volumes and a stop sign in place at the intersection. The problem is to assess the impact of the proposed complex on the performance of the arterial system, and more specifically on the intersection that is immediately affected.

Scope

Due to the length of this research, a full impact assessment of the entire artery, requiring all volumes at every intersection to change according to the added volume at 5th Street, will not be examined as such. The system under study will be limited to six intersections along Valley Boulevard. To facilitate the analysis, the volumes will be held constant throughout the system with the exception of the 5th Street complex.

Study Objectives

The first study objective of this report is to assess the impact of the added volume, in the manner previously discussed, on the arterial system with the aid of computer models. The results of the analysis will hopefully lead to some trends which will be addressed in the conclusion. The second study objective of this report is to gain a better understanding of computer modeling in arterial analysis. This report is mainly written for the reader who is unfamiliar with the problem at hand and the computer models, PASSER-II and TRANSYT-7F. For this reason, the procedural analysis will be well documented in order that the reader become more familiar with the two computer models. The sensitivity of the computer models will also be analyzed by observing the various outputs to corresponding inputs. These trends will also be addressed in the concluding remarks.

Methodology

In order to study this problem, two computer models will be used, namely TRANSYT-7F and PASSER-II. Data for Valley Boulevard in its present state, before the proposed construction, has already been gathered and furnished to the Texas Transportation Institute by the Los
FIGURE 1. DIAGRAM OF SYSTEM
Ana County Department of Transportation in February of 1989. This data will be used as inputs to the two computer models just mentioned.

The study will involve verifying the possible need of installing a traffic signal at the intersection of Valley Boulevard and 5th Street. By analyzing the existing system with high volumes, the necessity will become apparent.

A "volume screen" will be placed around the 5th Street intersection in order that the volumes remain constant throughout the system. With this hypothetical model, the traffic signal's effect on the system can be separately analyzed without considering the apparent volume effects.

To further assess the system's and model's sensitivity to changing conditions, the new higher volume, from the added shopping mall, will be varied from 10 to 200 percent; and two cross streets, namely 5th Street and 9th Street, will be moved to different locations along the artery. Such added changing conditions will lead to a better understanding of the sensitivity of the system to possible signal location.

Description of Computer Modeling
Used in Arterial Analysis

Before discussing the computer models used in this project, a few terms must be defined.

Cycle length - the amount of time during which all movements at a signalized intersection are accommodated. There is usually a minimum cycle length associated with each coordinated system.

Offset - the time from the system reference point to the beginning point of the cycle at the individual signal controller.

Splits or interval, phase length - the amount of time devoted to each phase.

Pretimed control or fixed time - the traffic signal timings are pre-set for a given time of the day.

Actuated control - the duration of phases may vary from cycle to cycle according to traffic demands. The cycle length may also vary (1).

Simulated outputs - arterial traffic performance for a given timing plan. These outputs are analyses of the systems performance as it exists.

Optimized outputs - The optimal combination of cycles, splits, and offsets are determined to give the lowest PI or performance index value (a dimensionless value denoting total delay plus total stops). A traffic engineer may use the optimized settings given by the models to improve the performance of the system.

Computer models are used to facilitate the analysis of an arterial system. TRANSYT-7F is a macroscopic computer program, meaning it analyzes traffic flow as groups of vehicles as opposed to singular vehicles, which determines cycles, splits, and offsets to minimize PI, whereby phase sequences are inputs. PASSER-II determines phase sequences, cycle length, splits, and offsets to maximize bandwidth in an arterial system. Bandwidth applies to the band associated with the time-space diagram, which will be discussed in more detail later. By maximizing the bandwidth, PASSER-II allows more vehicles to travel with progression, namely never stopping at a red, through the artery (1).

Data is entered into TRANSYT-7F in card image format, since the model was created in 1967 when mainframe computers were still widely used. There are more user friendly programs in use today which facilitate the use of the model, such as AAP and McT7F, developed by the University of Florida; however, understanding the card image format can be very useful. Various manuals exist which explain the card format, such as the TRANSYT-7F User's Manual prepared by the University of Florida for the U.S. Department of Transportation. Please refer to the manual when creating an input file.

PASSER-II is a much easier model to work with because the input format is very user friendly. The user is given a picture of each intersection and then asked to input data in the spaces provided, which leaves very little room for error. AAP can combine both PASSER-II and TRANSYT-7F into one input interface. Values can easily be transferred between the two models.

With the aid of TRANSYT-7F and PASSER-II the system in this project can easily be examined. All the data is entered into each program and the corresponding outputs are then analyzed. Since both models have the ability to simulate and optimize the outputs, each stage in the analysis, from the present system to the proposed system, after the shopping mall is constructed, will be analyzed with simulated and optimized runs. These two runs will show the difference between the actual and idealized performance at each stage.

For this project, PASSER-II will be used to ascertain better starting values for TRANSYT-7F. The output from PASSER's optimized run will be used as inputs into TRANSYT, and supposedly, this will give the best results.

Existing System Defined

At present, assuming no changes since February of 1989 when the data was collected, the system, seen in
Figure 1, contains five major cross streets on a 0.65 mile stretch; namely, Garfield, 4th St., 6th St., 7th St., 9th St., and Atlantic Avenue. Garfield and Atlantic contain the largest traffic volumes. Valley Boulevard, Garfield and Atlantic are four lane streets, with two lanes each way, while the remaining cross streets are two-lane streets, with one lane each way. There are five traffic signals in place, one located at each of these major intersections. The Garfield and Atlantic intersections are the only intersections which contain left turn bays on the cross street. All intersections contain left turn bays on Valley Boulevard since there is an open lane in the middle of the artery.

All the minor cross streets, including 5th Street, have two-way stop signs facing the cross streets. Atlantic and Garfield Avenues have leading protected lefts without overlap on both the cross streets and artery. The other major cross streets have a two phase timing pattern, where the through lanes and lefts proceed together. The lefts are then classified as permitted, as opposed to protected, since they have to wait for the opposing through traffic.

ANALYSIS

Analysis of the Original System

Before increasing the volumes on 5th Street, a study was made of the existing system in order to evaluate the performance of the final system. In this system, 5th Street, a two-lane roadway with one lane each way, only contains a two-way stop sign on the cross street. The volumes are 59 vph northbound, 81 vph southbound, 743 vph westbound, and 1225 vph eastbound where Valley Blvd. runs East-West. The turning volumes are as noted in Table 1.

Once the data has been entered into TRANSYT-7F, the program is ready to run. The first run used in this analysis was a simulated-pretimed run used to check the performance of the existing system. The output showed acceptable delays per vehicle (DPV) on 5th Street of 12.6 sec/veh northbound and 8.9 sec/veh southbound. There was no DPV on the artery since the stop sign did not apply to the arterial traffic. The system wide DPV and performance index, PI, were 43.18 sec and 249.40, respectively.

The next run was pretimed-optimized. In this case there were no significant changes in DPV or PI at 5th Street. However, due to the optimized signal timing at the other intersections, the systemwide totals had changed considerably. The new values for DPV and PI were 20.06 and 270.53, respectively.

Analysis of Original System with High Volumes

The original system with low volumes was seen to perform without excessive delays. The traffic flow volumes were increased to the predicted values associated with the proposed construction. A value of 230 vph was chosen since it matched the volumes of the other two lane cross streets. Also, according to the Manual on Uniform Traffic Control Devices (MUTCD), if the volume on an artery of two or more lanes each way exceeds 600 vph for an eight hour period, and the cross street of one lane in each direction exceeds 150 vph for that same period, then the traffic signal is warranted at that intersection (2). Although it is warranted, the signal may not be installed.

The turning volumes were also adjusted to simulate an actual shopping mall's vehicular traffic. Through traffic from the cross street constituted 40 percent of the volumes, and the remaining 60 percent was split evenly among the left and right turners. In order to maintain the "volume screen" on the artery, the turning volumes were adjusted there as well. More vehicles turned onto the cross streets, which would also tend to simulate a real life scenario.

To prove the need for a traffic signal, there must be some excessive delays or dangerous conditions. After running TRANSYT-7F through a simulated-pretimed run with the higher volumes, the DPV on the cross streets did not experience much change. They went from 12.8 to 15.8 northbound and from 8.9 to 13.8 southbound, still within the acceptable range. However, the westbound left turners went from 0 to 384 sec delay per vehicle. The left turn bay became 120 percent over saturated; a condition which could lead to considerable backup and possible accidents. Saturation flow rate is the capacity of an intersection approach lane as if the signal were always green and the flow of vehicles were never stopped (2). TRANSYT-7F manipulates this rate to account for the time the signal is red to arrive at the percent saturation.

The optimized-pretimed run did not give any better results at the 5th Street intersection. However, the total system delay per vehicle and Pi had gone down from 43.95 to 21.32 sec and from 265.5 to 177.68, respectively, as would be expected in a optimized run. Yet the intersection performance was still considered unsafe and within the MUTCD warrant. Therefore, a traffic signal will be needed at the intersection of 5th Street and Valley Boulevard.

Analysis of the New System

The new system now contains a traffic signal at the intersection of 5th Street and Valley Boulevard with the
TABLE 1. TRAFFIC FLOW VOLUMES AT EACH INTERSECTION IN VEHICLES PER HOUR

<table>
<thead>
<tr>
<th>INTERSECTION</th>
<th>North</th>
<th>South</th>
<th>East</th>
<th>West</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1371</td>
<td>791</td>
<td>1201</td>
<td>823</td>
</tr>
<tr>
<td>2</td>
<td>100</td>
<td>180</td>
<td>1276</td>
<td>830</td>
</tr>
<tr>
<td>3</td>
<td>437</td>
<td>520</td>
<td>1311</td>
<td>777</td>
</tr>
<tr>
<td>4</td>
<td>69</td>
<td>81</td>
<td>1225</td>
<td>743</td>
</tr>
<tr>
<td>5</td>
<td>87</td>
<td>105</td>
<td>1273</td>
<td>762</td>
</tr>
<tr>
<td>6</td>
<td>1105</td>
<td>1136</td>
<td>1190</td>
<td>911</td>
</tr>
</tbody>
</table>

volumes remaining at 230 vph each way on the cross street. The analysis from a simulated-pretimed run shows that, due to the signal, the DPV had been evenly distributed throughout the intersection, and the excessive DPV that existed on the westbound left lane had been drastically reduced from 384 sec to 30.5 sec. The PI for the intersection also increased from 10.5 to 17.8. This can be explained by the fact that a signal causes all lanes to be delayed due to lost start up times - the time it takes a vehicle to react to a change from red to green, clearance lost time, the time between the last vehicle from one approach entering the intersection and the initiation of the green signal for conflicting movements, and actual stopping at red signals (2).

Of particular importance is the time-space diagram shown in Figure 3. Note the band discussed in the introduction which represents the path of vehicles through the artery. It was plotted with an average speed of 35 mph, with the x-axis as time and the y-axis as distance. As can be seen in this diagram, a certain amount of vehicles can pass through the artery with excellent progression. The exact number of vehicles that can pass through can be calculated as follows: bandwidth (sec) * # of lanes * veh/sec/lane.

The optimized-pretimed output did not significantly change the intersections performance, as in the other cases. Only the left turn DPV was decreased from 30.5 to 17 sec. Again, the systemwide totals for DPV and PI were reduced significantly from 43.95 to 21.16 sec/veh and from 265.54 to 175.58, respectively. A considerable change could also be seen in the time-space diagram in Figure 4. Whereas before the band progressed straight through the artery, the band was now broken in the middle of the artery. This highlights a familiar trait with TRANSYT-7F. Unlike PASSER-II, it does not look to maximize bandwidth and progression, but rather minimize the PI value.

As mentioned earlier, PASSER-II combined with TRANSYT-7F will usually give the best results. In this case, the data for the new system was entered into PASSER, and the signal timings generated in PASSER-II's optimized mode were then input into TRANSYT-7F. The resulting optimized-pretimed run gave system wide results that were slightly lower than TRANSYT-7F working alone. The new system wide DPV and PI were 20.3 sec and 172.83, respectively.

A summary of all the systemwide and local intersection totals for the different stages can be seen in Figures 4-7.

Varying Geometry And Percent Volumes

Another interesting sensitivity study of the system is to move 5th Street to different locations along the artery without changing the volume. It was moved a total distance of 450 ft. between 6th and 4th Streets. The fuel consumption values did not show much of a variance. Also, the values for DPV and PI did not change at all, therefore; no optimal location can be obtained from these results. I also varied the location of 6th Street, assuming 5th Street was not signalized, and obtained similar results.

Finally, different percent volumes of 5th Street were taken and graphed. In this case, optimized-actuated and optimized-pretimed runs were taken for each percent volume. In the graph of percent volumes vs. system PI (see Figure 6), the relationship was directly linear and the values for actuated were almost identical to those for pretimed. In the graph of percent volumes vs. system DPV (see Figure 7), there was no change in DPV until
FIGURE 2. TIME-SPACE DIAGRAM FOR TRANSYT-7F SIMULATED-PRETIMED RUN
FIGURE 3. TIME-SPACE DIAGRAM FOR TRANSYT-7F OPTIMIZED-PRETIMED RUN
FIGURE 4. SYSTEMWIDE DPV VALUES AT DIFFERENT STAGES

FIGURE 5. SYSTEMWIDE PI VALUES AT DIFFERENT STAGES
FIGURE 6. 5TH STREET PI VALUES AT DIFFERENT STAGES

FIGURE 7. 5TH STREET DPV VALUES AT DIFFERENT STAGES
after 60 percent volume when the relationship became directly linear. Again there was very little difference between the actuated and pretimed outputs.

CONCLUSIONS

In this project there were two main objectives. To assess the sensitivity of the system to the proposed shopping mall, and to assess the sensitivity of the computer models, TRANSYT-7F and PASSER, to various changing inputs.

System

In regards to the former objective, Figure 4 shows how the system DPV changed at different stages. It is only necessary to look at the simulated outputs since these are manipulated to a lesser extent by TRANSYT-7F than the optimized runs. Looking at the simulated runs and comparing the high volume with a signal to the low volume without a signal outputs, the DPV rose slightly from 43.18 to 43.95. This is understandable since a "volume screen" was set up around the given intersection and the rest of the system would only be affected by the added signal, not the increased volume. The stop time and lost time associated with the signal caused the slight increase in DPV.

The graphs of the other systemwide values depict the same trends. In the PI graph, Figure 5, there is a slightly larger increase, from 258 to 265, compared to the DPV graph. This can be explained by the fact that PI is total stops plus total delays.

The isolated intersection values at each stage show a large increase in the same two outputs, Figures 6 and 7. This seems reasonable considering that the volumes are changing throughout the intersection.

Models

In addressing the second objective's results, namely the sensitivity of the computer models, there are three trends that arise. First, the combination of PASSER-II and TRANSYT-7F in the last analysis, a signal with high volume, gives the best results in the optimized outputs for all the systemwide values. However, in the case of the isolated intersection, the values get worse. The PI and DPV values for each intersection change after being optimized when combining the models. For some discernible reason, some improve and others worsen with the net effect being a lower system PI and DPV. Second, TRANSYT-7F does not differentiate very well between actuated and pretimed control. This is exemplified by the percent volume variation graphs previously discussed. Actuation is based upon random theory and is very difficult to predict. Third, TRANSYT-7F did not respond well to changes in location. If the "volume screen" had not been implemented, then the results may have been different.

RECOMMENDATIONS

System

System operators should make the following changes as required: First, the timing plans of TRANSYT-7F should be installed to see if the system performance improves. At the present moment, the system is being timed to provide progression, but this may not be the optimal solution. Second, the existing dimensions of 5th Street may have to be changed if the new volumes exceed expectations or if the computer models did not accurately reflect the real-life conditions. Finally, if the through traffic across Valley Boulevard at 5th Street is extremely high, an overpass for through traffic would drastically improve the arterial performance.

Models

The new user of the before mentioned computer models should first familiarize himself or herself with the manuals and become well versed with traffic engineering terminology. Second, learning how to enter data using the card format is very useful for quick error assessment and a better understanding of the data entry process. However, programs such as McT7F and AAP can quicken the process. Third, using the computer models under different circumstances will improve the understanding of its uses and its capabilities.

The program itself is a very powerful tool for traffic engineering, yet it needs improvement in the area of actuation, which may require the development of a whole new model. Also, the program needs better computer interfacing capabilities. Reprogramming the model in C Language may be the only solution. Third, the program should contain a help menu to facilitate the use of the model and minimize the use of the manual.

The manuals were very helpful in creating the input file and understanding the various terms associated with TRANSYT-7F. The manual prepared by the University of Florida was very informative since it described each card entry in detail (3). However, it should have a quick reference guide in the back or front, similar to Texas A&M's manual.

REFERENCES


**ACKNOWLEDGEMENTS**

I would like to thank Dr. Carroll Messer for his support and thoughts throughout this project. I would also like to thank Dr. Daniel Fambro for his effort in making us feel at home in a far away place. Special appreciation is extended to Joseph Koothrappally and Kenneth Vaughn, the graduate students who helped me along the way. Finally, I would like to acknowledge Los Angeles County Department of Transportation for the use of their data.
Microscopic Analysis of Fiber Modified Asphalt Concrete

DEVON WILLIAMS

Three types of fiber-modified asphalt concrete pavement were examined using a scanning electron microscope, each containing either cellulose, mineral or nylon fibers. Two of the samples were received from highway sections and one was produced in the lab.

The photographs resulting from the examination showed that the nylon fibers tended to form large bunches in the mix, whereas the mineral and cellulose fibers appeared to be able to reinforce the binder film coating the aggregate.

Photographic analysis suggests that mechanical testing should be done to determine the response of fiber-modified asphalt concrete to cyclic repeated loads, freeze-thaw damage, and crack propagation.

INTRODUCTION

Fiber additives are one of the most recent developments in the United States to improve asphalt concrete pavement performance. They are used extensively in Europe in asphalt-rich "stone-mastic" mixtures and porous friction course mixtures. Fibers are available in many forms and from several firms in the market.

The broad range of fiber additives available for use in asphalt concrete have not been adequately studied and investigated to determine their effects on pavement performance. Fiber additives are supposed to allow an increase in asphalt content compared to conventional asphalt concrete (1). This should create a thicker asphalt cement coating on the aggregate which is reinforced by the fibers (2). The augmented coating should provide improved resistance to cracking, freeze-thaw damage, and aging. In a standard, non-fiber asphalt concrete mix, the greater asphalt content would induce extensive rheological deformation leading to shoving and rutting. However, the mixtures which normally utilize fibers have special aggregate gradation which can accommodate the high binder content without deformation. In fact, the most attractive feature of these mixtures is the ability to accommodate high binder contents without producing rutting.

Despite high expectations, some fiber modified mixes perform reasonably well in service whereas others do not. While the action of the fibers in the mix has been modeled and discussed, the exact mechanisms which affect performance of these materials are unknown. The minute size of the fibers, their dispersion and the degree of coating within the mix make it difficult to determine fiber behavior within an asphalt concrete mix. It is now possible to perform analysis on a microscopic scale. This may provide a better understanding of macroscopic events.

OBJECTIVES

The primary goal of this project is to hypothesize the micromechanic behavior of fiber additives in asphalt materials. This is an important first step in researching and formulating an accurate model of fiber-asphalt mechanics.

The project also seeks to compare the mix microstructures created by the inclusion of different fibers. This should help explain why some fibers perform well while others fail.

BACKGROUND

The three fiber materials used to modify the asphalt concrete samples in this study are each produced exclusively for use in asphalt concretes and are currently available on the market. They are a cellulose fiber, a mineral fiber, and a nylon fiber. Asphalt concrete samples containing cellulose fiber were produced in the lab at Texas A & M University. The mineral-modified samples came from a section of I-85 near Atlanta, Georgia, and the nylon-modified samples are from I-70 near Glenwood Springs in Colorado.

Several studies have been concluded on the mechanical properties and performance of various fiber
mixtures. They concentrated on macroscopic characteristics and comparisons to non-fiber mixtures. These reports contain limited information on the shape and size of the fibers involved. The sales literature for some of these products uses microscopic photographs of fibers and artists' conceptions of the mixed product to help customers visualize their descriptions of its effectiveness. However, there is little information showing any visual evidence of the fiber-asphalt concrete interaction.

One way to visually evaluate interaction of the fibers with the asphalt mixture is through electron microscopy. Electron microscopes afford the researcher many advantages pertinent to this study, such as outstanding magnification and resolution. These instruments can also accommodate clear image photography without the shallow field of view limitations inherent with light microscopes (3). This image depth is particularly valuable because it allows the entire image to be focused despite morphological differences, thereby making it possible to better observe aggregate "peaks and valleys" and fibers sticking out of the observed surface without any blurred regions.

Two common types of electron microscopes available are the scanning electron microscope (SEM) and the transmission electron microscope (TEM). The TEM will accept a sample with a maximum thickness of 500 angstroms, therefore the microscopic analysis of fiber modified asphalt concrete was limited to the SEM.

The SEM used in this project was a JEOL model T330A located at the Electron Microscopy Center at Texas A & M University. In SEMs, an electron gun directs a beam onto a sample within an evacuated chamber, and strategically placed detectors collect the scattered electrons as they leave the sample. This collected signal is processed through a video amplifier and then sent to a cathode ray tube. This signal is used to mimic the sample image on a television screen. This image can also be sent to a photographic apparatus which can produce either instant photographs or black and white negatives. The accurate, detailed views obtained with SEMs has made them a widely-accepted mainstay of scientific research for decades (4).

RESEARCH APPROACH

The sample cores were four inches in diameter. To prepare them for microanalysis, they were reduced to a manageable size. The microscope requires a sample no larger than three-quarters of an inch square, and although no set guidelines were given to us by the microscope technicians, a sample height of about a half-inch was produced. A masonry saw was used to cut the frozen cores in half longitudinally. A slice between three-quarters and a half-inch thick was cut. This slice was then cut into strips, also between three-quarters and a half-inch wide. The actual thickness was a function of how close the technician wished to place his hands to the blade.

These strips were then placed in a freezing room for several hours before the next step, in which two options were examined in this project to prepare the faces that would be examined with the scope. Both fractured and cut surfaces were used.

The fractured surfaces were created simply by cooling part of the strip piece with liquid nitrogen and striking it with a small hammer. Broken faces show areas of aggregate tearout and surface coatings present in the sample. The greater surface area and tearout holes also permit easier location of fibers with the scope to confirm the size, presence, and shape of the fibers. This preliminary view was valuable in finding the fibers in the cut-faced samples. The exposed fibers were more visible on the fractured faces, and these views are useful in determining the best magnification level to use and what shapes to look for when making the more difficult examinations on the smooth-faced specimens.

The fracture surfaces gave unexpectedly sharp views of the coated fiber in the mix; however, it was difficult to ascertain the binder coat thickness in most cases. These types of pictures serve more of a visual reference than a measuring device.

The smooth face samples were prepared with a diamond blade saw. This instrument has a diamond-encrusted blade and is used for fine, precision cutting. It prepared the thin slices necessary for the small vacuum chamber of the microscope. The saw is cooled with water and operates at low speed; reducing fiber and binder slippage along the surface of the cut. The sharp, hard blade and its low-speed, low-vibration action virtually eliminated grooves left by the blade and prevented aggregate tearout.

The flat, smooth surface with intact aggregate reveals the placement of fibers in the mix along the aggregate-binder interfaces and among the distributed aggregate particles. A two dimensional surface presents a clear picture of any uneven distribution or placement of the fibers in a representative cross-section.

Once the samples had been cut or fractured to the applicable dimensions, they were mounted on metal disks (viewing stages) and a layer of gold-palladium plating was electro-deposited on the observed surface. Without adequate electrical conductivity, a charge builds up on the
surfaces of the sample, which causes the irregular emission of electrons. This irregular flow distorts the image and hampers the ability to distinguish among the characteristics of the object's surface. Plating nearly eliminates this effect without altering the morphology of the asphalt sample. The coating is applied in thicknesses of 200-300 angstroms and is shiny gray in color. This color change is not important since electron microscopy provides no color information.

During examination, three types of output can be received: monitor screen, polaroid instant photograph, or photographic negative. The monitor is used for quick scanning and changing magnifications. It was used to search for the fiber regions in this project. When a good image is located, a picture was taken. A polaroid can be taken for instant feedback; this is valuable when a more detailed image is needed than seen on the monitor. The photographic scanning operation is precise and slower than the scanning for monitor output, producing a continuous image. Photographic negatives are prepared with the better images for data that can be reproduced by contact printing of photographs. These pictures had the optimum finish quality and can also be used to make excellent slides.

Three general subjects were photographed: regions between aggregate, fiber/binder interfaces, and the fibers themselves, especially those located in apparent clumps or asphalt-aggregate voids. Procuring a telling photograph typically required searching samples for one or two particular conditions, then deciding which image was the best. The viewing stage could be adjusted by changing viewing distance, position, and angle. This procedure offers many options, especially with the significantly smaller cellulose and mineral fibers.

The fibers are easily identified in the finished images. Their relative magnitude as compared to the other mix components is also apparent. This is aided greatly by the measurement marker on each photo micrograph. After positive identification of individual fibers is accomplished, the magnification can be reduced and larger scale features of the fiber system can be analyzed. These subjects include the fiber clumps and behavior in voids. Orientation of the fibers is also important and visually apparent as are the aggregate edges, which can be examined for unequal or unusual fiber distribution.

RESULTS

The data received are a series of micro-photographs. A representative group of these photographs appears in the appendix and is discussed below.

Photo 1

This is a sample of cellulose fibers at 500 times magnification. The irregular ribbon shape may allow for better coating by the binder.

Photo 2

A view of the plain mineral fibers, also magnified 500 times, shows their regular rod shape and small diameter. These fibers looked stiff and non-flexible in the mix, whereas the cellulose fibers appear flexible and tortuous.

Photo 3

This closeup of a cellulose fiber-modified asphalt concrete sample shows good binder reinforcement. This occurs around aggregate, and the fiber seems to fit fairly well within the binder film which coats the aggregate.

Photo 4

This individual cellulose fiber demonstrates the potential benefit of its ribbon shape. The asphalt seems to gather in the folds of the fiber, and this accumulated asphalt may improve the adhesion of the fiber within the film.

Photo 5

The small, stiff mineral fibers in this photograph are embedded in the asphalt coating around an aggregate surface. Individual fibers are evident at the left center, center, and center right edge. Once again, the small mineral fibers appear to fit relatively well in the asphalt film which encapsulates the aggregate.

Photo 6

This closeup of nylon fibers within a mixed core reveals their comparatively large size with respect to the other fibers. There also appears to be interaction among the fibers, creating the intertwined appearance. As opposed to the mineral and cellulose fiber, the nylon fibers are quite large with respect to the asphalt cement film coating the aggregate. Thus, they have a greater potential to actually interfere with the asphalt-aggregate bonding process. This could be a problem with regard to accumulated damage in cyclic loading or durability.

Photo 7

Reducing the magnification of photo 6 shows the fibers grouped into a sizeable clump which is absent of aggregate.
Photo 8

With the magnification further reduced in the same area, the relative size of this fiber construction is roughly equal to or larger than that of some of the neighboring fine aggregate. This suggests that the nylon fibers may act as an odd-shaped aggregate.

Photo 9

This photograph of a different aggregate boundary region within a nylon-modified sample shows no fibers in a region somewhat larger than that in photo 8. These two images display the spotty distribution evident throughout the nylon-modified samples.

The photos in this report indicate trends which were noticed throughout the research. The nylon fibers were characterized by their large size, tendency to group together, and interaction with each other. The nylon fibers simply did not appear to reinforce the asphalt coating, but rather to behave like aggregate. The cellulose and mineral fibers appeared to remain within the binder coatings around the aggregate, due in part to their substantially smaller size.

CONCLUSIONS

Photographic data may suggest future research and provide a valuable interpretational basis for other information, it does not alone prove or disprove the effectiveness of materials in service. However, this research has suggested the following:

1. The nylon fibers do not reinforce the binder film coating the aggregate. The rope-like constructions they create in the mix could be significant weak points.

2. The cellulose and mineral fibers appear to be coated acceptably well with asphalt and well enough dispersed in the mix to reinforce the film to a degree without introducing a larger asphalt demand.

3. Mechanical testing should be done to determine whether the reinforced film of the cellulose fiber samples offers any benefits pertaining to cracking-resistance, aging-resistance, and water resistance without significant additional rheological deformation.

The role of fiber in stone mastic and porous friction coarse mixtures is primarily to allow for higher binder contents. These binder contents can be accommodated by the mix in practice, but usually not during construction. The fiber is hence a carrier during construction. In order for the fiber to perform in an acceptable manner following construction, it must not detract from the mechanical and/or durability properties of the mix. This research indicates that some potential for the fiber to detract from desired mixture mechanical and durability properties may exist if the fibers are large (with respect to the binder film) and poorly dispersed.

RECOMMENDATIONS

Moisture conditioning tests should be used to determine the durability of the fiber modified asphalt concretes. This test would not only determine the fiber-reinforced coatings' resistance to damage, but also could look for any water channeling effects of the nylon bunches.

Repeated stress permanent deformation tests should be used to determine the relative cyclic damage potential of the fiber-modified materials. The samples would be subjected to cyclic haversine-wave stresses which simulate the stresses of service. Saturated samples may be used to investigate strengths or weaknesses induced by water. A minimum of 30,000 to 40,000 cycles would be necessary.

Localized fracture testing should be used to examine crack resistance. Cracking would be introduced to the sample with a cut notch. Stress would be concentrated on the fracture point, and crack growth in the process zone would be recorded through a magnification lens with a video camera. The laws of fracture mechanics would be used to analyze the results.

REFERENCES


Biographical Data
Kent Michael Collins was born in Dallas, Texas on February 16, 1971. Kent grew up in DeSoto, Texas, where he received his primary and secondary schooling. After graduating from DeSoto High School in 1989, Kent entered Texas A&M University. He will receive his bachelor's degree in Civil Engineering in December, 1993.

Mr. Collins has been employed by Cardenas-Salcedo and Associates Civil Engineering firm and by the Texas Department of Highways and Public Transportation, now the Texas Department of Transportation, for the past two summers. During those summers, he gained practical experience within the field of Civil Engineering and developed interests both in Structural and Transportation Engineering.

Kent is currently employed by the Texas Transportation Institute where he received a fellowship for the summer of 1992. During the fellowship, he was employed as a Research Assistant.

Kent is a member of Chi Epsilon Civil Engineering Honor Society. Upon graduating, Mr. Collins plans to attend graduate school at Texas A&M University and pursue a Master's degree in Transportation Engineering.

Brian Patrick Cronin was born in Washington D.C. on December 7, 1971. He lived in College Park, Maryland for one year then moved to Silver Spring, Maryland. Brian grew up in Silver Spring and attended primary and secondary schools in the Montgomery County School system. He graduated from Albert Einstein High School in June 1989. In August of 1989 he entered the College of Engineering at Virginia Polytechnic Institute and State University and will receive his undergraduate degree in Civil Engineering in May, 1993.

Mr. Cronin received an undergraduate transportation engineering fellowship at Texas A&M University for the summer of 1992. During this fellowship, he was employed as a Research Assistant with the Texas Transportation Institute.

Brian is a member of the Golden Key National Honor Society, Chi Epsilon Civil Engineering Honor Society, and the American Society of Civil Engineers. After graduation, Mr. Cronin plans to pursue a career in Transportation or Structural Engineering, before attending graduate school to obtain a master's degree in Civil Engineering.
James Lee DeSanto was born in Athens, Ohio on January 1, 1971. He grew up in Ashland County, Ohio, where he attended primary and secondary schools. He graduated from Ashland High School in June 1989. In September of that year he entered the College of Engineering at Ohio University and will receive his undergraduate degree in Civil Engineering in March 1994.

In September, 1990 James entered the cooperative education program at Ohio U. He was employed by the Goodyear Tire and Rubber Company in Logan, Ohio, as a co-op mechanical engineer. During the 1991-92 academic year, James was employed by Ohio University to conduct help sessions for students enrolled in Civil Engineering Statics classes. James received an undergraduate transportation engineering fellowship at Texas A & M University during the summer of 1992 and was employed as a Research Assistant with the Texas Transportation Institute.

James is a member of Theta Tau, the national professional engineering fraternity. He is also a member of the American Society of Civil Engineers. After graduation, Mr. DeSanto is planning to pursue a masters degree in a field of civil engineering but has not yet decided on specific course work or on the university he wishes to attend.

Glen Alden Hanks was born in West Hartford, Connecticut on July 30, 1970. Glen attended West Hartford Public Schools and graduated from William H. Hall High School in June 1988. In September 1988 he enrolled in the University of Delaware's civil engineering program. He plans to receive his undergraduate degree in December 1992.

Glen has been involved in several design projects while at the University of Delaware. He has designed foundation systems for a 22,000 and 40,000 square foot warehouse and office building facilities. He has also designed a 100 ft. span concrete pedestrian bridge and been involved in the location and design of a three mile section or roadway.

Mr. Hanks is student member of the American Society of Civil Engineers and the Institute of Transportation Engineers. He is also a member of Alpha Phi Omega, National Service Fraternity. After graduating, Glen, plans on pursuing a masters degree in civil engineering.
Amy Rebecca Kohls was born in Fort Knox, Kentucky on March 16, 1972. She lived in Paris, Texas, before moving to Blacksburg, Virginia, in January 1986. She graduated from Blacksburg High School in June 1989. In August of 1989, she enrolled in the college of engineering at the Catholic University of America in Washington, D.C. She will receive her undergraduate degree in Civil Engineering in May 1993.

Ms. Kohls entered into a cooperative education program with the Federal Highway Administration and Catholic University in May 1991. She is employed by the FHWA Design Concepts Research Division at the Turner-Fairbank Highway Research Center in McLean, Virginia. Ms. Kohls received an undergraduate transportation engineering fellowship at Texas A&M University for the summer of 1992. While employed as a Research Assistant at the Texas Transportation Institute at TAMU, she continued to build upon the work done with FHWA.

Ms. Kohls is a member of the American Society of Civil Engineers, Phi Eta Sigma Honor Society, and the College Republicans. Ms. Kohls eventually wants to pursue higher level degrees in Transportation Engineering. However, she has not yet decided on her plans for immediately after graduation. She is presently looking at graduate schools but has not focused on one university she would like to attend.


Currently, Greg is a Training Manager to engineering students at Colorado State University computer labs. Greg received an undergraduate fellowship at Texas A&M University for the summer of 1992. During his fellowship, he was employed as a Research Assistant with the Texas Transportation Institute.

Greg is very active within the College of Engineering at Colorado State University. He is currently President of the Engineering Legislature, and has been active in the American Society of Civil Engineers. He also spent a year as an Associate Senator for the Associated Senate of Colorado State University. Greg is currently receiving the Presidents Merit Non-Resident Scholarship, as well as the Jim Murray Scholarship sponsored by the American Society of Public Works. After graduation, Mr. Krueger is planning to attend graduate school in Transportation Engineering.
Marty T. Lance was born in Greenwood, South Carolina on the 13th of April 1968. She graduated as high school salutatorian from Cambridge Academy located in Greenwood. Upon graduation in 1986, Marty was admitted to the North Carolina State University on an Air Force ROTC Scholarship.

In 1987, she entered a dual degree program at Lander College from which she earned her first B.S. degree in Mathematics. While at Lander College, Marty was awarded the Cincinnati Milacron Scholarship and the Math/Engineering Award. She also participated in college plays and performed with the Lander Dancers.

Marty will be receiving her second B.S. degree in the field of Civil Engineering from Clemson University in December, 1992. At Clemson, she is a member of Chi Epsilon and ASCE. She is also recipient of the American Public Works Scholarship. While pursuing her engineering degree at Clemson she utilizes her math background by teaching Skill Enhancement classes in industry through the Governor's Initiative for Work Force Excellence. These courses include Algebra and GED Preparation.

Mark Luszcz was born in Philadelphia, Pennsylvania on February 5, 1971. He lived in Palmyra, New Jersey until he was seven when his family moved to Hockessin, Delaware. He graduated from Alexis I. duPont High School and began college as a Chemical Engineering major at the University of Delaware in 1989. Realizing his error, he changed his major to Civil Engineering in which he will receive his undergraduate degree in May of 1993.

Mark received an undergraduate transportation fellowship at Texas A&M for the summer of 1992. He was employed as a Research Assistant with the Texas Transportation Institute.

Mark is an officer in Tau Beta Pi Engineering Honor Society, the Delaware Undergraduate Student Congress, Kappa Delta Rho Fraternity and the Off Campus Student Association. He is also a member of Chi Epsilon Civil Engineering Honor Society, the American Society of Civil Engineering, Golden Key National Honor Society and the University of Delaware Marching Band. After graduation, Mr. Luszcz is planning on attending graduate school for either Transportation or Structural Engineering.
Ronald Lewis Nowlin was born in Rankin, Texas on June 18, 1970. He grew up in Rochelle, Texas and graduated from Rochelle High School in May, 1988. In August of 1988, he entered Tarleton State University enrolled in pre-engineering. Lewis enrolled in the College of Engineering at Texas A&M University in August 1990. He will receive his undergraduate degree in civil engineering in May 1993.

Mr. Nowlin began working for the Texas Department of Transportation in the summer of 1988 and continued working there for the following three summers. He worked for the Brady office the first three summers and moved to the Brownwood office the last summer. For the summer of 1992, Lewis received an undergraduate transportation engineering fellowship at Texas A&M University. During this time, he was employed as a research assistant with the Texas Transportation Institute.

Lewis is a member of the Golden Key National Honor Society, Tau Beta Pi Engineering Honor Society, Chi Epsilon Civil Engineering Honor Society, and the American Society of Civil Engineers. After graduation, Mr. Nowlin is planning on attending graduate school in transportation engineering at Texas A&M University.

Dale L. Picha was born in Waco, Texas on December 9, 1969. He grew up in Waco and graduated from Reicher Catholic High School in 1988. He attended Tarleton State University majoring in Pre-Engineering in September of 1988 and transferred to Texas A&M University, enrolling in the College of Engineering, in September of 1990. He will receive his Bachelor of Science Degree in Civil Engineering in May 1993.

Mr. Picha was employed by the Texas Department of Transportation in the District Bridge Design Office in Waco, Texas, during the summers of 1989-1991. He has also been employed as a Research Assistant with the Texas Transportation Institute since January of 1992. Mr. Picha received an undergraduate transportation engineering fellowship at Texas A&M University for the summer of 1992.

Dale is a member of Chi Epsilon, Tau Beta Pi, the Institute of Transportation Engineers, and the National Dean’s List. He has been selected for the Distinguished Student Award at Tarleton State University and at Texas A&M University and is the recipient of the Chevron U.S.A. Scholarship in Civil Engineering and the D.C. Greer Highway Engineering Scholarship. After graduation, Mr. Picha will marry Ms. Tracy June Nagel from San Antonio, Texas, and will pursue a masters degree in Transportation Engineering at Texas A&M University.
Janet Ricci was born on November 24, 1971 in the Bronx, New York. Nearly five years later, her family moved to Franklin Square, New York, on Long Island. She attended H. Frank Carey High School and later transferred to Sewanhaka High School from which she graduated in June, 1989. Janet received a full scholarship to The Cooper Union for the Advancement of Science and Art in New York City, and began attending classes September 1989. She will receive her undergraduate degree in Civil Engineering in May 1993.

In May of 1991, Janet began working as a student researcher at the Cooper Union Infrastructure Institute. In August 1991, she assisted in writing and researching the Institute's first publication, "The Age of New York City's Infrastructure." Janet received an undergraduate transportation engineering fellowship at Texas A&M University for the summer of 1992. During her fellowship, she was employed as a Student Technician with the Texas Transportation Institute.

Janet is a member of the American Society of Civil Engineers, the Society of Women Engineers, and the Delta Eta Social Sorority. After graduation, Ms. Ricci is planning on attending graduate school at an as-of-yet undetermined graduate school.

Tricia Thomason was born in Fresno, California on November 10, 1969. She grew up in Fresno and graduated from Fresno High School in June of 1987. She attended Kings River Community College where she received an Associate of Arts degree in General Education. In August of 1989, she enrolled in the School of Engineering at California State University in Fresno and will receive her undergraduate degree in Civil Engineering in May 1993.

In May of 1990, Tricia began a student assistant position with Caltrans. During vacations, she worked on a survey crew in District 7 at the Camarillo office. Currently, Tricia has been working year around at the Fresno Right-of-Way Engineering office in District 6. Tricia received an undergraduate transportation engineering fellowship at Texas A&M University for the summer of 1992. During her fellowship, she was employed as a research assistant with the Texas Transportation Institute.

Tricia served as the president of the Engineering Student Joint Council in 1992 and the president of the Society of Women Engineers in 1991. She is an active member in the American Society of Civil Engineers and just received a scholarship from the local chapter. After graduation, Ms. Thomason is planning on attending graduate school in either Transportation or Environmental Engineering.
Joe Van Arendonk was born in Baghdad, Iraq, on September 8, 1965, and lived in Indonesia for five years before moving to New York. Joe grew up in Queens, New York, and graduated from The Kew Forest School, a local high school, in June 1983. In August of 1983, he entered the University of Massachusetts at Amherst in the field of mechanical engineering. In August of 1985 he entered Fordham University in the Bronx, New York City. In May of 1988, Joe received his undergraduate degree in Political Science from Fordham University.

Joe then worked for Bankers Trust Co. as a collector until August of 1989 when he entered City College in Manhattan in the field of civil engineering. In August of 1990 he transferred to S.U.N.Y. at Buffalo in the same major. Joe will be graduating from S.U.N.Y at Buffalo in May of 1993.

Mr. Van Arendonk worked for the New York State Department of Transportation as an inspector on the Brooklyn Bridge in the summer of 1991. He received an undergraduate transportation engineering fellowship at Texas A&M University for the summer of 1992. During his fellowship, he was employed as a Research Assistant with the Texas Transportation Institute. Joe is a member of Tau Beta Pi, and is on the Dean’s List at S.U.N.Y. at Buffalo. After graduation, Mr. Van Arendonk is planning on attending graduate school but has not decided on the university he would like to attend.

Devon Andrew Williams was born in Emporia, Kansas, on May 1, 1971. He moved several times in his youth and graduated from Parkway West Senior High near St. Louis in June 1989. In August of the same year, he entered the College of Engineering at the University of Missouri-Columbia and will receive his undergraduate degree in Civil Engineering in December 1993.

Williams received an Undergraduate Transportation Engineering Fellowship at Texas A & M University for the summer of 1992, and was employed as a Research Assistant at Texas Transportation Institute.

Devon is a member of the American Society of Civil Engineers, the Chi Epsilon Civil Engineering Honor Society, and the Tau Beta Pi Association. After graduation, he is planning on graduate studies in either Materials or Environmental Engineering, or attending law school.