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16. Abstract

The rate of deterioration of highway pavement in Texas over the years appears to have been accelerating. During this time, there has also been an observed increase in truck weights and sizes. This report is the first in a series regarding a study entitled "Evaluation of Truck Sizes, Weights, and Tire Pressures on Pavement Deterioration," being conducted by the Center for Transportation Research at The University of Texas at Austin. The overall study examines several aspects of possible cause and effect relationships between increasing truck weights, sizes, tire pressures and pavement deterioration.

The first phase of this study includes three scenarios. The first is a base scenario, which is characterized by an assessment of the effects on pavements of the entire vehicle fleet operating with currently prescribed weight limits and pre-1973 tire pressures. The second is an existing traffic scenario, characterized by the most recently-observed (1984) SDHPT vehicle weight data and tire pressures. The third is a future-traffic scenario, whereby hypothetical vehicle configurations are utilized to evaluate a possible way of reducing pavement damage. This report presents methods of data forecasting that are required for the overall study.

The second phase of the study focuses on the problem of pavement damage as related to axle configurations. Theoretical models are developed to relate axle configurations to pavement damage. ESAL values for a wide range of single axle weights are computed based on this theory. The results are compared with the AASHTO ESAL values. The model is then applied to the forecasting of pavement damage as a function of axle spacing in tandem axles and ESAL values are computed for different tandem axle profiles. The methodology is general and it may be applied to any truck axle configuration for determining its effect on the pavement with respect to a standard axle weight.

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by

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Research Project 3-8-86-378

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Texas
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by the

CENTER FOR TRANSPORTATION RESEARCH
THE UNIVERSITY OF TEXAS AT AUSTIN

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The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

There was no invention or discovery conceived or first actually reduced to practice in the course of or under this contract, including any art, method, process, machine, manufacture, design or composition of matter, or any new and useful improvement thereof, or any variety of plant which is or may be patentable under the patent laws of the United States of America or any foreign country.

SUMMARY

The rate of deterioration of highway pavement in Texas over the years appears to have been accelerating. During this time, there has also been an observed increase in truck weights and sizes. This report is the first in a series regarding a study entitled "Evaluation of Truck Sizes, Weights, and Tire Pressures on Pavement Deterioration", being conducted by the Center for Transportation Research at the University of Texas at Austin. The overall study examines several aspects of possible cause and effect relationships between increasing truck weights, sizes, tire pressures and pavement deterioration.

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The second phase of the study focusses on the problem of pavement damage as related to axle configurations. Theoretical models are developed to relate axle configurations to pavement damage. ESAL vales for a wide range of single axle weights are computed based on this theory. The results are compared with the AASHTO ESAL values. The model is then applied to the forecasting of pavement damage as function of axle spacing in tandem axles and ESAL values are computed for different tandem axle profiles. The methodology is general and it may be applied to any truck axle configuration for determining its effect on the pavement with respect to a standard axle weight.

IMPLEMENTATION STATEMENT

Implementations stemming from the predictions regarding axle weights are obvious. These numbers could impact not only the allocation of resources for maintenance and rehabilitations of pavement, but could also predict the viable pavement life due to future growth in both weighs and number of axles on the road. The procedures for the determination of ESAL values for tandem axle spacing have been mentioned in this study. However, the rationale for the procedures used have been mostly deleted for brevity. Readers interested in following up on the techniques used here to derive the ESAL values are referred to Ref 8 listed in the Bibliography.

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CHAPTER 1. INTRODUCTION

During the past fifty years, truck weights, sizes and tire pressures have consistently been on the rise. Pavement and geometric designs have been similarly upgraded in an attempt to accomodate larger trucks and heavier axle loads. This however, has in no way modified the more rapid than predicted deterioration of pavements. Rigid and flexible pavement designs and materials have been, and continue to be, evaluated in an attempt to to improve their load bearing capacities over their design life. It seems at this point that increasing tire pressures and axle loads have reached a stage at which it is not economically or realistically possible to accomodate such loads by upgrading the existing pavement system [Ref 6].

The increasing cost of fuel has forced the trucking industry to seek methods of increasing vehicular fuel economy. One such method is the reduction of rolling resistence which has led tire manufacturers to design and market both bias and radialply tires that operate at higher inflation pressures. The increased rutting and fatigue failures in asphaltic concrete pavements are suspected of being related to high truck tire pressures and heavier axle loads.

Axle loads are also critically important in pavement design and in the analysis of damage to pavements. The AASHO Road Test showed the relative effect of axle loads on pavement deterioration to be dramatic. For example a single axle load of 24 kips causes twice the pavement damage as a 20 kip single-axle load [Ref 6].

Another problem deals with determining pavement damage as a function of the variation of dynamic stresses and displacements in the pavement system with changes in axle configuration, vehicle velocity and the composition of the pavement cross-section. No previous attempt is made to relate these dynamic responses to the phenomenon of pavement damage due to fatigue.

There is a continuing pressure by members of the trucking and transport industry to increase allowable axle loads even further. The situation is complicated further by the fact that present Texas laws with respect to overloads are complex, but typically provide very small penalties for operators of overloaded vehicles. Further, a number of states have pending legislation to legalize longer and wider combination trucks. The effect of some of these newly proposed axle configurations on fatigue damage to pavements is completely unknown.

Rather than continue efforts to design pavements or develop materials capable of sustaining larger vehicle and axle weights, an attempt might made, instead, to identify the vehicle loads (weights and tire pressures) and axle configurations that can be reasonably and economically accomodated on existing and future highways.

BACKGROUND AND SIGNIFICANCE

There are several actions and events since the early 1970's that help explain why Texas is experiencing an acceleration of pavement deterioration, despite an on-going process of improving highway construction and maintenance procedures. These actions and events have been basically outside the control of the State Department of Highways and Public Transportation. They include the following :

- In 1974, Federal Legislation forced the states to increase gross-vehicle weight limits from 73,280 pounds to 80,000 pounds on interstate highways.
- Since 1973, prices of motor fuels have increased significantly and this, in turn, has had an impact on motor carrier competition.
- The Motor Carrier Act of 1980 mandated a de facto deregulation of the motor carrier industry by the Interstate Commerce Commission. The relaxation of entry requirements has resulted in a larger number of motor carrier firms, most of which are relatively small as compared to pre 1980 firm size. They have tended to concentrate on truck-less-than-truck load (LTL) service. Both of these changes may relate to vehicle overloading. New, small carriers may view overloading as an accepted method of reducing costs and/or rates in order to penetrate the market.
- The Surface Transportation Assistance Act (STAA) of 1982 authorized longer and wider trucks on Interstate Highways and required the states to designate additional highways to accommodate these vehicles.
- A major downturn of economic activity in the states and in the Nation which occurred in 1982 put severe financial pressures on all segments of the economy. The motor carrier industry, which had recently expanded, experienced a highly competitive environment. The number of trucks which have travelled overloaded may have increased as carriers have attempted to reduce costs since this time.
- The technology of tire manufacturing has advanced and even higher pressure truck tires will soon be available. Also, some truckers tend to over inflate tires in order to reduce rolling resistance and increase fuel mileage.

The actions and events cited above have all had an impact on highway facilities - some more than others [Ref 6].

OBJECTIVES

The primary objective of this study is to evaluate the effects of truck sizes, weights, and tire pressures on pavement deterioration. This is achieved through evaluation of the following three scenarios.

a) a base scenario characterized by the entire vehicle fleet operating within currently prescribed weight limits and pre-1974 tire pressures.
b) an existing-traffic scenario characterized by 1984 SDHPT weight-survey data along with vehicle tire pressure data from recent research studies.
c) a future-traffic scenario in which vehicle configurations are modified to redistribute the loads in an attempt to reduce pavement damage.

The three scenarios mentioned above imply rather different combinations of total traffic, vehicle gross weights and axle weight distributions. The existing traffic scenario consists of real survey information, for the selected sections including ADT's, percentages of trucks, vehicle classification and truck-weight distributions. The traffic data requirements for the base scenario are basically the same as for the existing-traffic, however, because it is an hypothetical scenario, traffic data assembly will require more effort.

In this report, the first two scenarios are studied. The Average Daily Traffic, truck-weight distribution and vehicle classification data are forecasted to the year 2005. These forecasted values provide a basis for evaluating the effects of truck sizes, weights and tire pressures on pavement deterioration and costs. Also included in this study is the calculation of "1974 equivalent number of trucks from 1984 load data", which gives a percentage comparison of 1974 and 1984 vehicles that could carry the net load carried by 1984 vehicles.

In addition, an attempt is made to determine the effect of axle configuration on pavement damage. It has been implicitly accepted in the AASHTO design equations that two axles placed close to each other so that they form a tandem axle group will cause less damage than if the axles were placed apart and treated as two single axles. For example, a tandem axle group of 36 kips is comprised of two axles, each having a load of 18 kips. The ESAL value for two single axles, each having a load of 18 kips is 2. However, the tandem axle group of 36 kips has an ESAL value of only 1.34. The placement of axles and their effect on pavement damage shall be investigated.

ORGANIZATION OF THE REPORT

Chapter 1 gives a brief description of the problems that arise due to the ever increasing weights of trucks and tire pressures, the effects of these factors and the damages that might result. The reasons for the overloading of trucks and further increase in tire pressures are discussed.

Chapter 2 deals with the previous studies conducted in Texas. The research areas now under study can be grouped into three general categories, which play an important part in the evaluation of the effects of truck sizes, weights, and tire pressures on pavement deterioration.

Chapter 3 describes the methodology involved in the selection of test cross-sections, which are in turn linked to Weigh-In-Motion stations. This linking is based of the vehicle classification.

Chapter 4 gives a brief description of the various techniques that can be used in the forecasting process. Basically there are three general methods of forecasting, namely: persistence, forces at work, and time series techniques. Each technique is described in detail.

Chapter 5 describes the forecasting of Average Daily Traffic, Truck Weights and Classification. It also includes the calculation of the 1974 equivalent number of trucks from 1984 load data.

Chapter 6 contains a brief summary of proposed methods to scale the effects of different cyclic loads. Theories are developed that would transform stresses and displacements to damage. A discussion on how to apply these theories is presented.

Chapter 7 presents the development and callibration of a damage model. Differences between the dynamic response of various axle configurations and axle weights are investigated and discussions presented. A relative damage scale for different axle configurations and weights is established.

Chapter 8 includes a summary of the entire report and a list of recommendations.

CHAPTER 2. PREVIOUS STUDIES IN TEXAS

As noted in Chapter 1, there has been a growing concern about the possible relationship between increases in vehicle sizes and weights and observed acceleration of pavement deterioration. The relative effects of changes in truck size and weights upon pavement deterioration has yet to be quantified. Hence, this study is one of several which has been conducted to evaluate the effects of truck sizes, weights and tire pressures on pavement deterioration.

Over the past decade, a number of studies have been conducted nationwide, and in Texas in particular,in an attempt to quantify the effects of changes in truck weights, sizes and tire pressures on pavement deterioration. Some of these studies have been reviewed to determine their applicability to the needs of this project. Of particular interest to our project is the study by C. Michael Walton, Chien-Pei Yu and Paul Ng [Ref. 22], which basically develops a truck-weight shifting methodology, whereby truck weight frequency distributions can be shifted (predicted) to a later year, as the result of changes in legal weights. Some of the studies that are reviewed were grouped into different research study areas as mentioned below.

The research areas can be grouped into three general categories:

(a) Traffic data collection and forecasting: Data collection, analysis and forecasting is imperative for a proper, systematic analysis of the effects of truck weights, volumes and sizes on pavement deterioration.

(b) Pavement modelling: Once data are collected regarding the volume and classification of traffic, and distributions of truck loadings, the information is translated into pavement damage effects in order to evaluate the impact on pavement design, maintenance and rehabilitation costs. Historically, models and procedures of American Association of State Highway Officials were used to predict and design for these damage effects. These procedures are still currently used, however, new information has been collected in Texas, which is more representative of environmental and traffic characteristics in this state.

(c) Highway system and truck operating cost analysis: Once the effects of various truck loadings are quantified, they are translated into cost effects. These cost effects are useful in planning, programming and policy development with regard to the laws governing the highway system.

In 1975, a study was conducted by Randy B. Machemehl, Clyde E. Lee, and C. Michael Walton [Ref 16], which reasserts the growing importance of the in-motion weighing system, and recommends a plan for implementing this system into the traffic survey program of the State Highway Department. The objectives of this report are three-fold. First, it presents an evaluation of the ability of an in-motion weighing system to predict static vehicle loads. Second, there is an analysis of existing weight data obtained by the SDHPT from static weighing stations and a determination of the overall level of sampling efforts needed to provide satisfactory estimates of vehicle weights. This also includes a study of timewise variations in vehicle weights, using data collected by the in-motion weighing system. Third, a comparison of the economics of static and in-motion vehicle weighing is provided. Some of the recommendations of the project are listed below.

(a) The number of weight survey sites could be reduced from 21 to 6. The 21 sites produce only six different weight frequency distributions and therefore represent a duplication of effort.
(b) The location and number of sites should be periodically evaluated to make allowances for future vehicle weight changes that might influence the number of survey sites and their locations.

In 1980, a study was conducted by Han-Jei Lin, Clyde E. Lee, and Randy B. Machemehl [Ref 13], whereby the procurement and distribution of traffic data, i.e., traffic volume, speed, vehicle classification and vehicle weight data, as performed by the Transportation Planning Division (D-10) of the State Department of Highways and Public Transportation was studied, and recommendations were made to satisfy some of the growing needs of the system. The study also recommended the replacement of the old 1969 in-motion weighing systems with modern equipment, greater utilization (or more consistent use) of the in-motion weighing system for procurement of more meaningful data, and the installation of more in-motion weighing systems to improve the adequacy of the truck weight survey program.

An improved truck weight data shifting methodology was developed in 1983 by C. Michael Walton, Chien-pei Yu, and Paul Ng [Ref 22] for the projection of future truck-weight distribution patterns. Maximum legal truck size and weight limits have always been a major issue of concern, since the assessment of impacts due to changes in maximum limits is a difficult problem. This, in turn, is due to the difficulty in effectively predicting future truck-weight distribution patterns as affected by changes in legal weight limits. This report introduces a procedure (that can be applied either manually or by computers) that will help predict future truck-weight distribution patterns to a future time period for two cases. One, where a change in legal truck weights is assumed, and two, where no change in the maximum truck weight limit is considered.

In 1984, a study was conducted by Kenneth J. Cervenka and C. Michael Walton [Ref 5], which was a review of the current Texas State Department of Highways and Public Transportation load traffic forecasting procedure and computer model (RDTEST68). The findings showed that the model was extremely sensitive to user-specified input parameters such as percent trucks and selection of representative weigh-in-motion stations. The report recommended better use of lane-wise traffic load distributions, greater opportunity for highway district level evaluation of input data, changing output to include average equivalency factors per truck for each highway segment, and annual preparation of statistical summaries of weight data for trend analysis. It additionally recommended changing traffic load forecasting procedures from one that uses axle weight data by station, to one that uses data by truck type, and expansion of the truck weight survey program. Implementation of these findings would provide much better truck traffic data.

A study was conducted in 1978 by J. L. Brown, D. Burke, F. L. Roberts, and C. M. Walton [Ref 3], which evaluated the effects of heavy trucks on Texas highways. This study was an economic evaluation of two scenarios of maximum gross and axle weights of trucks on the state highway system over a twenty year period. The basic findings of this study were that the economic benefits to the trucking industry of either weight scenario were greater than the costs to the highway system. The REHAB computer program which was utilized in this study, and which utilizes AASHO equivalency factors and Texas-based survivor curves has been superceded by the more-current survivor curves and equivalency factors which have been incorporated into new computer programs. Although this was a good early study of the increased truck weight problem, it's limitations were : (1) it used AASHO equivalency factors, (2) no quantification of bridge costs was made, (3) no evaluation was made of the increased size or tire pressures of trucks, (4) no off-state-highway system cost estimates were made, and (5) no considerations were given to changes in technology, operating characteristics, highway safety and modal shifts.

A study was conducted in 1983 by C. Michael Walton and Chien-pei Yu [Ref 23], in an effort to assist the transportation professionals in their policy making concerning motor vehicle size and weight limits. This report summarizes the then-current size and weight related activities in Texas and presents an analysis of oversize-overweight truck movements within the state based on existing available data. In order to study the economic effects of the current oversize-overweight truck movements, two cases were studied: the first representing the existing condition, and the second, a hypothetical case in which a 100 percent compliance within the present truck weight size and weight limits was assumed. The study revealed that between 20-30 percent of the trucks ran overweight, causing extensive damage to the highway network. The report recommended that the current fines and permit fees be structured such that truck-size and weight violators would pay proportionally to the damage they cause.

A study was conducted in 1981 by C. M. Walton and O. Gericke [Ref 24], which was an assessment of change in truck dimensions on highway geometric design principles and practices. This study identified the geometric design elements which would be affected by certain legal changes in dimensions and weights, quantified the effects of these changes under different operating conditions, and derived cost estimates on the upgrading of road sections. These estimates were made for four different vehicle design scenarios and two highway class combinations. No analysis was done on urban, county or local roads.

In 1983, Clyde E. Lee, P.R. Shankar, and B. Izhadmehr [Ref 12], conducted a study that addressed two issues: development of a practical technique for estimating patterns of axle loads in each lane of multilane highways and definition of the relative frequency distributions of truck wheel placement within a traffic lane. It emphasized the importance of a four-lane weighing and classifying capability and recommended that the traffic load forecasting procedure consist of the use of axle equivalency factors for different vehicle types with data separated by direction and lane. It recommended special equivalency factors for steering and tridem axles but the same American Association of State Highway Officials (AASHO) factors for single and tandem axles. The major applications of these findings would be in establishing or modifying vehicle weight and classification programs and for pavement design and rehabilitation purposes.

A study was conducted in 1984, by W. R. McCasland and R. W. Stokes [Ref 15], which examined the effects on freeway safety and operation of six general classes of truck regulations, route restrictions, driver licensing and certification, and increased enforcement. This study's value lies in the information regarding current urban truck loading by lane, time of day, truck percentages, and other operating characteristics.

SUMMARY

After having briefly reviewed some of the studies conducted over the past decade, the objectives may be summarized and a determination made as to how best to apply this knowledge to fulfill the study goals. Objectives of the previous Texas truck studies are summarized in the following phrases :

- Recommendations to improve the Texas vehicle weighing program.
- Recommendations to update the procedures involved in acquiring traffic data.
- Development of a methodology to forecast truck-weight frequency distributions.
- Analysis of the effects of heavy trucks on the highway system.
- Assessment of changes in truck sizes and weights, and the effects on the highway network.
- Assessment of changes in truck dimensions on the geometric design principles of the highway network.

The above studies recommend improved data acquisition procedures, and quantified certain effects of changes in truck weights and sizes. This study moves one step further in an attempt to evaluate the effects of changes in truck sizes, weights and tire pressures on pavement deterioration.

CHAPTER 3. SELECTION OF TEST CROSS SECTIONS

INTRODUCTION

As stated in Chapter 1, the main objective of this project is to assess for Texas, the statewide effects of truck sizes, weights, and tire pressures on pavement deterioration. In order to make such an assessment for the aggregate state highway system, a sampling strategy was required. Pavement sections were selected that were typical of the flexible pavement structural sections and materials used in the Texas highway system. The selection process began with a set of cross sections used in TTI Project 298. A set of pavements were selected that represented different levels of the following factors [Ref 6].

Traffic:	High and low.
Surface Types:	Hot mix asphalt concrete (HMAC), surface treatment, and thin overlays on HMAC.
Highway Types:	Interstate (IH), U.S. numbered routes (US), state numbered routes (SH), and farm to market or ranch to market (FM or RM).
Location:	Urban, rural.

These representative test sections include 48 flexible pavements, which account for most of the flexible sections in the State of Texas.

Research Engineers from the Texas Transportation Institute, Texas A & M University, studied pavement performance for a large number of actual sections throughout the State of Texas. The types of highways considered were FM (rural), SH (rural), SH (urban), US (rural), US (suburban), IH (rural), IH (suburban), Urban Streets, Industrial Streets and Urban Freeways. They aggregated performance data using statistical procedures in order to develop a smaller, more managable number of generic sections. The data aggregation process, although statistically based, was not intended to produce performance functions for specific pavement sections, but rather a robust model which would produce comprehensive, state-wide statistics. Their analysis produced approximately 48 generic types of sections which together represent all flexible pavements on the Texas highway system. A listing of these test sections is shown in Appendix A. Of these 48 generic sections, 15 were selected as having properties that were sufficiently unique in terms of this study so as to warrant individual consideration.

Pavement section properties that were significant for the types of analyses to be considered under this study were selected by the research team. A variety of plausible section descriptions were prepared using the selected properties for each highway type. From six to ten pavement section descriptions were prepared for each highway type. The candidate section descriptions were submitted to maintenance engineers at the SDHPT who voted to select the section types which were most like those in his geographic of responsibilities area for each highway type. Results of the voting by 32 engineers are shown in the right-most column of Table 3-1.

FINAL SELECTION OF SECTIONS

For purposes of this study, the 15 representative flexible pavement sections shown in Table 3-1 had to be linked to real pavement sections so that measured traffic data could be related to performance. In order to develop such a link, structural properties of the sections, such as typical layer thicknesses and maintenance histories, were developed by TTI researchers in concert with SDHPT maintenance personnel as described in the previous section. These data along with geographic distribution information were used to tie the sections to appropriate real world locations (Table 3-2). This linking procedure was performed by scrutinizing various roadway sections that were characteristic of the theoretical test sections. This was done using the Texas State Department of Highways Design Division Control Log and the maintenance histories described in the previous paragraph.

The next step was to find the historical average daily traffic data (ADT's) for these selected sites. Since most of these sites were void of Automatic Traffic Recorders (ATR's), primary data were not available. Sites that had close geographic proximity and similar physical characteristics (with ATR's), were selected as surrogates for sites without historical count information.

A final listing of chosen ATR sites is shown in Table 3-3. The ADT's at these sites, collected over the years, were used as the basis for the development of future traffic trends and volumes. The ADT's collected at these sites are shown in Table 3-4. The tabular data demonstrates the range of ADT's represented by the sections and the duration of historical data.

CLASSIFICATION OF VEHICLES

After having estimated the ADT's at the various sites, the data next sought were classification counts. These data were gathered from the SDHPT's 1984 Manual Count Annual Report of " Locations and 24-Hour Average Traffic Classification". Of the various types of vehicles classified in the manual, eight were selected for the study :

TABLE 3-1. REPRESENTATIVE PAVEMENT SECTION PROPERTIES SELECTED BY STATE DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION ENGINEERS

HIGHWAY TYPE	NUMBER OF LANES	ADT	LAYER THICKNESS (INCHES)				OVERLAY (YEARS)	VOTES (OUT OF 32)
			SURFACE	BASE	SUBBASE	OVERLAY		
FM (Rural)	2	250	0.75	8.00	8.00	1.50	13	17
SH (Rural)	2	2000	0.75	13.00	8.00	1.50	12	20
SH (Urban)	2	4000	1.50	16.50	8.00	1.50	12	15
SH (Urban)	2	4000	1.50	13.00	8.00	1.50	14	14
US (Rural)	4	8000	1.50	20.00	8.00	1.50	12	18
US (Rural)	4	8000	1.50	12.00	8.00	1.50	12	11
US (Suburban)	6	18000	1.50	20.00	8.00	1.50	11	12
US (Suburban)	6	18000	1.50	14.50	8.00	1.50	13	12
IH (Rural)	4	10000	0.75	28.50	8.00	1.50	12	22
IH (Rural)	4	10000	1.50	24.50	8.00	1.50	12	16
IH (Rural)	4	10000	1.50	13.50	8.00	1.50	13	13
IH (Suburban)	4	25000	1.50	16.50	8.00	1.50	14	16
Urban Street	4	10000	1.50	12.00	8.00	1.50	12	12
Industrial Street	4	10000	1.50	15.00	8.00	1.50	12	13
Urban Freeway	6	50000	1.50	23.00	8.00	--	20	12

TABLE 3-2. SELECTED REAL PAVEMENT SECTION LOCATIONS

SECTION	COUNTY	HIGHWAY NUMBER	FROM	TO
1	Tom Green	FM 1223	Junction US 87 at Loop 306, SE of San Angelo	14 miles SE of US 87
2	Hunt	SH 50	Fannin County District 1	North Junction SH 24
3	Hunt	SH 66	Collin County Line	Greenville (Intersection of Zee and Johnson Street, otherwise known as Junction US 69 and 38 near SWCL
3a	Nacogdoches	SH 7	Junction ST 21 (1 mile SE of Nacogdoches)	NE end Ahoyac River Bridge at San Agustine County Line
4	Scurry	US 84	Garza County Line	0.5 miles East of FM 1142 (NW of Snyder)
5	Travis	US 290	Hayes County Line	Oak Hill (492 East of East end Williamson Creek Bridge)
6	Hunt	US 69	Fannin County Line	Celeste (0.2 miles East of FM 1562, i.e. at the intersection of 2nd and Sanger Streets
7	Angelina	US 69	Cherokee County Line	FM 2680 at North City Limits of Lufkin
8	Hubt	IH-30 & US 67	Rockwall County Line	Hopkins County Line
9	Potter	IH-40	Oldham County Line	Amarville (Junction US 87, 60, 287)
10	Bexar	IH-10 & US 90	San Antonio (East end Nogalotis Street overpass)	East end Cibolo Creek Bridge at Guadalupe County Line
11	Tarrant	IH-820	Fort Worth (South end of Lake Worth Bridge)	Fort Worth (Junction IH-20, near Horne Road US 377)
12	Bexar	Loop 368	San Antonio (Junction IH-410 at Fratt-Sta 2406 and 77 on NB lane)	San Antonio (Junction IH-35 - intersection Broadway and Newell Avenue
13	Angelina	US 69	Lufkin (Junction SH 103 at NW city limits)	Keltys
14	Dallas	IH-635	Tarrant County Line	Dallas (East end CRI & P Railroad Nicholson Street-Farmers Branch Channel Bridge)

TABLE 3-3. SELECTED AUTOMATIC TRAFFIC RECORDER STATIONS REPRESENTING TEST SECTIONS

SECTION NUMBER	COUNTY	LOCATION
1	Runnels	SH 158 - Northwest of Ballinger
2	Wheeler	US 83 - North of Shamrock
3	Brown	US 67 - Northeast of Brownwood
3A	McLennan	SH 06 - West of Waco
4	Kaufman	US 80 - East of Terrell
5	Bexar	US 181 - SE of San Antonio
6	Travis	US 183 - South of Austin
7	Nacogdoches	US 59 - South of Nacogdoches
8	Hidalgo	US 83 - Junction with Supr 374
9	Victoria	US 59 - East of Victoria
10	Fayette	IH 10 - East of Schulenburg
11	El Paso	IH 10 - North of El Paso
12	Lubbock	US 84 - NW of Lubbock
13	Callahan	IH 20 - East of Abilene
14	Dallas	IH 635 - SE of Dallas

TABLE 3-4. HISTORICAL ANNUAL DAILY TRAFFIC (ADT) BY STATION NUMBER

YEAR	STATION														
	1	2	3	3a	4	5	6	7	8	9	10	11	12	13	14
1959	0	1472	2025	2002	0	0	0	0	0	3728	0	2966	0	0	0
1960	0	1526	2099	2020	0	0	0	0	0	3796	0	3006	0	0	0
1961	0	1560	2297	2161	0	0	0	0	0	3996	0	3330	0	0	0
1962	0	1486	2496	2327	0	0	0	0	0	3885	0	4260	4787	0	0
1963	0	1529	2379	2142	0	0	0	0	0	3953	0	4848	4822	0	0
1964	853	1570	2573	2465	0	0	4494	0	0	4765	0	6405	5206	0	0
1965	837	1469	2679	2409	0	0	4598	0	0	5211	0	6982	5137	0	0
1966	811	1421	2748	2379	0	0	4700	0	0	5137	0	7426	5140	7200	0
1967	817	1436	2790	2434	4354	0	5073	0	0	5204	0	7592	5182	7787	0
1968	801	1409	2926	2514	4609	4176	5595	0	4292	5414	0	8184	5350	7971	0
1969	777	1413	3093	2728	4974	4259	6273	0	4433	5796	4776	9034	5497	7947	0
1970	810	1371	3253	2741	5224	4530	6693	0	4623	6355	5722	10926	5617	8079	0
1971	812	1340	3231	2887	5397	4821	7396	0	4984	6471	6613	12092	5795	8745	11979
1972	834	1360	3229	3043	5704	5195	8100	9816	5526	6954	7297	13145	6198	9727	20012
1973	836	1414	3289	3270	6038	5457	8772	10178	6116	7465	8260	14179	6597	10250	23946
1974	766	1404	3170	3093	5877	5251	8767	9900	6248	7633	7605	13529	6436	9616	24460
1975	824	1425	3269	3221	5982	5493	8997	10464	6744	8542	8422	14566	6767	10575	28315
1976	846	1505	3427	3460	6229	5693	9628	11171	7216	9388	9268	15393	7294	11147	33241
1977	872	1641	3557	3725	6319	6041	10202	12015	7731	10301	9975	16902	7608	11589	36895
1978	880	1793	3650	3847	6414	6370	10809	12653	8133	11299	10501	17934	7806	12018	39049
1979	884	1703	3575	4023	6376	6244	10986	12534	8337	11173	10250	17775	7650	11749	38695
1980	789	1643	3457	3900	6302	6205	11417	12944	8517	11516	10620	17779	7419	11505	38564
1981	870	1855	3556	3941	6598	6188	11902	13872	9615	12705	11560	17781	7792	12295	40341
1982	915	2091	3715	4114	7059	6278	12454	14075	10013	12913	11706	18328	7869	12787	43767
1983	946	1705	3898	4344	7693	6527	13959	14247	10189	12771	11501	19328	8144	12883	50213
1984	964	1676	3972	4651	8275	6863	16507	14823	10959	13610	11715	20859	8609	12533	56838

1. Passenger Cars (Passenger cars and single-unit panel and pickup trucks) - P.
2. Single-unit, other two axle trucks - 2D.
3. Single-unit three-axle trucks - 3A.
4. 3-axle semi-trailer combinations - 2S1.
5. 4-axle semi-trailer combinations - 2S2.
6. 4-axle semi-trailer combinations - 3S1.
7. 5-axle semi-trailer combinations - 3S2.
8. Truck & trailer 5 & 6-axle combinations - 2S12 & 3S12.

These vehicles were selected because they constitute the majority of the vehicles operating on Texas highways. Table B-1 (in Appendix B) provides sample classification data for the year 1984 and the percentages of total vehicles in each class at the different sites indicated in Table 3-3. Graphical representations of the percentages of vehicles for four different highway types, namely: high volume IH (Sta 11), low volume IH (Sta 8), high volume SH (Sta 3), low volume SH (Sta 2), are shown in Figs 3-1 thru 3-4. The rest of the figures, representing the other highway types are shown in Appendix B.

LINKING OF SELECTED SITES TO WIM STATION LOCATIONS

Vehicle weight is the most important factor for the planning, design, operation, and maintainence of roadway networks. In this study, the weight data collected at the six Weigh-In-Motion (WIM) stations situated in Texas was utilized. The WIM stations are identified with numbers as follows: 501, 502, 503, 504, 505 & 506. Figure 3-5 provides the geographic locations of these stations .

Because of the dearth of weighing stations in the State of Texas, linking of the selected sites (which had no weighing stations in the vicinity), to WIM stations with similar characteristics was required. This linking procedure became imperative because of the importance of weight data and was done based on similarity of vehicle classification distributions.

VEHICLE CLASSIFICATION DATA AT THE DIFFERENT WIM STATIONS

Frequency distributions of vehicle classes for each of the six WIM stations (graphical representations) are shown in Figs 3-6 thru 3-11. It is interesting to note, on observation of these figures, that the percentage distribution of vehicle classes weighed at each of these WIM stations are similar; this observation can be tested statistically using the chi-square test for homogeneity.

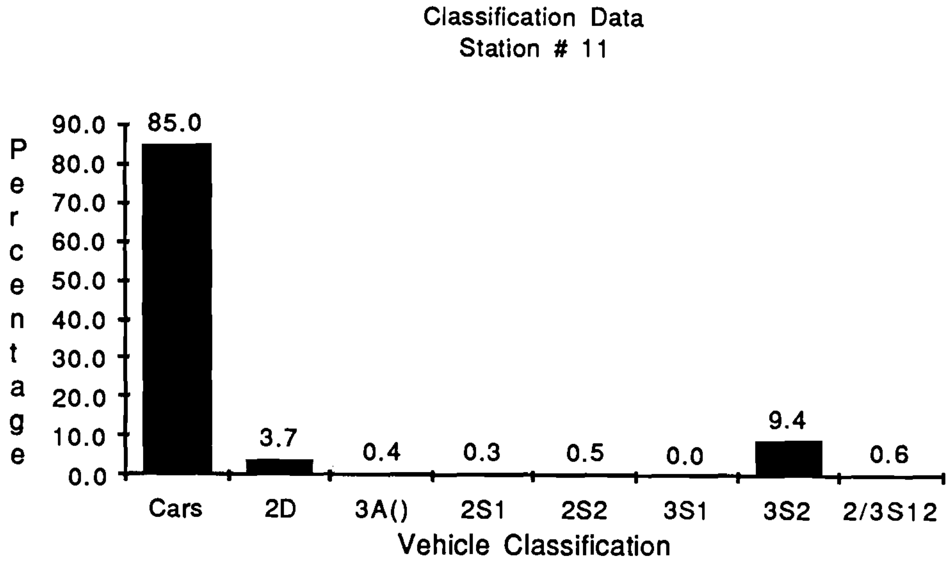


Figure 3-1. Vehicle classification data, Station Number 11, 1984.

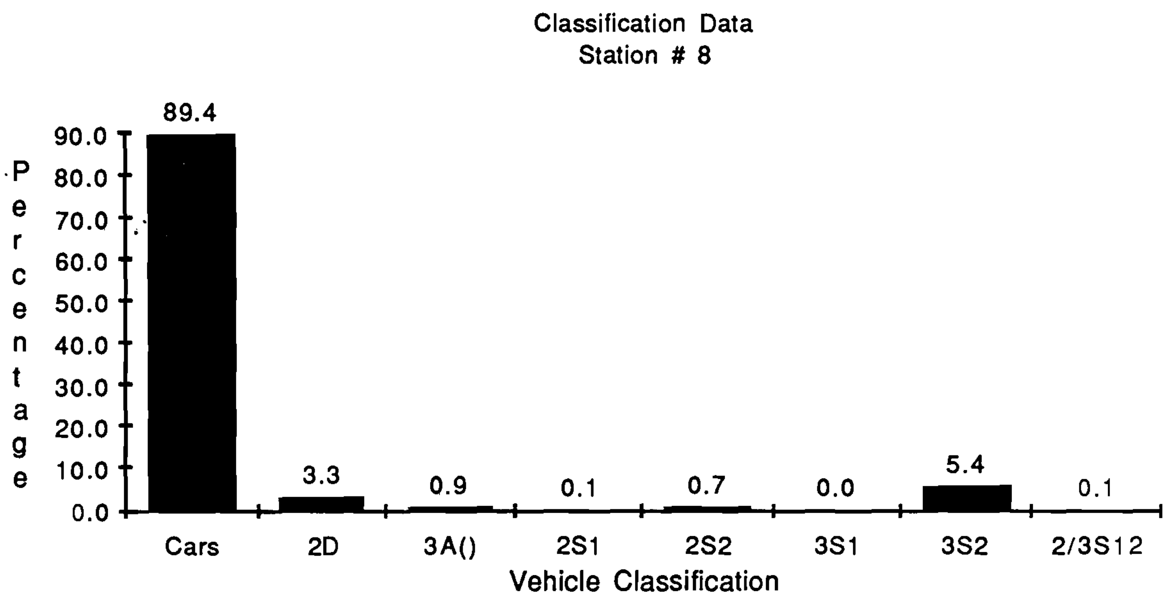


Figure 3-2. Vehicle classification data, Station Number 8, 1984.

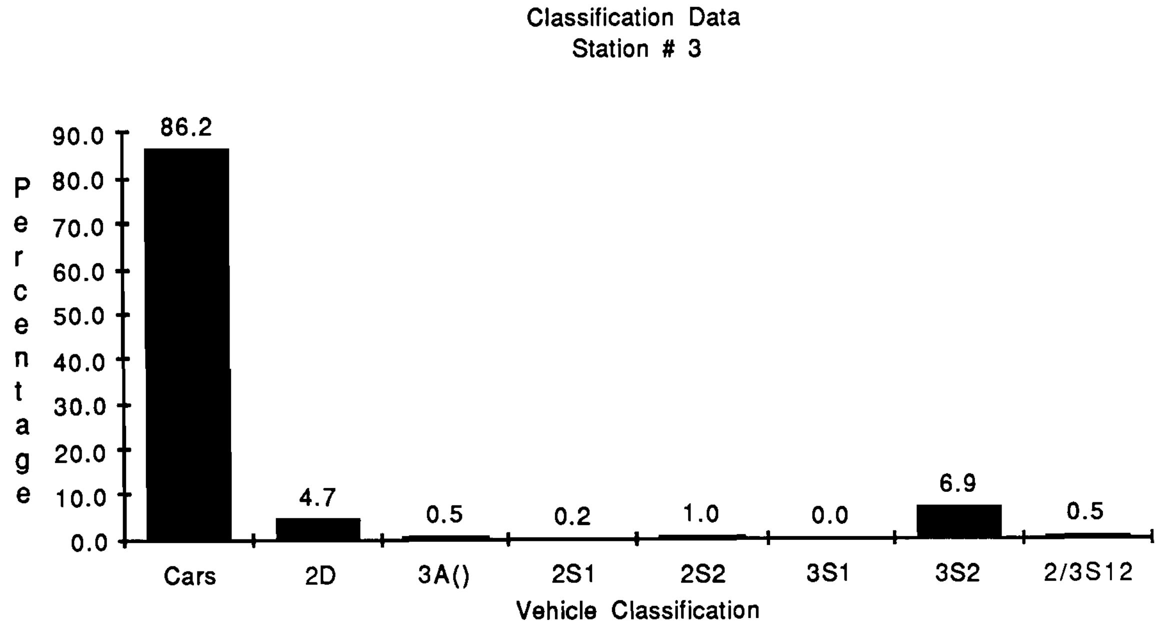


Figure 3-3. Vehicle classification data, Station Number 3, 1984.

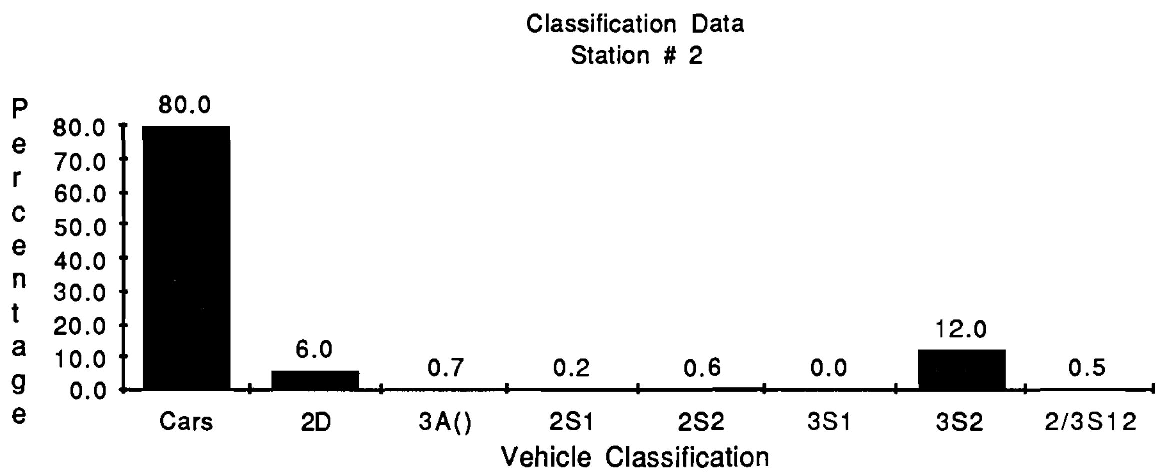


Figure 3-4. Vehicle classification data, Station Number 2, 1984.

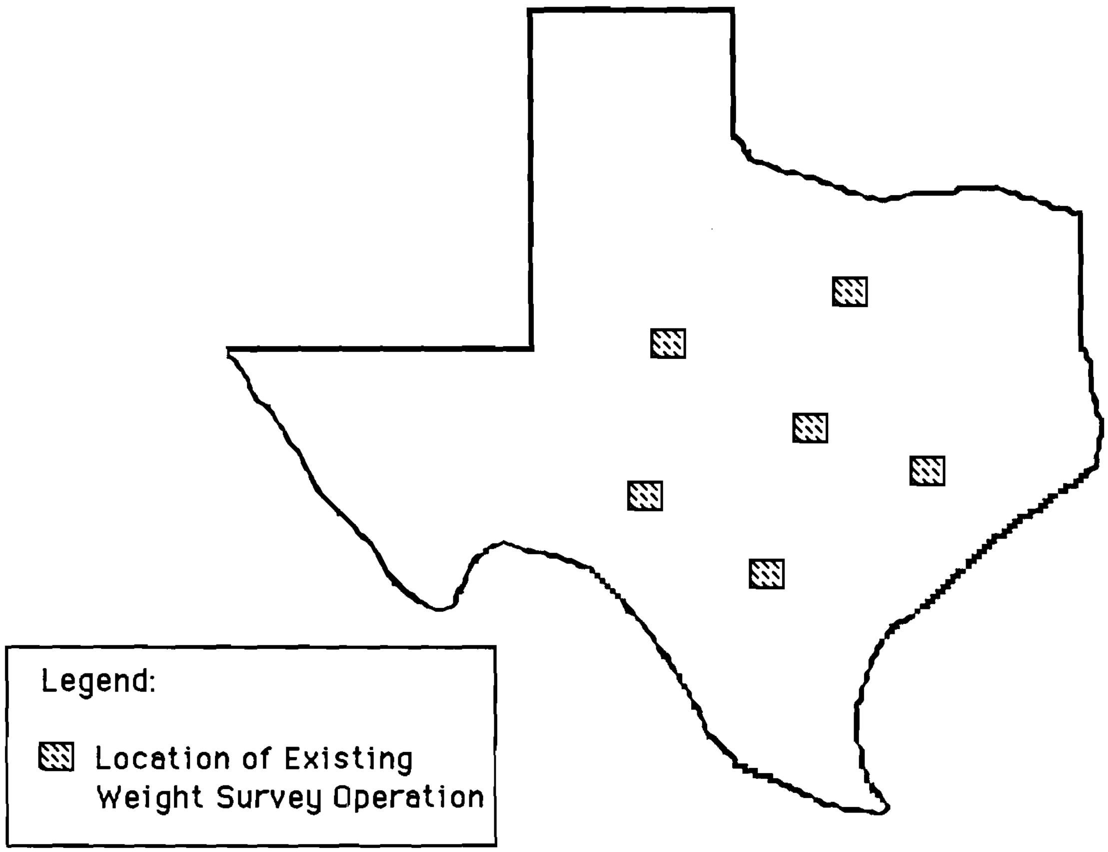


Figure 3-5. Geographical locations of existing weight survey stations.

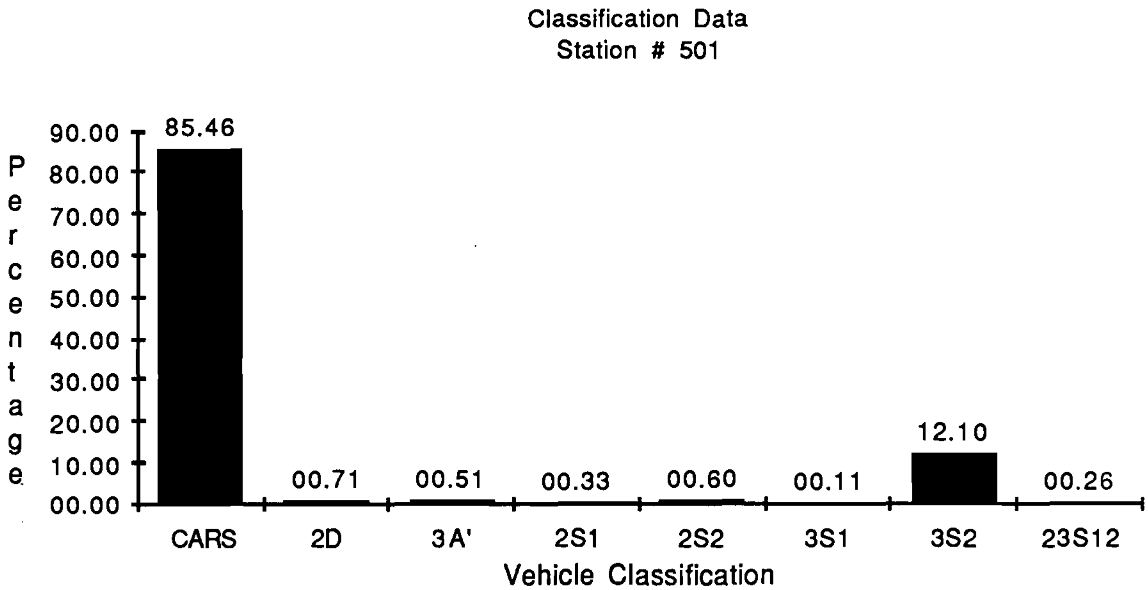


Figure 3-6. Vehicle classification data, WIM Station Number 501, 1984.

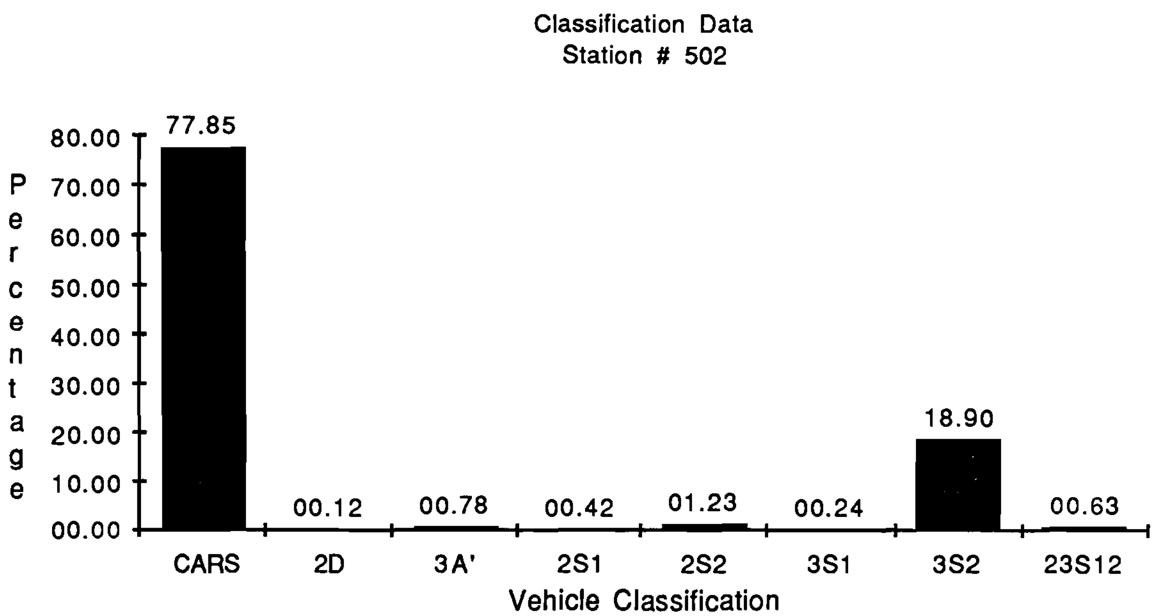


Figure 3-7. Vehicle classification data, WIM Station Number 502, 1984.

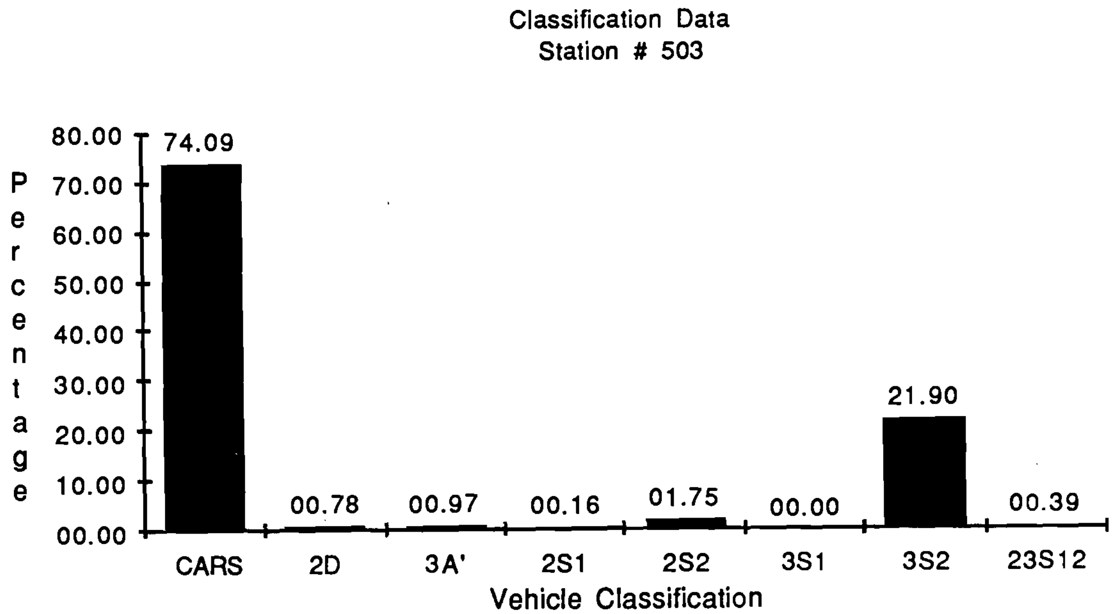


Figure 3-8. Vehicle classification data, WIM Station Number 503, 1984.

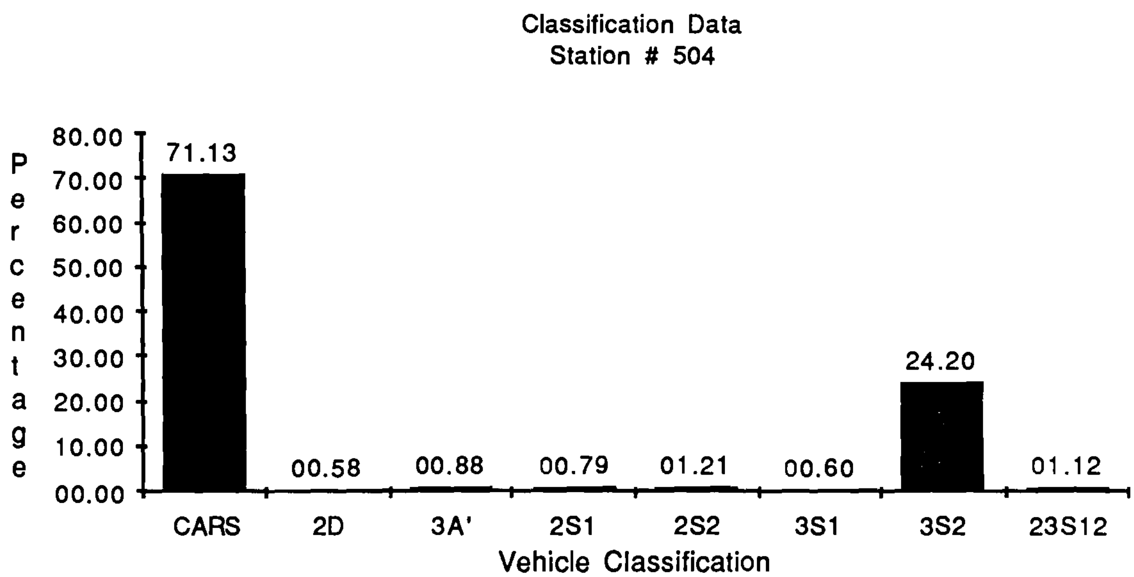


Figure 3-9. Vehicle classification data, Station Number 504, 1984.

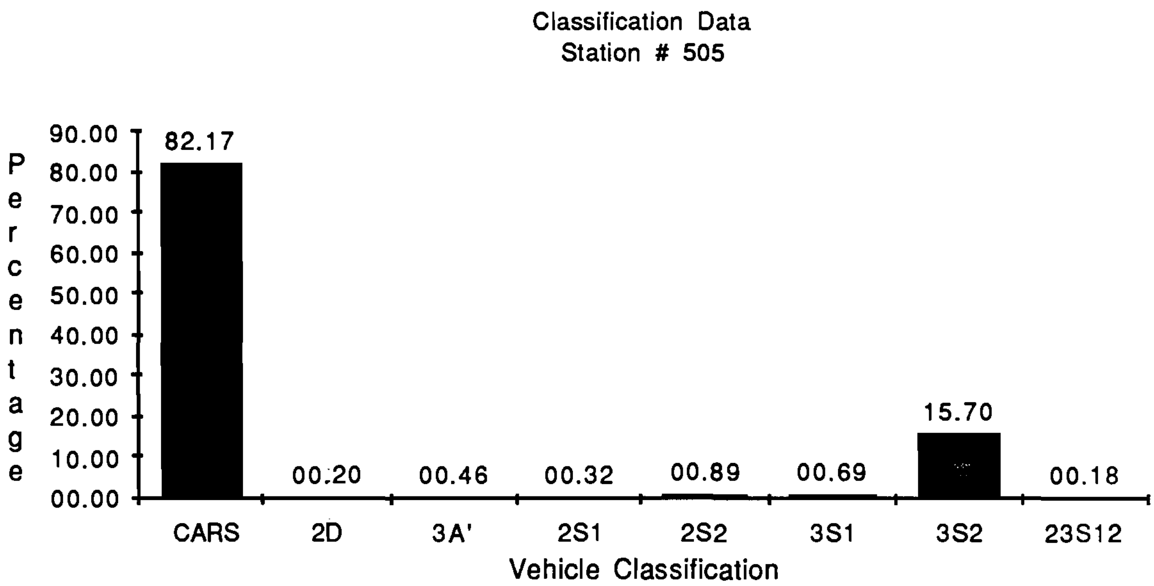


Figure 3-10. Vehicle classification data, WIM Station Number 505, 1984.

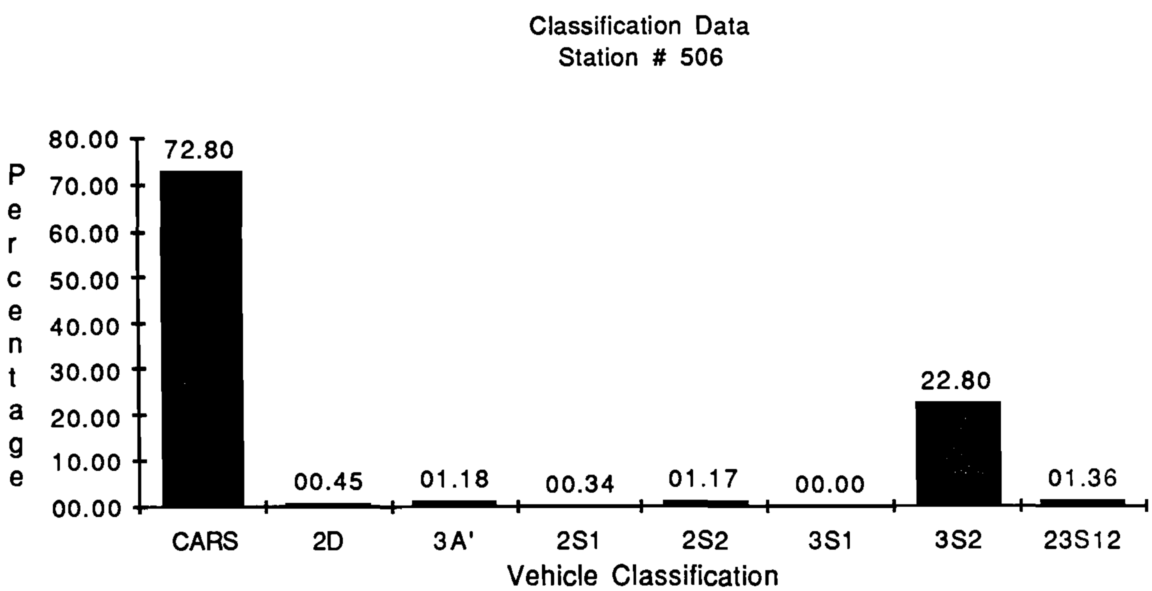


Figure 3-11. Vehicle classification data, WIM Station Number 506, 1984.

THE CHI-SQUARE TEST FOR HOMOGENIETY

The chi-square test is used to test the validity of an assumption of homogeneity among the frequency distributions of vehicle classes weighed at WIM stations. The hypothesis to be tested is that the frequency distributions come from the same population. In other words, one can test the significance of the difference that exists among the percentages of vehicle classes from the various WIM stations. One way to perform this homogeneity test is to compare the data from the various WIM locations, taken two at a time. For example, one could compare the data of Station 501 with that of 502, then with 503, etc., for all possible combinations. A comparison is made between two sets of observed frequencies in order to determine whether the differences among the chosen sets are due to chance, or due to the sets coming from different populations and therefore having different characteristics.

HYPOTHESIS TESTING

The testing of hypotheses about populations is a procedure that can be used for choosing between alternative courses of action. The tests are based on determining the likelihood of chance occurances of the observed samples versus the likelihood that the sample values are different because they represent different populations. In this case, the objective is to compare two selected sets of percentage classification distributions to test the null hypothesis (H_0) that they come from the same population against the alternative hypothesis (H_1) that they do not. If the null hypothesis cannot be rejected, the two samples can be described as being "not statistically" different which means that they either came from the same population, or, the sampling process biased the results to make them appear as if they came from the same population.

THE CHI-SQUARE PROCEDURE

Described below is a brief summary of the chi-square procedure used to test the hypothesis that percentages of each class of vehicles weighed at WIM Stations 501 and 502 are not significantly different. These data are presented in Table 3-5.

1. Columns 2 & 3 of Table 3-6 contain the percentages or the observed frequencies of two known data sets.
2. An hypothesis is formulated that states the following :

TABLE 3-5. VEHICLE CLASSIFICATION PERCENTAGES AT WIM LOCATIONS

VEHICLE TYPE	WIM STATION NUMBER, 1984					
	501	502	503	504	505	506
CARS	85.46	77.85	74.09	71.13	82.17	72.80
2D	00.71	00.12	00.78	00.58	00.20	00.45
3A'	00.51	00.78	00.97	00.88	00.46	01.18
2S1	00.33	00.42	00.16	00.79	00.32	00.34
2S2	00.60	01.23	01.75	01.21	00.89	01.17
3S1	00.11	00.24	00.00	00.60	00.69	00.00
3S2	12.10	18.90	21.90	24.20	15.70	22.80
23S12	00.26	00.63	00.39	01.12	00.18	01.36

TABLE 3-6. CHI-SQUARE TESTING, COMPARING WIM STATIONS 501 AND 502

VEHICLE TYPE NUMBER	STATION NUMBER 501 F_o	STATION NUMBER 502 F_c	$F_o - F_c$ (b - c)	$(F_o - F_c)$**2 (d)**2	$(F_o - F_c)$**2/F_c e/c
1	85.5	77.9	7.6	57.76	0.74
2	0.7	0.1	0.6	0.36	3.60
3	0.5	0.8	-0.3	0.09	0.11
4	0.3	0.4	-0.1	0.01	0.03
5	0.6	1.2	-0.6	0.36	0.30
6	0	0	0	0	0.00
7	12.1	18.9	-6.8	46.24	2.45
8	0.3	0.7	-0.4	0.16	0.23
TOTAL					7.45

H_o: the two sets of percentage classification distributions come from the same population.

H_a: the two sets of percentage classification distributions do not come from the same population.

3. The two data sets are compared by computing the 'chi-square', whose value depends upon the differences between the above two selected data sets. The computed value of chi-square, X^2, is given by the expression :

$$X^2 = \Sigma (f_o - f_c)^2 / f_2$$

where f_o = percentages of data set 1 (from Station 501)

f_c = percentages of data set 2 (from Station 502)

The computed chi-square value is compared with the critical value of chi-square, $X^2_{critical}$, obtained from the chi-square distribution table. The table provides the values of the test statistic which would occur due to chance alone. If the magnitude of the computed value is less than $X^2_{critical}$, the null hypothesis is not rejected. The number of degrees of freedom (df) were eight, and a 5 percent level of significance was used. The value obtained from the chi-square distribution table for df = 8, and 5 percent level of significance was 15.507. This means that the computed values of chi-square greater than or equal 15.507 will occur by chance 5 percent of the time (Fig 3-12). The shaded area is the rejection area and the rest of the area enclosed under the curve is the region of acceptance.

The results of the chi-square tests for all possible combinations of WIM stations taken two at a time are shown in Appendix C (Tables C-1 thru C-14). The results generally indicate that the observed frequencies produce X^2 values which are less than the $X^2_{critical}$ value of 15.507. This indicates that the null hypothesis that "the two sets of percentage classification distributions come from the same population" cannot be rejected. This is frequently interpreted as meaning that the sample percentage data sets come from the same population.

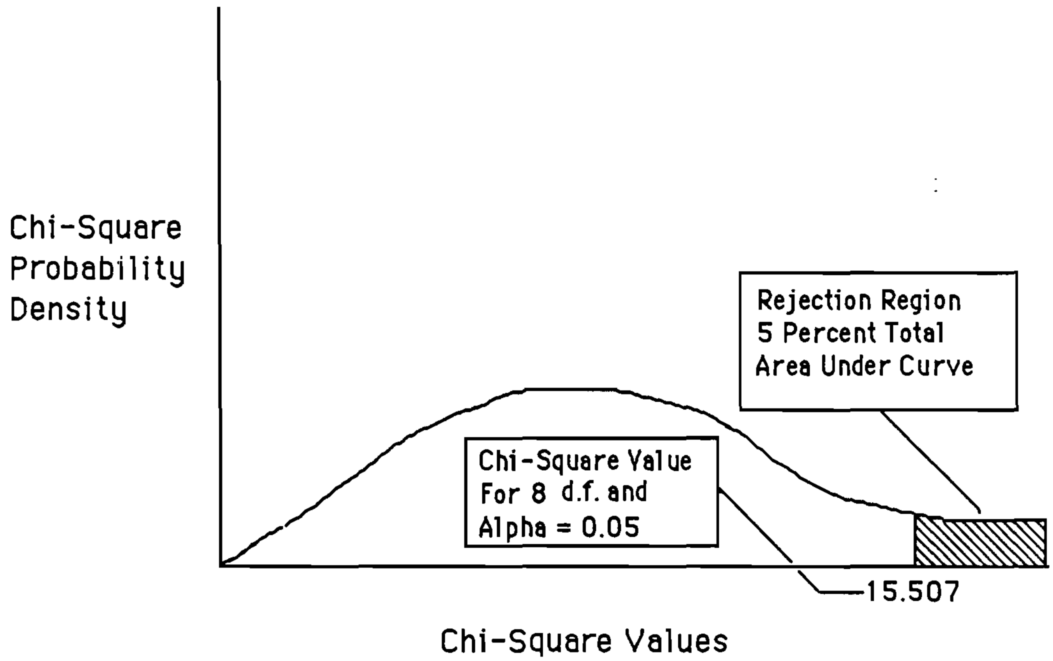


Figure 3-12. Chi-Square distribution for hypothesis tests.

SUMMARY

Since the percentage of vehicles at each of these WIM stations were similar, it was difficult to link the selected sites to any particular WIM station. Thus the weight data at each of the stations were combined to form a large data set, and the selected sites were all linked to this newly formed weight data set. This procedure was found to be the most practicable solution, due to the lack of sufficient weight stations in Texas. This will be discussed in greater depth in the next chapter.

CHAPTER 4. FORECASTING TECHNIQUES

After having selected the representative pavement structural-sections and obtaining present average daily traffic (ADT), the next step involved the forecasting of these ADT's to the year 2005 so as to develop future traffic trends and volumes. Techniques which might be used for forecasting ADT's or other phenomena whose magnitude varies with time are described within this chapter.

IMPORTANCE OF TRAFFIC VOLUME DATA

Traffic volume data are important from the point of view of characterizing the utilization of a roadway. This is often measured in terms of ADT which is generally defined as the average number of vehicles passing a selected location in both travel directions in a 24-hour period. Present and forecasted ADT's at the various selected sites can be used for the following purposes:

- To measure trends in traffic volume.
- To estimate highway rehabilitation costs.
- To estimate highway user costs.
- To estimate future highway demands and level of service.
- To design and plan highways to serve the more vehicles in the coming years.

For purposes of this study, various forecasting methodologies were analyzed so as to make forecasts of ADT's at the selected stations. Forecasting techniques which were considered for use in predicting ADT's at the various selected sites are described in the following sections.

FORECASTING METHODOLOGIES

Various forecasting techniques were studied to identify a candidate method for forecasting ADT at selected Automatic Traffic Recorder (ATR) sites, to the year 2005. These techniques range from the straight forward to the sophisticated. The techniques vary in the number of levels and degrees of complexity by which they may be approached. Basically, there exists just three general techniques [Ref 19]. They are:

1. Persistence
2. Forces at Work
3. Time Series Techniques

These will be discussed in detail in the following paragraphs.

Persistence

This is one of the simplest methods of forecasting. This method is built on the assumption that future trends will resemble those of the present. In other words, things will not change over time. This technique is generally used for short-term forecasts. However, the results from this technique become questionable as the period of the forecast becomes larger [Ref 19].

Forces at Work

This is one of the most rational techniques that is used in the forecasting process. This technique analyzes the causative forces that affect the variable to be predicted. The forecasts are based on the relationships disclosed and on any anticipated changes in these forces and their operation. Forces at work involve the usage of various techniques, both mathematical and non-mathematical, in the search for usable relationships, and in the quantification of these relationships. For example, consider a regression analysis mathematical technique which is relatively simple, but is probably the most important from the standpoint of frequency and extensiveness of use, [Ref 19].

Regression analysis refers to the technique of deriving an equation by which one of the variables, the dependant variable (ADT), may be estimated from the other variables, the independent variables (in our case, we consider just one independent variable namely, time (year). Discussions will be limited to straight-line relationships between the dependent and independent variables (ADT and time (year)). This is termed simple linear regression The general equation is of the form :

$$Y = a + bX + \partial$$

where
- a = the Y-intercept (a constant)
- b = the slope, or the rate of change of Y.
- Y = dependent variable (ADT)
- X = independent variable (time (year))
- ∂ = random error

The first item to be observed, is the relationship between the two chosen variables, namely (ADT) and time (year). Historical data of ADT versus year is obtained and the observations are plotted as points whose X-values are the values of the independent variable (year), and Y-values are the values of the dependent variable (ADT). This plot is called a scatter diagram. Close examination of the scatter diagram indicates, first, whether the relationship is sufficiently appropriate for the analysis to be worth carrying forward, and second, whether the relationship is approximated by a straight line.

The computer software program, STATWORKS, can be used to plot data points on a scatter diagram and construct the regression line, using the method of least squares. Within the linear relationship of the equation shown above, the random error ∂ will be small if the two variables are closely related by the hypothesized straight line relationship. Thus if the observed values of X & Y are: $(x_1,y_1), (x_2,y_2),........(x_n,y_n)$, then

$$y_i = a + bx_i + \partial_i \quad i = 1,2,..........,n \ .$$

The problem is to estimate the values of a and b such that the sum of the squared ∂_i's (i.e. $\partial_1^2 + \partial_2^2 + ...+ \partial_n^2$) is as small as possible. Thus the constants, a and b are chosen such that $\Sigma\, \partial_i^2$ = minimum.

The value of the coefficient of correlation, r, is an important numerical measure which serves as an index of the closeness of the relationship between the dependent and independent variable, namely X and Y.

$$r = \frac{\Sigma\, (x_i - x\dagger)(y_i - y\dagger)}{\sqrt{\Sigma\, (x_i - x\dagger)^2 \Sigma (y_i - y\dagger)^2}}$$

where $x\dagger = \Sigma\, x_i / n$

$y\dagger = \Sigma\, y_i / n$

Its value lies between 0 and 1, both positive and negative. As a matter of fact, the sign of r is given by the sign of b in the regression equation, so that the sign of r tells whether the relationship between X and Y is direct or inverse.

If the value of $r \approx 0$, it means that there exists no linear relationship between X and Y. On the other hand, if $r \approx \pm 1$, there exists a good linear relationship between X and Y.

Time Series Techniques

A time series can be considered as a collection of observations made sequentially in time. A simple example would be daily traffic volumes observed in a year. Time series data can be referred to as either continuous or discrete. It is said to be continuous when observations are made continuously in time. It is said to be discrete when observations are taken only at specific periods in time. The term discrete can also be used for a series of this type even when the measured variable is a continuous variable [Ref 4].

The special feature of time series analysis is the fact that successive observations are not independent and that the analysis must take into account the time order of the observations. When successive observations are dependent, future values may be predicted from past observations. When analyzing a time series, the first step in the analysis involves the plotting of data and observing some properties of the series (described later). From these observed properties, one can predict the future value of the series can be predicted. This is a rather difficult procedure, and requires the expertise of experienced people.

Statistical techniques for analyzing time series range from simple to more complicated methods. Time Series analysis can be referred to as a statistical procedure that employs time-series data, usually for the purpose of explaining past events or for forecasting future events. Some of the traditional methods of time series analysis are mainly concerned with decomposing a series into various components, namely trend, seasonal, cyclic, and irregular components. Some of these terms are described below.

1. Trend - This may be loosely defined as a 'long term' change in the mean, or the 'long-range' gradual change in the mean [Ref 4]. Intepretation of the term 'long term' or 'long-range' is generally misunderstood. It does not refer to any fixed or minimum length of time, but rather to a length of time which is variable and dependent upon the nature of the quantity being measured. Thus in speaking of a 'trend', we must consider the number of observations available and make a subjective assessment of what exactly is meant by 'long term'.

2. Seasonal Variation - Certain regularly recurring forces cause periodic fluctuations to occur rather frequently within particular time intervals. These may last from a day to several months of the year. These types of fluctuations are generally easy to understand.

TIME SERIES IN FORECASTING

The process of forecasting a given set of data as a time series is generally an important problem. The problems that arise in such a process range from estimating the x_{n+1} value of a series x_1, x_2, ...,x_n, to finding a suitable procedure to help in the forecast.

The Time Series Process

Before getting into the details of the time series process, the idea of a stationary time series needs to be introduced, since most of the probability theory of time series is related to this concept. A time series is said to be stationary if there is no systematic change in the mean and variance, and if periodic fluctuations have been removed. Non-stationary series are generally converted to stationary ones to utilize the time series probability theory [Ref 4]. The process starts by first plotting a given data set, i.e. observations against time. The advantages of plotting are:

(a) The different features of the plot, namely trend, seasonality, and discontinuities are observed.

(b) The plot may indicate a need to transform the values of the observed variable. The reasons for this are listed below.

   (i) To stabilize the variance - if a trend is observed in the series and variance appears to increase, it is advisable to transform the data.

   (ii) To make the seasonal effect additive - if a trend is observed in the series and the size of the seasonal effect appears to increase with the mean, then it is advisable to transform the data so as to make the seasonal effect constant.

Series Containing a Trend

If the series contains a trend, it could be analyzed or removed in order to analyze local fluctuations. Some of the commonly used methods for this process are:

1. Curve Fitting
2. Smoothing
3. Differencing

Curve Fitting

This is a method of dealing with non-seasonal data which contains a trend. When there is no theory to specify the trend of a given series as a certain function of time, it may be possible to approximate it by a low-degree polynomial. The polynomial trend is basically a descriptive means of summarizing the overall characteristics of the series. For example, a given series could be approximated by a Gompertz curve, which is

$$\log x_t = a - br^t$$

where a, b, r are parameters with $0 < r < 1$
x_t = observations at time t

The fitted function provides a measure of trend and the residuals provide an estimate of local fluctuations. The residuals are the differences between the observations and the corresponding values of the fitted curve.

Smoothing

Sometimes the trend is a smooth function of time and does not fluctuate greatly in any small time interval, but still is not closely approximated by a simple function of time over the entire range under consideration. This method uses a linear filter which converts one time series, X_t into another, Y_t. This process, which is used to smooth local fluctuations and estimate the local mean is often referred to as a moving average. The simple moving average is generally not recommended by itself for measuring trend, although it can be useful for removing seasonal variation. The reason being that (a) the moving average series is still quite irregular in appearence, (b) it tends to exhibit strong cyclical fluctuations even though none exist in the original time series data , and (c) it could produce all kinds of false impressions about the underlying data. A mathematical representation of the above process is as follows:

$$Y_t = \Sigma C_r * X_{t+r}$$

where C_r = set of weights.
ΣC_r should be equal to 1, to smooth out local fluctuations and estimate local mean.

Differencing

This is a special type of filtering which is used in removing a trend by successively subtracting selected values from the observations until they become stationary. For non-seasonal data, first order differencing is usually sufficient to stationize the series [Ref 4]. Thus the new series, $(z_1, z_2, \ldots\ldots\ldots, z_{n-1})$ is formed from the original series, $(y_1, y_2, \ldots\ldots\ldots\ldots, y_n)$, i.e.:

$$z_t = y_{t+1} - y_t = -y_{t+1}$$

Series Containing Seasonal Fluctuations

On the other hand, if a series contains seasonal fluctuations, it could either be analyzed or removed. For a series showing little trend, it is usually adequate to simply calculate the average for each time interval (for a year in our case) and compare it with the overall average figure, either as a difference or as a ratio. Thus the seasonal effects should be eliminated to stationize the series.

More About The Series

One of the problems of analyzing a time series is to decide whether an observed time series results from a process of independent random variables. A simple alternative to independence is a process in which successive observations are correlated. This process is termed autocorrelation , since it measures the correlation between successive observations. Autocorrelation coefficients are a result of this process and these coefficients help tounderstand the time series structure.

Consider a large series of N observations, namely x_1, x_2, ...,x_n. Pairs of observations, namely (x_1,x_2), (x_2,x_3),.....(x_{n-1},x_n) can be formed. Regarding the first observations in each pair as one variable, and the second observations as a second variable, the correlation coefficient between x_t and x_{t+1} is given by:

$$r_1 = \frac{\Sigma(x_t - x\dagger)(x_{t+1} - x\dagger)}{\Sigma(x_t - x\dagger)^2}$$

where $\quad x\dagger = \Sigma x_t/N$

In this way, the correlation between observations selected any distance apart can be found. A set of autocorrelation coefficients can be plotted on a graph, called a correlogram. This is used as a tool in better understanding the properties and relationships between observations of a given time-series data set.

Stochastic Processes

Consideration can also be given to the different kinds of models for the generation of a time series in which the characteristics and useful properties appropriate to the time sequence are not in a deterministic mean value function, but are in the probability structure itself. Most physical processes in the real world involve a stochastic or random element in their structure, as well as a stochastic process that evolves in time according to probabilistic laws [Ref 4]. A stochastic process is said to be stationary if it's probability structure does not change with time. The processes that are listed below are processes that are stationary in nature.

1. A purely random process.
2. Random Walk.
3. Autoregressive (AR) Process.
4. Moving Average (MA) Process.
5. Mixed Models (ARMA)
6. Integrated Models (ARIMA)

Some of the important processes mentioned above will be described in detail.

Autoregressive (AR) Process

This is one of the simplest and most useful models. If we consider $[Z_t]$ to be a purely random process with mean zero and variance s_z^2, then a process X_t is said to be an autoregressive process of order m if

$$X_t = a_1 x_{t-1} + \cdots\cdots + Z_t$$

For example, let us examine the first order case, where m = 1.

$$X_t = a x_{t-1} + Z_t$$

The first order (AR) process is sometimes called the Markov Process [Ref 4].

Moving Average (MA) Process

Another simple model of a stationary stochastic process is the moving average. If we consider $[Z_t]$ to be a purely random process with mean zero and variance s_z^2, then a process X_t is said to be a moving average process of order m if,

$$Xt = B_0 Z_t + B_1 Z_{t-1} + \text{------} + B_m Z_{t-m}$$

where $\{B_i\}$ are constants. The B's are usually scaled so that B_o=1 [Ref 11].

After having described the time series process, let us come back to the forecasting procedure. There are basically three models that aid in the forecasting procedure. They are:

1. The time trend model - for long term, deterministic change.
2. The time series model - for short term fluctuations.
3. The seasonal model - for regular seasonal fluctuations.

The more widely used of the above models, namely the time trend model and the time series model.

Time Trend Approach

This approach assumes a fixed pattern of behavior across time, with little flexibility for change. This method observes any long-term behavior in the historical data and fits equations as functions of time. Observations in the distant past can be as influential as recent observations in determining the forecast. This approach could be subdivided into different cases for solving a given problem. One such case would be to assume that the series is a constant, plus purely random fluctuations that are independent from one time period to the next [Ref 20]. The general form of the equation is

$$X = b + e$$

where e = independent zero mean random error.
b = time series mean.

The model will change, however, if the series exhibits growth.

Time Series Approach

This approach assumes that a future value is a linear function of past values. If the model is a function of past values for a finite number of periods, it is called an Auto Regression (AR) Model [Ref 2]. This approach models short-term fluctuations. This is rather like a multiple regression model, where X_i is regressed on past values of x_i and not on independent variables. Hence it is called auto-regression [Ref 20]. The general equation is of the form

$$X_i = L_1X_{i-1} + L_2X_{i-2} + \text{- - - - -} + L_mX_{i-m} + Z_t$$

The coefficient $L_i(i = 1,\text{----},m)$ are called auto regressive parameters. Z_t is a discrete, purely random process with mean m and variance s^2.

SUMMARY

As stated earlier in the chapter, choosing a specific method for forecasting any given data requires careful analysis of the data plotted, in the form of scatter plots, and then the selection of the right methodology for the forecasting procedure. After having carefully studied the scatter plots of the historical ADT data collected at the various locations, the forecasting procedures selected for use in this study were limited to two general techniques, namely forces at work (regression analysis), and time series techniques. For the scatter plots that exhibited a linear trend, regression techniques were used. For the plots that exhibited cyclic variation, time series techniques were used. The details of the types of methods used for the different ADT's will be discussed in the next chapter.

CHAPTER 5. FORECASTING AVERAGE DAILY TRAFFIC, TRUCK WEIGHT, AND CLASSIFICATION DATA

In keeping with the basic objective of evaluating the effects of truck sizes, weights and tire pressures on pavement deterioration, both current and historical data describing average daily traffic (ADT), truck axle weight distributions, and vehicle classification were obtained. These data were used to characterize traffic demands for two traffic scenarios described earlier. The existing-traffic scenario is characterized by the actual 1984 traffic survey data while the base scenario (considered the hypothetical case), consists of modified truck-weight distributions as well as ADT's. Data are modified for the base case so that the total net cargo transported is that for 1984, but legal vehicle weights are reduced (discussed in more detail later in the chapter). This, in effect, permits a comparison of the numbers of trucks which would be required if all vehicles were legally loaded and the weight of cargo to be transported was that which was earned in 1984. Data from both scenarios must be forecasted so that it will cover a chosen 20-year time span.

In this chapter, procedures for forecasting ADT, truck-weight distribution data, and vehicle classification data are discussed. These results provide the basis for evaluating the effects of truck sizes, weights, and tire pressures, through use of the existing-traffic and the base scenarios.

FORECASTING OF AVERAGE DAILY TRAFFIC

Average Daily Traffic was to be forecasted at the fifteen selected sites to the year 2005. Although various techniques were analyzed, as discussed in the previous chapter, only three were selected. Linear regression was used when plots of the data exhibited a linear trend and time series techniques were used when the plots of the data exhibited cyclic variations. The time series models that were used were the time series and the time trend models.

Methodology

Historical data of ADT at the various test sites under consideration were plotted with time (year) as the abscissa and ADT as the ordinate. The variations in the various plots of ADT vs. time were observed to contain both linear and cyclic variations. The plots that exhibited linear variations were forecasted using simple linear regression. However, if the coefficients of correlation weren't approximately ± 1, the forecast was discarded, and the data was forecasted using more sophisticated time series techniques. The plots that exhibited cyclic variations were forecasted using time series techniques contained in the computer software package called SAS [Ref 20].

A time series model, STEPAR (STEPwise Auto Regression) was used to forecast ADT's for some stations. The STEPAR method combines a time trend with an autoregressive model and uses a stepwise method to select lags for the autoregressive process.

For test sites, where the number of observations was not sufficient, a time trend model, called EXPO (EXPOnential smoothing) was used to forecast ADT. The EXPO method produces a time trend forecast, but in fitting the trend, the parameters are allowed to change gradually over time, with earlier observations given exponentially declining weights. This means that the most recent data is weighted more heavily than past data [Ref 20]. Listed below are the methods that were used to forecast ADT's at the different locations.

METHOD	LOCATION
1. Simple Linear Regression	# 8
2. Time trend Model (EXPO)	# 1,4,5,6,7,10,12,13,14
3. Time Series Model (STEPAR)	# 2,3,3A,9,11

Graphical presentations of ADT's are presented in Figs 5-1 and 5-2 in the text and Figs D-1 through D-13 in Appendix D. Both linear and cyclic trends are observed in Figs 5-1 and 5-2 respectively. These figures illustrate the types and shapes of the trendlines for each of the forecasted sites. Complete tabular listings are presented in Table E-1 in Appendix E.

FORECASTING OF TRUCK WEIGHT DATA DISTRIBUTIONS

As mentioned earlier in the chapter, truck weight data is the most important factor for the planning, design, operation and maintenance of roadway networks. These data help in the planning and design of new highway systems to accomodate heavier and larger trucks in the years to come, and the maintenancence of existing roadways for smooth, continuous flow. This data is collected annually by the State Department of Highways and Public Transportation. The data collected in the most recent year (1984), is used as the base data in the forecasting of truck-weight distributions.

BRIEF HISTORY OF THE TRUCK WEIGHING PROGRAM IN TEXAS

For nearly four decades, truck weight data was collected at 21 pre-selected roadway sites in Texas by a process called static weighing, which involved the arduous task of selecting trucks from the roadway and weighing them on portable wheel-load weighers [Ref 16]. After 1967, the process was

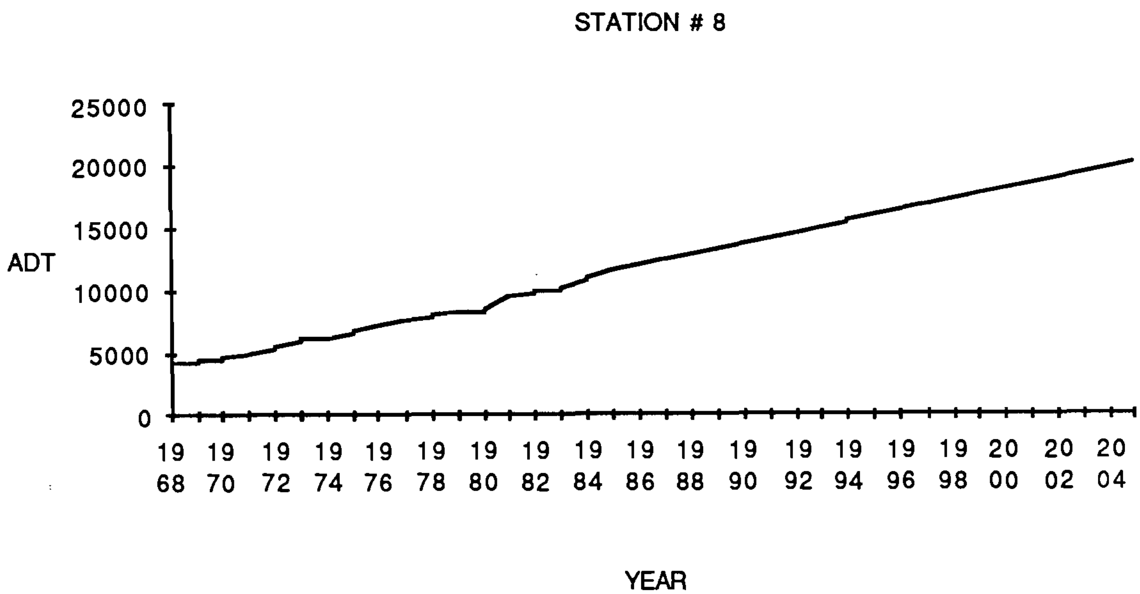


Figure 5-1. Average daily traffic for Station Number 8 (1968-2005).

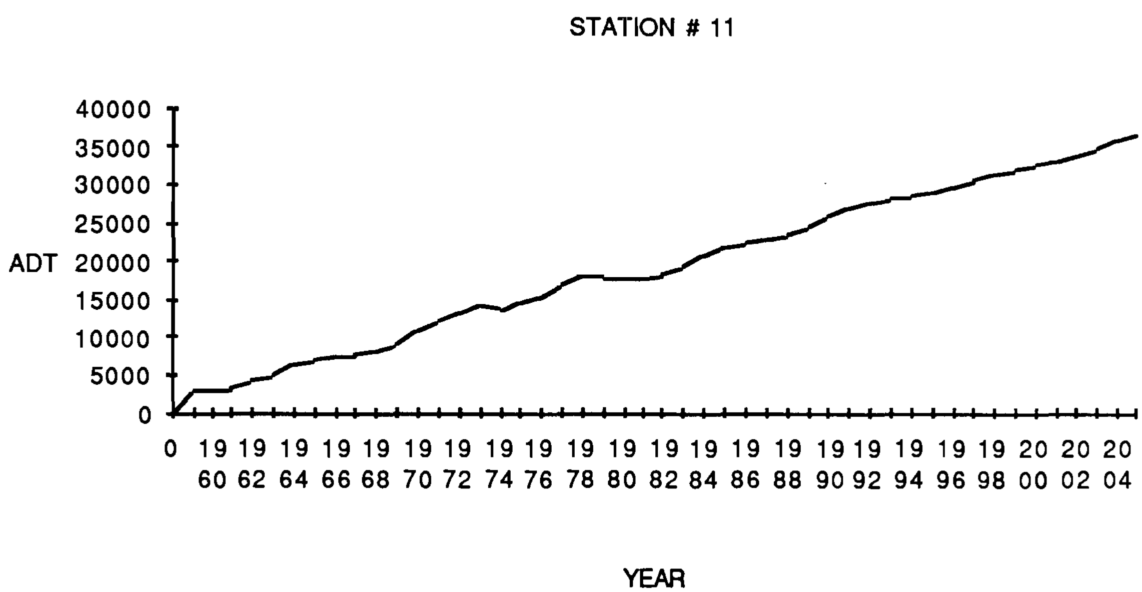


Figure 5-2. Average daily traffic for Station Number 11 (1959-2005).

reduced to sampling vehicles only during certain parts of the year, and financial considerations reduced the number of static weighing stations by 1975 to 10.

In the early 60's, research was conducted on a new in-motion weighing program, which was effective in dynamically weighing vehicles. This system became operative on a limited basis by 1971. By 1975, the in-motion system, called the Weigh-In-Motion (WIM) system, was developed to the point that it rendered the time and labor intensive static weighing process obsolete.

In 1974, a study was conducted by Machemehl, Lee and Walton, which pointed out that the 21 original stations could be combined into six groups, in such a way that weight data from any station in a selected group would not be statistically different from any other of the same group [Ref 16]. Thus six locations were selected, one from each group, and the WIM systems were installed at these locations. These stations are numbered serially from numbers 501 through 506.

Procedure Outline

Truck weight data collected at the six WIM stations in Texas were linked to the selected "test" sites based on the percentage classification of vehicles (explained in the previous chapter). These data were categorized by vehicle type and broken down into different weight intervals to form weight distribution sets, which were forecasted to the year 2005, with the help of a computerized shifting program called 'SHIFTIN'. The detailed procedure is explained below.

General Procedure

The truck weight data (raw data), supplied by the State Department of Highways and Public Transportation, contain the weights of trucks (GVW & AW's) and other relevant information gathered from the six WIM stations. With the help of the statistical software SAS, the raw data was analyzed, sorted and subdivided into smaller data sets that contain important elements for further augmentation of the forecasting process. These newly formed SAS data sets, called primary data sets, contain weight data categorized by

1. Station (501-506).
2. VehicleType (i.e. ,2D, 3A, 2S1, 2S2, 3S1, 3S2, 2S12 & 3S12).
3. Axle Weights.
4. Gross Vehicle Weights.

The primary data sets incorporate data from the six selected WIM stations. These data sets are apportioned into smaller data sets by vehicle type, called secondary data sets, which are further divided into different truck-weight distribution intervals, i.e. 5-10 kips, 10-15kips, etc. The data sets formed from this process are called final data sets. An example of this is seen in Table 5-1, which contains the number of trucks that lie within the different weight-distribution intervals. The final data set also contains frequency, percentage and cumulative percentage of trucks under each weight interval. A plot of weight vs. frequency of trucks is presented in Fig 5-3.

The final data sets were then forecasted by a computerized shifting procedure to the year 2005. To get a better forecast, the truck-weight distribution data was divided into light trucks and heavy trucks. The reason being, that some of the trucks run either empty or half-full, and thus produce large variations in the mean and variance of the truck-weight distribution data. Another reason being that seasonal effects of truck loads (commodities) are not considered while the data is collected. The forecasting of truck-weight distributions, called shifting, is performed with the help of a procedure that was developed by Walton et al [Ref 22].

SHIFTING PROCEDURE

Introduction

The truck weight shifting methodology for predicting highway loads can be performed either manually or with the help of a computer. For this analysis, the computerized method was used. There are two different applications of the shifting procedure, namely,

1. shifting the weight distribution to a future period, with the present truck weight limits remaining the same, and
2. shifting the distribution to a future period, conforming to a change in the present weight limits.

For this study, it was assumed that there was no weight limit change during the forecasted period i.e. to the year 2005.

The flow chart of the shifting procedure is shown in Fig 5-5. As seen in the figure, the procedure is divided into two parts, namely :

(a) Predicting the average weight for a selected truck type.

TABLE 5-1. WEIGHT FREQUENCY DISTRIBUTION FOR 3S2 CLASS VEHICLES

WEIGHT INTERVAL (KIPS)	MID-POINT	FREQUENCY	PERCENT	CUMULATIVE PERCENT
0 - 4.99	2.5	0	00.00	00.00
5 - 9.99	7.5	0	00.00	00.00
10 - 14.99	12.5	16	03.27	03.27
15 - 19.99	17.5	49	10.00	13.27
20 - 24.99	22.5	84	17.14	30.41
25 - 29.99	27.5	134	27.35	57.76
30 - 34.99	32.5	105	21.43	79.18
35 - 39.99	37.5	57	11.63	90.82
40 - 44.99	42.5	35	07.14	97.96
45 - 49.99	47.5	8	01.63	99.59
50 - 54.99	52.5	1	00.20	99.80
55 - 59.99	57.5	1	00.20	100.00

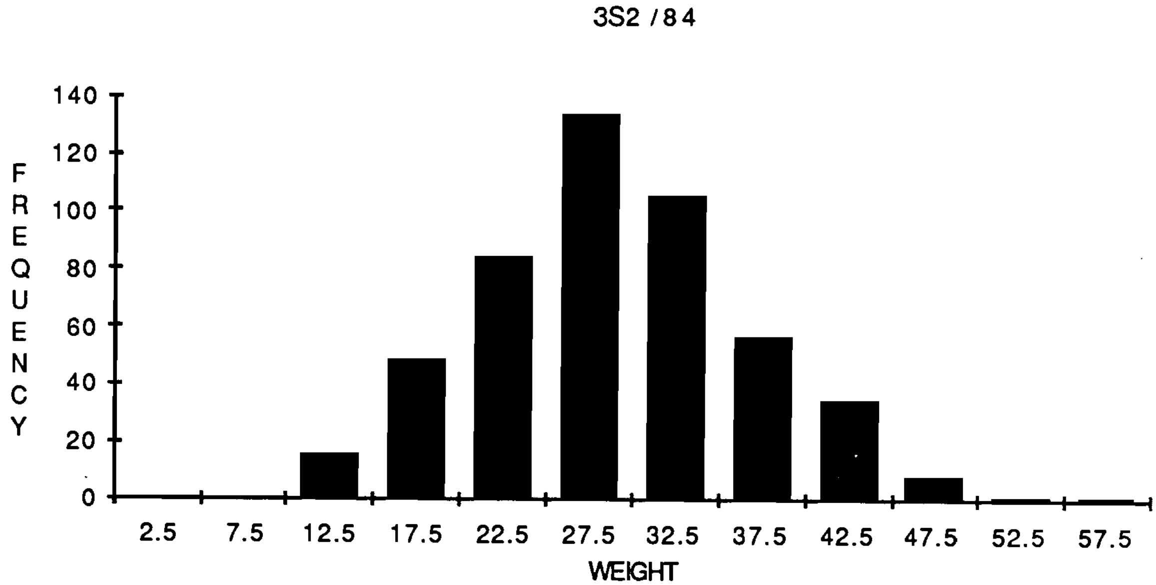


Figure 5-3. Histogram of GVW frequency distribution for truck type 3S2 (1984).

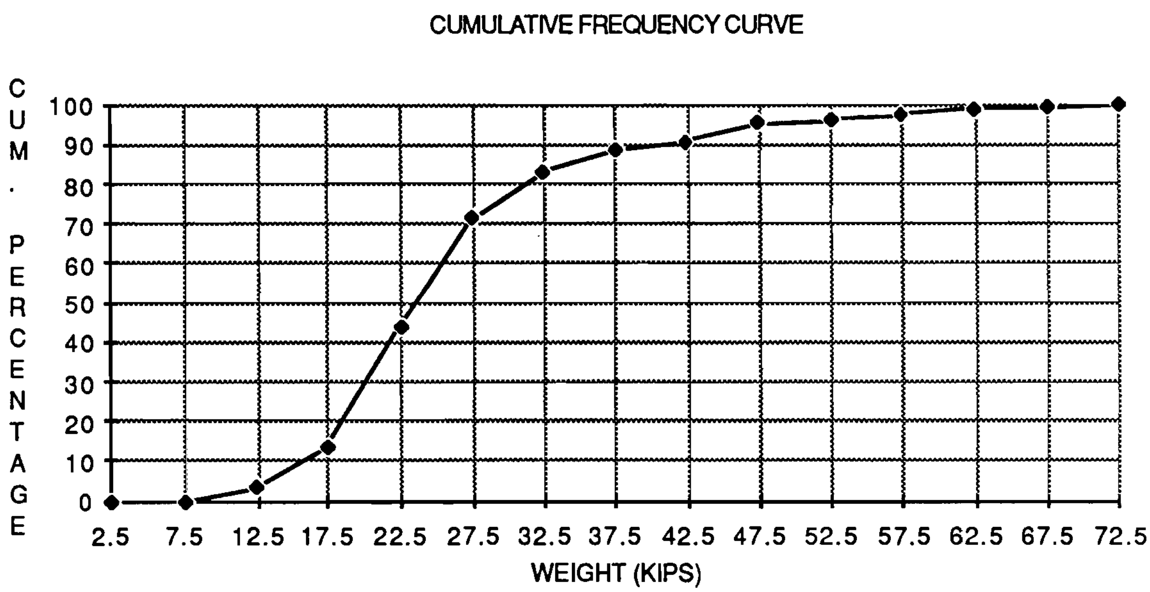
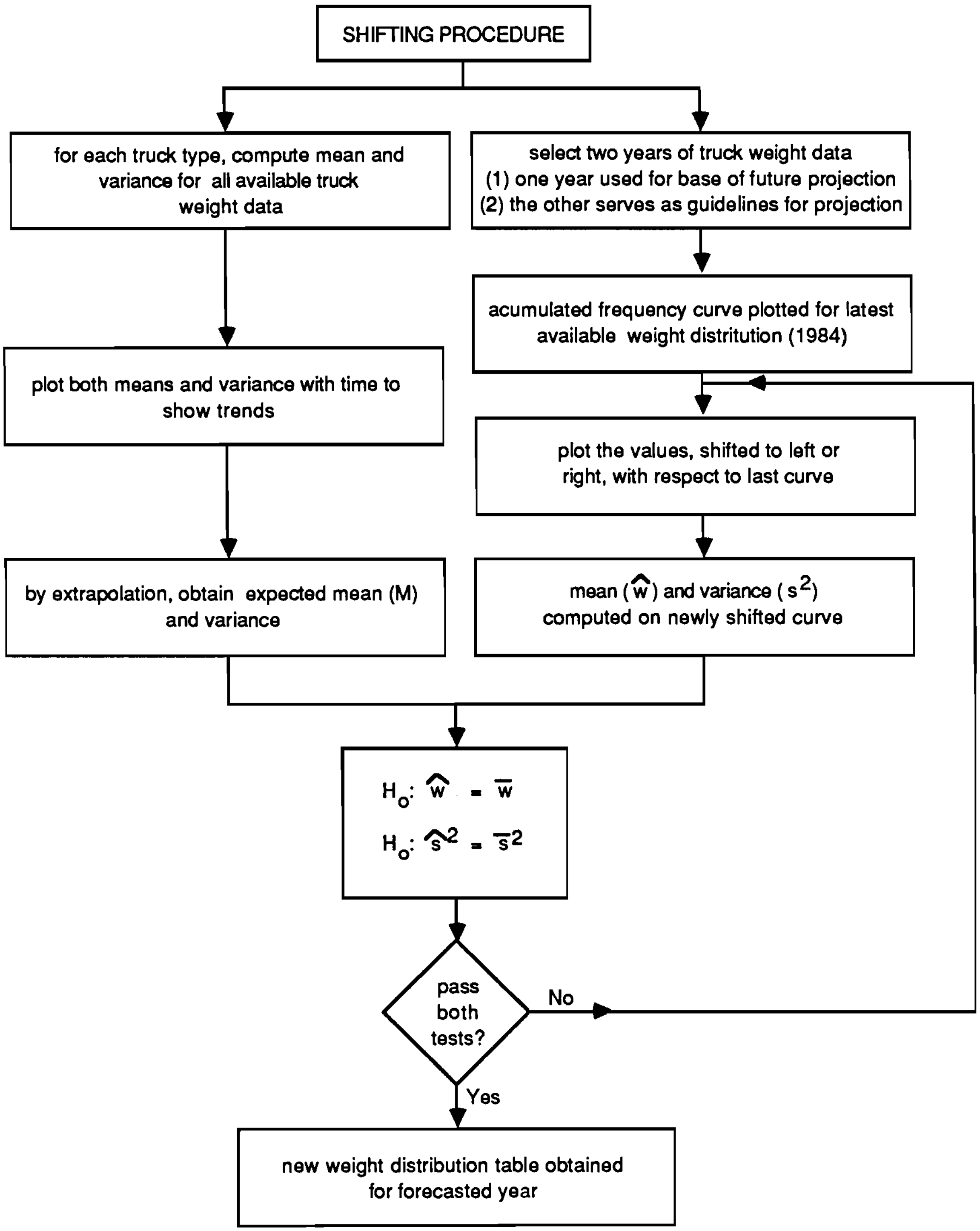


Figure 5-4. GVW cumulative frequency curve for truck type 3S2 (1984).



Figurer 5-5. Flow chart describing weight shifting procedure.

(b) Shifting the cumulative truck weight distribution curve to a new position, so that the mean of the shifted curve conforms statistically with the predicted average weight, from (a).

The shifting procedure was conducted for the following types of vehicle weights. The results of this procedure are seen in Appendix D.

1. Gross Vehicle Weight (GVW)]
   (a) Light trucks
   (b) Heavy trucks
   (c) Total (light + heavy) trucks
2. Axle Weight (AW)
   (a) Single axles
   (b) Tandem axles

A methodology for shifting total GVW's was also considered. The procedure is similar for the other types of vehicle weights mentioned above.

Procedure

The newly-formed final data sets are taken for 1984 and plotted by truck type. In general, the plotted truck-weight distribution data resembles a normal distribution pattern. Based on this, both the mean and variance can be used as estimators. The cumulative percentage of trucks for each of the truck-weight distribution intervals were plotted to form a cumulative frequency curve (Fig 5-4). As stated before, the shifting procedure is divided into two parts.

1. Predicting the average GVW for a selected truck type: The mean or average GVW of the distribution is the first parameter to be estimated. Historical data of the average GVW's for each distribution is plotted with average GVW against time (year) (eg. Fig 5-6). The trend over the years is observed. Since the trend of the mean (from 1974-1984) deviates very little over time, a linear regression line is fitted to the observed data, with the help of which the average GVW for the proposed year, 2005 is obtained. In a similar manner, historical data of the variance, for each distribution, is plotted with variance against time (see Fig 5-7). The trend over the years is observed. Because of the lack of sufficient data, the trend is extrapolated and the new variance for year 2005 is obtained.

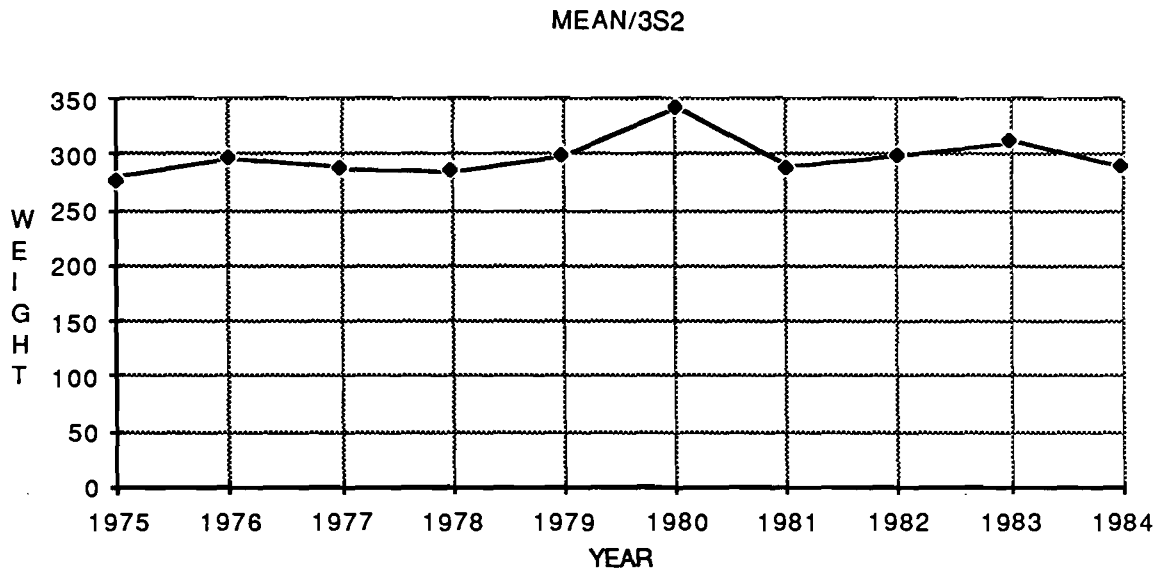


Figure 5-6. Trends of mean of GVW distribution for truck type 3S2 (1984).

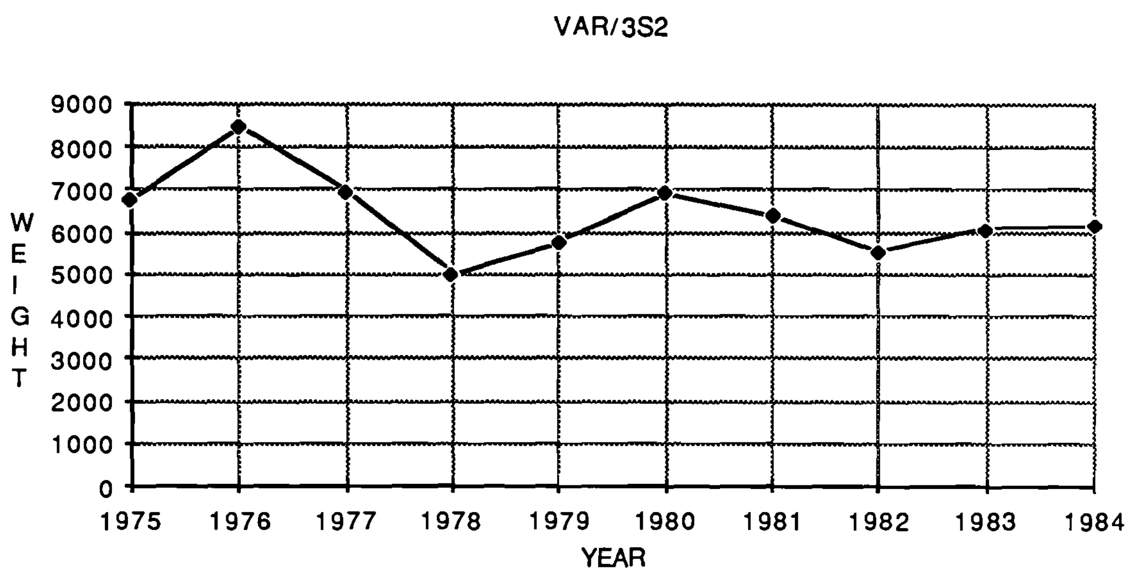


Figure 5-7. Trends of variance of GVW distribution for truck type 3S2 (1984).

2. Shifting the truck weight distribution to a new position: A computer program called "Shiftin" is used in the shifting procedure. The 1984 cumulative distribution curve is shifted to conform to the newly estimated mean and variance, as derived from part 1. The computer program uses an iterative method to move the curve such that both the expected mean and the variance of the newly shifted curve are within acceptable limits. A chi-square test is used to either accept or reject the new cumulative distribution curve within a 95 percent confidence interval [Ref 22].

An example of the results of the shifting procedure is shown in Table 5-2. The diagramatic representation of Table 5-2 is seen in Fig 5-8, which shows the comparison of cumulative weight distributions between the present year, 1984 and the forecasted year 2005. Refer to Appendix F for the results of the above mentioned shifting procedure, for the different truck types, axle and gross vehicle weights.

FORECASTING OF VEHICLE CLASSIFICATION DATA

After having forecasted the Annual Daily Traffic (ADT's) and truck weight distributions, it was neccessary to obtain the forecasted values of vehicle classification data, which is the third important element required. This, in turn, would provide the percentage mix of traffic for the forecasted year 2005.

As explained before, there are eight different types of vehicles that constitute the majority of vehicles operating on the Texas Highway Network. Vehicle counts, that were classified under the eight different categories (in percentage of vehicles), were forecasted at each of the WIM stations (501 - 506) to the year 2005. This data were grouped by station and under each class category for the years that data were available. Historical data of classification counts for the above mentioned stations were available from 1978 to 1984. Tables 5-3 thru 5-8 describes the grouping of data in percentage of vehicles.

Methodology

As an example of the methodology involved in the forecasting of classification data, refer to the data collected at WIM Station 506. Different graphs were plotted with 'vehicle type' as the abscissa and 'year 'as the ordinate for the different classes of vehicles. The number of data points available were insufficient to use any sophisticated technique, such as time series analysis, for the forecasting

TABLE 5-2. SHIFTING OF GVW DISTRIBUTION FOR TRUCK TYPE 3S2 (1984-2005)

MID-POINT OF GVW (KIPS)	3S1/84	PERCENT	CUMULATIVE PERCENT	3S1/05	PERCENT	CUMULATIVE PERCENT
2.5	0	0.00	0.00	2	0.41	0.41
7.5	0	0.00	0.00	3	0.61	1.02
12.5	16	3.27	3.27	19	3.88	4.90
17.5	49	10.00	13.27	31	6.33	11.22
22.5	84	17.14	30.41	33	6.73	17.96
27.5	134	27.35	57.76	53	10.82	28.78
32.5	105	21.43	79.18	109	22.24	51.02
37.5	57	11.63	90.82	88	17.96	68.98
42.5	35	7.14	97.96	59	12.04	81.02
47.5	8	1.63	99.59	52	10.61	91.63
52.5	1	0.20	99.80	41	8.37	100.00
57.5	1	0.20	100.00			100.00

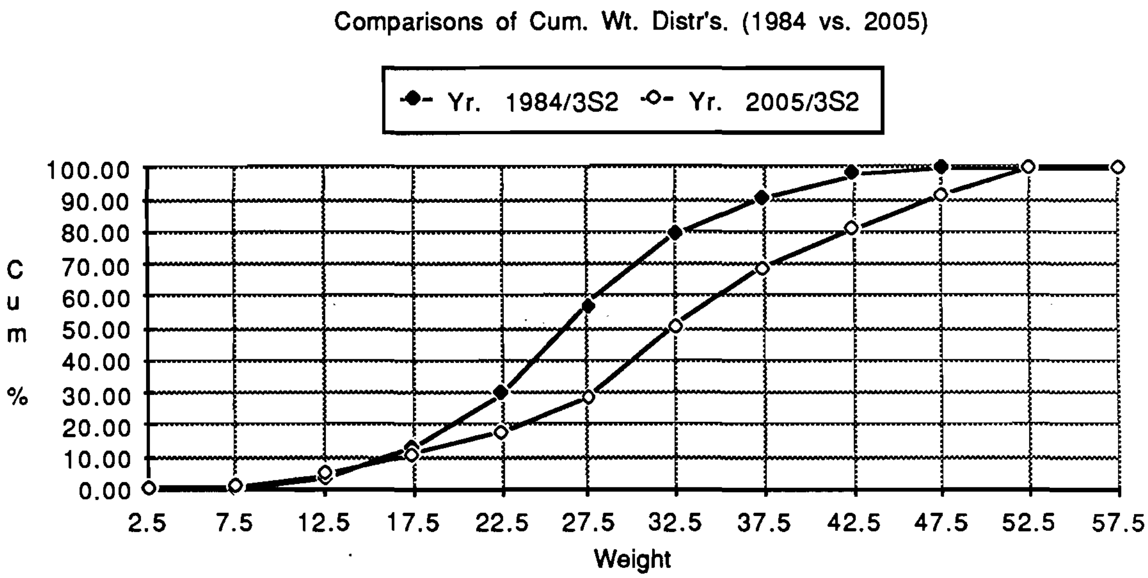


Figure 5-8. Comparison of cumulative GVW distribution for truck type 3S2 (1984-2005).

TABLE 5-3. PERCENTAGE OF VEHICLES UNDER EACH VEHICLE TYPE AT WIM STATION NUMBER 501 (1981-1984)

YEAR	VEHICLE CLASSIFICATION							
	CLASS 1	CLASS 2	CLASS 3	CLASS 4	CLASS 5	CLASS 6	CLASS 7	CLASS 8
1981	35.5	0.1	0.9	0.5	0.9	0	11.7	0.4
1982	84.9	0.5	0.8	0.6	0.7	0.1	12.1	0.3
1983	86.1	0.1	0.8	0.6	1	0	11.3	0.2
1984	85.5	0.7	0.5	0.3	0.6	0	12.1	0.3

TABLE 5-4. PERCENTAGE OF VEHICLES UNDER EACH VEHICLE TYPE AT WIM STATION NUMBER 502 (1978-1984)

YEAR	VEHICLE CLASSIFICATION							
	CLASS 1	CLASS 2	CLASS 3	CLASS 4	CLASS 5	CLASS 6	CLASS 7	CLASS 8
1978	81.7	0.3	0.7	0.7	1.1	0	14.1	0.7
1979	75.4	0.2	0.7	0.9	1.2	0	18.7	0.75
1980	68.6	0.1	0.7	1.1	2.3	0	26.5	0.8
1981	69	0.5	0.85	1	2	0	26	0.8
1982	69.7	0.1	1	0.9	1.7	0	25.7	0.8
1983	79.6	0.3	0.6	0.4	1.2	0	17.4	0.5
1984	77.9	0.1	0.8	0.4	1.2	0	18.9	0.6

TABLE 5-5. PERCENTAGE OF VEHICLES UNDER EACH VEHICLE TYPE AT WIM STATION NUMBER 503 (1978-1984)

YEAR	VEHICLE CLASSIFICATION							
	CLASS 1	CLASS 2	CLASS 3	CLASS 4	CLASS 5	CLASS 6	CLASS 7	CLASS 8
1978	89.1	0.2	0.5	0.3	1.1	0	8.4	0.2
1979	89.7	0.2	0.7	0.3	0.4	0	8.4	0.3
1980	82.3	0.2	0.5	0.2	1	0	15.3	0.5
1981	73.3	0.2	0.5	0.3	1.5	0	23.8	0.4
1982	71.5	0.4	0.9	0.3	1.2	0	25.2	0.4
1983	78.2	0.1	0.5	0.3	1.1	0.1	19.4	0.4
1984	74.1	0.8	1	0.2	1.7	0	21.9	0.4

TABLE 5-6. PERCENTAGE OF VEHICLES UNDER EACH VEHICLE TYPE AT WIM STATION NUMBER 504 (1978-1984)

YEAR	VEHICLE CLASSIFICATION							
	CLASS 1	CLASS 2	CLASS 3	CLASS 4	CLASS 5	CLASS 6	CLASS 7	CLASS 8
1978	75.8	0.2	1	0.6	1.3	0.3	19.6	1
1979	75.1	0.1	0.6	0.8	1.1	0.2	21	1.2
1980	78.3	0.1	0.6	0.3	0.5	0.1	19.2	0.8
1981	73	0.2	1.1	0.5	1.5	0	22.6	1.1
1982	71	0.2	1.3	0.6	1.5	0	24.4	0.9
1983	74.6	0.1	0.8	0.5	1.3	0	21.9	0.7
1984	71.1	0.6	0.9	0.8	1.2	0.1	24.2	1.1

TABLE 5-7. PERCENTAGE OF VEHICLES UNDER EACH VEHICLE TYPE AT WIM STATION NUMBER 505 (1978-1984)

YEAR	VEHICLE CLASSIFICATION							
	CLASS 1	CLASS 2	CLASS 3	CLASS 4	CLASS 5	CLASS 6	CLASS 7	CLASS 8
1978	81.9	0.2	0.7	0.3	1.3	0	15.4	0.1
1979	81.4	0.3	0.7	0.3	1.5	0	15.7	0.2
1980	86.4	0.3	0.4	0.2	0.7	0	11.8	0.2
1981	82.8	0.3	0.6	0.2	1.2	0	14.7	0.1
1982	84.5	0.2	0.6	0.3	0.8	0	13.5	0.1
1983	81.3	0.2	0.6	0.3	0.7	0	16.9	0.1
1984	82.2	0.2	0.5	0.3	0.9	0.1	15.7	0.2

TABLE 5-8. PERCENTAGE OF VEHICLES UNDER EACH VEHICLE TYPE AT WIM STATION NUMBER 506 (1978-1984)

YEAR	VEHICLE CLASSIFICATION							
	CLASS 1	CLASS 2	CLASS 3	CLASS 4	CLASS 5	CLASS 6	CLASS 7	CLASS 8
1978	86.9	0.1	0.6	0.2	0.6	0	11.4	0.1
1979	80.5	0.15	0.8	0.4	0.8	0	18.6	0.8
1980	70.3	0.2	1	0.7	1.2	0	25.5	1.2
1981	74.8	0.2	0.8	0.5	1	0.1	21.4	1.2
1982	78	0.2	0.7	0.5	1	0	18.8	0.8
1983	74.5	0.1	0.6	0.2	1.7	0	21.5	1.4
1984	72.7	0.4	1.2	0.3	1.2	0	22.8	1.4

of vehicle classification data for a long term period (20 years in our case). Thus a simple linear regression analysis was performed on each of the plots. The independent variable, sometimes called the regressor, is the year and the dependent variable, sometimes called the regressand, is the vehicle type (in percentage of vehicles). The simple linear regression is of the form

$$Y = a + bX$$

where
- Y = regressand, percentage vehicles
- X = regressor, year
- a = value on Y axis when X equals 0
- b = slope of the line

Based on the equations that resulted for each vehicle type at each of the selected WIM stations, forecasts were made to the year 2005. The results were tabulated and regrouped to represent the percentage of vehicles at each of the respective WIM stations.

The linear regression equations produced a few negative values, but appropriate corrections were made, based on the minimum values of each vehicle class as observed from Tables 5-5 thru 5-8. The present classification percentage data (year 1984) and the forecasted values to the year 2005 are tabulated in Table 5-9.

PROCEDURE INVOLVED IN OBTAINING THE "1974 EQUIVALENT NUMBER OF TRUCKS " FROM 1984 LOAD DATA

As mentioned earlier in the chapter, the procedure described below describes a method for estimating the number of trucks required to carry 1984 cargo quantities if none was loaded beyond current legal limits. Weight distributions for 1974 were chosen because prior to this time, legal limits were lower and historical data indicated that pre-1975 weight distributions exhibited almost no overweight vehicles if post 1975 vehicle weight laws were the basis for judging legality. The analysis revealed significant results, which will be discussed in greater detail further in the chapter. Described below is a brief summary of the change in weight laws, and a detailed description of the procedure used for obtaining the 1974 equivalent number of trucks from 1984 load data.

Concerning the maximum weight limit of trucks, the Federal-Aid Highway Amendments of 1974 established new vehicle weight limitations for the Interstate System. The changes were as follows:

TABLE 5-9. PERCENTAGE OF VEHICLES UNDER EACH VEHICLE TYPE AT WIM STATIONS 501-506 FOR THE YEARS 1984 AND 2005

CLASS/ STATION (1984)	NO. 501	NO. 502	NO. 503	NO. 504	NO. 505	NO. 506
Class 1	85.5	77.9	74.1	71.1	82.2	72.7
Class 2	0.7	0.1	0.8	0.6	0.2	0.4
Class 3	0.5	0.8	0.9	0.9	0.5	1.2
Class 4	0.3	0.4	0.2	0.8	0.3	0.3
Class 5	0.6	1.3	1.7	1.2	0.9	1.2
Class 6	0	0	0	0.1	0.1	0
Class 7	12.1	18.9	21.9	24.2	15.6	22.8
Class 8	0.3	0.6	0.4	1.1	0.2	1.4

CLASS/ STATION (2005)	NO. 501	NO. 502	NO. 503	NO. 504	NO. 505	NO. 506
Class 1	82.9	68.56	18.79	54.2	79.17	37.64
Class 2	3.29	0.5	1.87	0.42	0.2	0.95
Class 3	0.5	1.1	2	1.62	0.15	1.81
Class 4	0.3	0.4	0.2	1.42	0.37	0.2
Class 5	0.6	1.26	4.06	2.09	0.7	4.02
Class 6	0	0	0	0.1	0.85	0.48
Class 7	12.2	27.88	72.11	39.64	18.42	49.98
Class 8	0.21	0.3	0.97	0.51	0.14	4.92

LOAD	PRE'1975 (LBS)	POST'1975 (LBS)
Single Axle Loads	18000	20000
Tandem Axle Loads	32000	34000
Gross Vehicle Weights	73280	80000

For 1984 the number of vehicles for each truck type, and the net load they carried, at each ATR location was computed. Empty weights were estimated from historical records of field weighing of typical trucks of each class. After having obtained the net load and the number of trucks for 1984, corresponding numbers of 1974 trucks of each type that would be required to carry the same net load but within 1974 weight laws was computed. A comparison was then made between the number of trucks required under 1974 and 1984 weight laws.

The analysis revealed significant differences in the number of trucks. The percentage difference ranged from 11 percent to 150 percent. This clearly indicates an increasing trend in truck load-carrying capacity. A listing of the resulting number of vehicles are shown in Table 5-10.

PROCEDURE SUMMARY

This analysis was conducted separately for each truck type. Gross Vehicle Weight distributions for seven of the most predominant truck types for 1974 and 1984 data are shown in Table 5-12 and 5-13. Percentages of these truck types counted through SDHPT classification counts during 1984 and are shown for the 14 count stations chosen as surrogates for the 14 test sections in Table 5-14. As an example, consider truck type 2D. The data includes the total number of trucks (type 2D), counted at each selected ATR station. The net load carried is calculated by subtracting the assumed empty weights of trucks from the observed GVW. This, in effect, produces the net load carried by truck type 2D in the year 1984, at each of the stations, for the different selected stations. The next step involves the calculation of the number of 1974 trucks that would carry the same payload. The total number of trucks carrying this previously calculated load is obtained by equating the total net load carried by 1974 trucks (in terms of an unknown, say X), to that carried by 1984 trucks. The procedure is explained below.

Note, that Table 5-11 refers to the procedures involved with truck data in the year 1984, and Table 5-14 refers to the procedures involved with truck data in the year 1974. Described below is a simple worked example to obtain the 1974 equivalent number of trucks. The 1974 equivalent number of trucks (for truck type 2D and station 1) are obtained by equating :

TABLE 5-10. PERCENTAGE DIFFERENCE IN THE NUMBER OF TRUCKS BETWEEN 1984 AND 1974, NEEDED TO CARRY THE SAME PAYLOAD

TRUCK TYPE	THEORETICAL PERCENTAGE INCREASE IN NUMBER OF TRUCKS 1974 TO 1984
2D	20%
3A	58%
2S1	150%
2S2	11%
3S2	51%
2/3S12	47%

TABLE 5-11. PROCEDURE TO OBTAIN THE NET LOAD (PAYLOAD) CARRIED BY TRUCK TYPE 2D BY STATION (1984)

MIDPOINT (KIPS)	EMPTY WEIGHT (KIPS)	PAYLOAD (KIPS)	NUMBERS 2D'S WEIGHED 1984	PERCENT 2D'S WEIGHED 1984	STATION (2D PAYLOADS, 1984)															
					1		2		3		3a		4		5		6		7	
					NUMBER	KIPS	NUMBER	KIPS	NUMBER	KIPS	NUMBER	KIPS	NUMBER	KIPS	NUMBER	KIPS	NUMBER	KIPS	NUMBER	KIPS
b	c	d	e	f	g	h	i	j	k	l	m	n	o	p	q	r	s	t	u	v
2.5	9.76	0	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.5	9.76	0	369	12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
12.5	9.76	2.74	1087	37	24	66	41	112	98	269	2295	6288	123	337	102	279	326	893	293	803
17.5	9.76	7.74	845	29	14	107	24	185	57	439	1327	10269	71	547	59	455	189	1459	169	1311
22.5	9.76	12.74	445	15	7	92	13	161	30	381	699	8902	37	474	31	394	99	1265	89	1136
27.5	9.76	17.74	190	6	3	55	5	95	13	226	298	5292	16	282	13	235	42	752	38	675
32.5	9.76	22.74	16	1	0	6	0	10	1	24	25	571	1	30	1	25	4	81	3	73
37.5	9.76	27.74	1	0	0	0	0	1	0	2	2	44	0	2	0	2	0	6	0	6
42.5	9.76	32.74	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
47.5	9.76	37.74	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
52.5	9.76	42.74	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
57.5	9.76	47.74	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
62.5	9.76	52.74	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

DESCRIPTION:

Column D: Contains the net weights or pay loads (gross vehicle weight minus empty vehicle weight) i.e., column (b-c)
Column E: Contains a listing of the total number of trucks under each specific weight category (values from Table 5-12)
Column F: Contains a listing of the percentages of column e
Column G: This column refers to station 1. It contains a listing of the number of trucks under each specific category. These values were produced by multiplying the ADT of station 1 (from Table E-1) with the percentages of trucks, truck type 2D in this case (Table 5-14), and column f, i.e., column g = ADT(station 1) * (%) of 2D trucks * column f.
Column H: Contains net total weight, obtained by multiplying columns d and g

continued

TABLE 5-11. CONTINUED

MIDPOINT (KIPS)	EMPTY WEIGHT (KIPS)	PAYLOAD (KIPS)	NUMBERS 2D'S WEIGHED 1984	PERCENT 2D'S WEIGHED 1984	STATION (2D PAYLOADS, 1984)													
					8		9		10		11		12		13		14	
					NUMBER	KIPS	NUMBER	KIPS	NUMBER	KIPS	NUMBER	KIPS	NUMBER	KIPS	NUMBER	KIPS	NUMBER	KIPS
w	x	y	z	AA	BB	CC	DD	EE	FF	GG	HH	II	JJ	KK	LL	MM	NN	OO
2.5	9.76	0	6	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7.5	9.76	0	369	12	0	0	0	0	0	0	0	0	0	0	0	0	0	0
12.5	9.76	2.74	1087	37	163	447	269	737	231	633	413	1132	128	351	186	510	842	1716
17.5	9.76	7.74	845	29	94	727	155	1203	134	1036	238	1844	74	571	107	831	487	3769
22.5	9.76	12.74	445	15	49	630	82	1043	70	898	125	1599	39	495	57	720	256	3267
27.5	9.76	17.74	190	6	21	375	35	620	30	534	54	950	17	294	24	428	109	1942
32.5	9.76	22.74	16	1	2	40	3	67	3	58	5	103	1	32	2	46	9	210
37.5	9.76	27.74	1	0	0	3	0	5	0	4	0	8	0	2	0	4	1	16
42.5	9.76	32.74	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
47.5	9.76	37.74	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
52.5	9.76	42.74	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
57.5	9.76	47.74	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
62.5	9.76	52.74	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

DESCRIPTION:

Column Y: Contains the net weights or pay loads (gross vehicle weight minus empty vehicle weight) i.e., column (w-x)
Column Z: Contains a listing of the total number of trucks under each specific weight category (values from Table 5-12)
Column AA: Contains a listing of the percentages of column z
Column BB: This column refers to station 1. It contains a listing of the number of trucks under each specific category. These values were produced by multiplying the ADT of station 1 (from Table E-1) with the percentages of trucks, truck type 2D in this case (Table 5-14), and column AA, i.e., column BB = ADT(station 1) * (%) of 2D trucks * column AA.
Column CC: Contains net total weight, obtained by multiplying columns y and BB

TABLE 5-12. GVW DISTRIBUTION FOR THE DIFFERENT TRUCK TYPES (1984)

MIDPOINT KIPS	YEAR 1984						
	2D	'3A	2S1	2S2	3S1	3S2	2/3S12
2.5	6	0	0	0	0	0	0
7.5	369	9	0	0	0	0	0
12.5	1087	46	16	3	10	17	0
17.5	845	196	49	28	29	131	0
22.5	445	178	84	118	84	269	0
27.5	190	134	134	293	76	1453	4
32.5	16	89	105	291	32	3491	6
37.5	1	77	57	190	16	2376	10
42.5	0	55	35	210	6	1507	11
47.5	0	35	8	145	12	1299	8
52.5	0	17	1	77	3	1267	11
57.5	0	3	1	26	3	1311	14
62.5	0	2	0	17	4	1533	18
67.5	0	0	0	4	1	2407	15
72.5	0	0	0	4	2	3117	13
77.5	0	0	0	1	0	2432	16
82.5	0	0	0	1	0	1006	3
87.5	0	0	0	0	0	324	4
92.5	0	0	0	0	0	139	0
97.5	0	0	0	0	0	41	0
102.5	0	0	0	0	0	12	0
107.5	0	0	0	0	0	9	0
112.5	0	0	0	0	0	2	0
117.5	0	0	0	0	0	1	0

TABLE 5-13. PERCENTAGE GVW DISTRIBUTION OF THE DIFFERENT TRUCK TYPES

MIDPOINT KIPS	YEAR 1974						
	2D	'3A	2S1	2S2	3S1	3S2	2/3S12
2.5	0	0	0	0	0	0	0
7.5	30	2	0	0	0	0	0
12.5	34	20	4	2	0	0	0
17.5	19	33	16	15	0	1	0
22.5	10	14	25	16	14	16	1
27.5	5	14	21	13	57	15	8
32.5	1	6	22	11	0	9	13
37.5	0	5	8	11	0	4	7
42.5	0	3	3	10	14	4	5
47.5	0	0	2	10	14	6	13
52.5	0	1	0	9	0	7	15
57.5	0	2	0	3	0	12	14
62.5	0	0	0	1	0	11	13
67.5	0	0	0	0	0	6	7
72.5	0	0	0	0	0	3	3
77.5	0	0	0	0	0	1	1
82.5	0	0	0	0	0	1	1
87.5	0	0	0	0	0	1	0

TABLE 5-14. PERCENTAGE OF VEHICLE CLASSIFICATION AT THE DIFFERENT SELECTED STATIONS (1984)

VEHICLE CLASS	STATION 1		STATION 2		STATION 3		STATION 3A		STATION 4	
	NO.VEH.	PERCENT	NO. VEH.	PERCENT	NO. VEH.	PERCENT	NO. VEH.	PERCENT	NO. VEH.	PERCENT
Cars	735	79	1825	80	3388	86	4217	91	7786	94
T 2D	49	5	105	5	183	5	238	5	214	3
T 3A	0	0	15	1	21	1	15	0	29	0
T 2S1	0	0	6	0	7	0	9	0	10	0
T 2S2	10	1	30	1	39	1	18	0	33	0
T 3S1	0	0	1	0	1	0	1	0	0	0
T 3S2	141	15	282	12	272	7	129	3	143	2
T 2/3S12	1	0	4	0	18	0	19	0	31	0
TOTAL	936	100	2268	100	3929	100	4646	100	8246	100

continued

TABLE 5-14. CONTINUED

VEHICLE	STATION 5		STATION 6		STATION 7		STATION 8		STATION 9	
CLASS	NO. VEH.	PERCENT	NO. VEH.	PERCENT	NO. VEH.	PERCENT	NO. VEH.	PERCENT	NO. VEH.	PERCENT
Cars	6471	94	15142	93	11506	79	9742	89	10770	81
T 2D	192	3	702	4	525	4	359	3	551	4
T 3A	14	0	195	1	67	0	99	1	129	1
T 2S1	8	0	20	0	46	0	12	0	17	0
T 2S2	25	0	18	0	130	1	78	1	133	1
T 3S1	0	0	8	0	10	0	4	0	2	0
T 3S2	144	2	254	2	2295	16	592	5	1747	13
T 2/3S12	3	0	3	0	26	0	9	0	27	0
TOTAL	6857	100	16342	100	14605	100	10895	100	13376	100

continued

TABLE 5-14. CONTINUED

VEHICLE CLASS	STATION 10		STATION 11		STATION 12		STATION 13		STATION 14	
	NO. VEH.	PERCENT	NO. VEH.	PERCENT	NO. VEH.	PERCENT	NO. VEH.	PERCENT	NO. VEH.	PERCENT
Cars	7024	61	18506	85	7592	89	9974	71	48654	86
T 2D	471	4	806	4	222	3	467	3	1815	3
T 3A	81	1	98	0	53	1	86	1	1406	2
T 2S1	63	1	68	0	16	0	43	0	119	0
T 2S2	213	2	110	1	29	0	156	1	245	0
T 3S1	5	0	3	0	3	0	4	0	10	0
T 3S2	3605	31	2055	9	602	7	3104	22	4027	7
T 2/3S12	109	1	135	1	11	0	166	1	38	0
TOTAL	11571	100	21781	100	8528	100	14000	100	56314	100

The total net load (right most column) in Table 5-15 with the total weight of, for example, column H in Table 5-11.

The results are shown in Tables H-1 thru H-6. A final listing of the analysis is shown in Table H-7.

For example, consider Table H-1, Station 1.

Equating total net load (1984) to the total net load (1974).

$$325.93 = 5.6126x$$

Therefore,

$$x = 325.93/5.6126$$

$$x = 58$$

This is the 1974 equivalent number of trucks, carrying a total net load of 325.93 Kips.

Refer to Tables G-1 thru G-10 in Appendix G for similar details, regarding other truck types. Thus from Tables 5-11, G-1 thru G-5, 5-12 and G-6 thru G-10, one may obtain the number of trucks and the total net load carried for each category, at each of the chosen ATR locations.

SUMMARY

In this chapter, we described the forecasting of Annual Daily Traffic, and the trends of historical data for both the past and future (to the year 2005). This was followed by the forecasting of truck weight distribution, with a brief history of the truck weighting program in Texas. The methodology of shifting the distribution involved the moving of the GVW cumulative frequency curve, in relation to the forecasted mean and variance values. We also described the forecasting of vehicle classification data and a procedure in obtaining the "1974 equivalent number of trucks" from 1984 load data. The methodology produced a marked increase in the loads carried by trucks, which ranged from 11 percent to 150 percent for the different truck types.

TABLE 5-15. PROCEDURE TO OBTAIN THE NET TOTAL LOAD CARRIED BY TRUCK TYPE 2D (IN TERMS OF X), (1984)

MIDPOINT KIPS	TRUCK WEIGHT (2D)	NET WEIGHT	NUMBER OF TRUCKS (% of X)	NET TOTAL WEIGHT (in terms of X)
a	b	c	d	e
2.5	9.76	-7.26	0	0
7.5	9.76	-2.26	0	0
12.5	9.76	2.74	0.64	1.7536
17.5	9.76	7.74	0.19	1.4706
22.5	9.76	12.74	0.1	1.274
27.5	9.76	17.74	0.05	0.887
32.5	9.76	22.74	0.01	0.2274
37.5	9.76	27.74	0	0
42.5	9.76	32.74	0	0
47.5	9.76	37.74	0	0
52.5	9.76	42.74	0	0
57.5	9.76	47.74	0	0
62.5	9.76	52.74	0	0
67.5	9.76	57.74	0	0
72.5	9.76	62.74	0	0
77.5	9.76	67.74	0	0
82.5	9.76	72.74	0	0
87.5	9.76	77.74	0	0
TOTAL			0.99	5.6126

DESCRIPTION OF TABLE 5-15:

Column C: Contains the net weights or pay loads (gross vehicle weight minus empty vehicle weight), i.e., (a-b)

Column D: Contains the number of trucks (as a percentage of x (the total number of trucks), which is an unknown factor at this stage). Column d from Table 5-13.

Column E: Contains the net total weight

CHAPTER 6. DAMAGE TRANSFORMS

INTRODUCTION

For the purpose of this study, we shall define damage transforms as any system of mapping from a load (or stress) domain to a linear damage domain. Any ordinary solid body is constantly subjected to a number of internal and external forces that keeps it in equilibrium. In fact, we have no experience of any solid body that is free of all of these forces. We also know that the application of these forces provides the basis for the existence of stress within the body. Further, if it is assumed that the solid is not rigid, the application of suitable forces can make the body change its shape and size. It is the damage caused by the repeated application and withdrawal of these forces that we are studying.

When the induced changes due to a load are not too large, the body will (apparently) tend to regain its original shape and size once the applied forces are removed. However, when the changes are large the body will not, in general, regain its original configuration. It is this property (or lack thereof) of recovery that is denoted by elasticity. The changes in size and shape are expressed by specifying strains. The elastic limits of a body would then denote the levels of stress and strain beyond which the body will not regain its original configuration. Application of forces that induce levels of stress and strain larger than those at the elastic limit would result in the body going into the plastic state or rupturing. It should be noted that for any one sample, a number of elastic limits can be defined - e.g., the limits of linear elasticity or the limits of perfect elasticity etc. It should also be remembered that these limits are not permanent and can be changed by overstrain.

A body may be strained well within its elastic limits repeatedly without showing any signs of damaged. For example, a watch spring may coil and uncoil tens of millions of times a year for several years and not show any deterioration. The situation, however, is different when the body is strained repeatedly by rapidly varying loads which exceed or are close to exceeding the elastic limits of the body. It has been verified by observation that after a large number of applications and removals of a load, test bars can be broken by a stress significantly lower than their statical breaking stress. Lord Kelvin first called attention to this phenomenon under the name "fatigue of elasticity". This fatigue factor appears to follow a power law with respect to the number of occurrences of the loading event. Fatigue may be of special interest to us since it seems to be an important player in assessing damage.

One valid criticism of Hooke's law of elasticity, even within the limits of perfect elasticity, is the exclusion of hysteresis. For most materials it is observed that the strains recorded for a sample under gradually increased loads do not correspond to the strains recorded at similar load levels while unloading. The earliest accounts of this phenomenon is given by Ewing who described it as "hysterisis".

For hard metals, there seems to be no appreciable hysterisis. However, rocks such as granite show hysterisis at moderate levels of load. The general nature of the effect can be said to be that the stress-strain diagram is a closed curve. This would imply that some energy has been dissipated in putting the specimen through one cycle of loading and unloading [Ref. 14]. One of the approaches to assessing damage that is discussed later involves accounting for these dissipations by consideration of energy principles.

SCHEMES FOR DAMAGE TRANSFORMS

This section provides an synopsis of the damage transforms currently in use. Often referred to as cycle counting techniques, these methodologies assume that damage is linear with respect to the number of load cycles The object of these cycle counting methods is to obtain the relative damage effect of irregular load histories. The assumption of linear damage requires that the mean and amplitude of the stress or strain to which the damaging event is to compared should be known. Different counting methods exist for this purpose. However, one of the problems with these techniques is that the use of different counting methods could change the resulting prediction by an order of magnitude. A detailed discussion on this subject may be found in Fuchs and Stephens [Ref.7].

If it is assumed that damage is a function of the magnitude of the hysteresis loop, then the cycle must be counted with the range from the highest peak to the lowest valley. All intermediate cycles should also be counted in a manner that maximizes their range. This technique can also be justified by the consideration that in fatigue (as in many other instances) the intermediate fluctuations are less important compared to the overall differences between the high and the low points. All "good" counting methods should count each part of the cycle once and only once. There should also be a mechanism to count smaller ranges down to some predetermined threshold. Three counting methods that achieve these objectives are well documented in the literature. They are commonly referred to as range-pair, rainflow and racetrack. Each is discussed briefly here.

In the range-pair method, smaller cycles are counted first and their reversal points (peaks and valleys) are eliminated from further consideration. The procedure is repeated for the remaining cycles till only the.largest peak and valley remain. The result of this procedure is a table of the occurrence of ranges and, if desired of their mean values. Figure 6-1 has an illustrative example for this methodology. In part (a), the original stress-time curve is shown. The pairs of peaks and valleys that are hatched are counted and eliminated, leaving the stress curve looking like part (b). The procedure is repeated in a similar fashion by counting and eliminating the hatched peaks and valleys leading to the figure in part (c). This illustrates the range-pair cycle counting method attributed to Hayes.

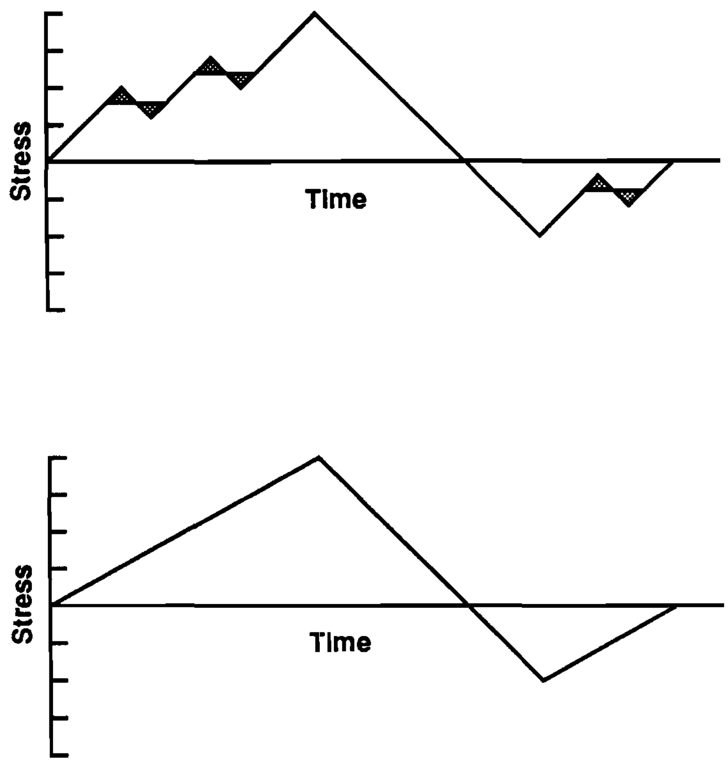


Figure 6-1. Graphical representation of the range-pair cycle counting technique (attributed to Hayes).

The operations for the rainflow method is a series of rules for counting peaks and valleys. These rules can be listed as

1. Rearrange the history to start from the highest peak.
2. Starting from the highest peak go down to the next reversal. Proceed horizontally to the next downward range. If there is no range going down from the level of the valley at which stopped, go upwards to the next reversal.
3. Repeat the same procedure upward instead of downward and continue these steps to the end.
4. Repeat the procedure for all the ranges and parts of a range that were not used in the previous steps.

Figure 6-2 illustrates the procedure of rainflow counting. The thick lines indicate the cycle being counted in each of the the two parts (a) and (b). The advantage of this procedure is manifested when it is combined with a strain analysis. The damage can be computed for each cycle as soon as it has been identified in the counting procedure and the the corresponding reversal points can be discarded. First worked out by Matsuishi and Endo, there are similar methods that accomplish the same purpose that have been recently devised by other authors as well.

The racetrack method of cycle counting is illustrated in Figure 6-3. Part (a) shows the original stress history. A "racetrack" of width S with the same profile as the original curve is constructed around the stress cycle curve. Only those reversals are counted at which a "racer" would have to change from upward to downward. The width of the race track determines the number of reversals that will be counted. Originally called the ordered overall range method, the object of this procedure is to condense a long and complex history of reversals or a large number of peaks and valleys to a single simple cycle.

All three methods can achieve the same objective of listing overall ranges in order of magnitude. The rainflow method seems to be currently the most popular method. Cycle counting is however, not the only method that can be used for assessing cumulative damage. It has been suggested that the damage produced by random loadings is proportional to the root mean sixth power of the load ranges. Since this is a statistical approach, the details which may be important in fatigue analysis, are not accounted for. It is mentioned in the literature less to recommend it than to discourage the use of this procedure.

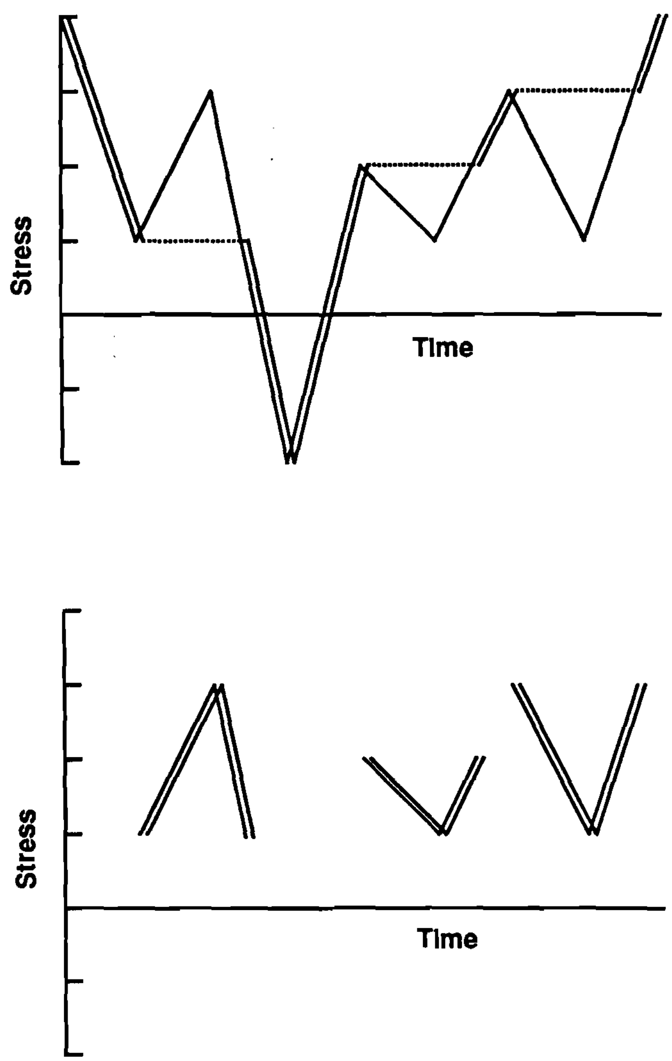


Figure 6-2. Graphical representation of the rain-flow cycle counting technique (attributed to Matsuishi and Endo).

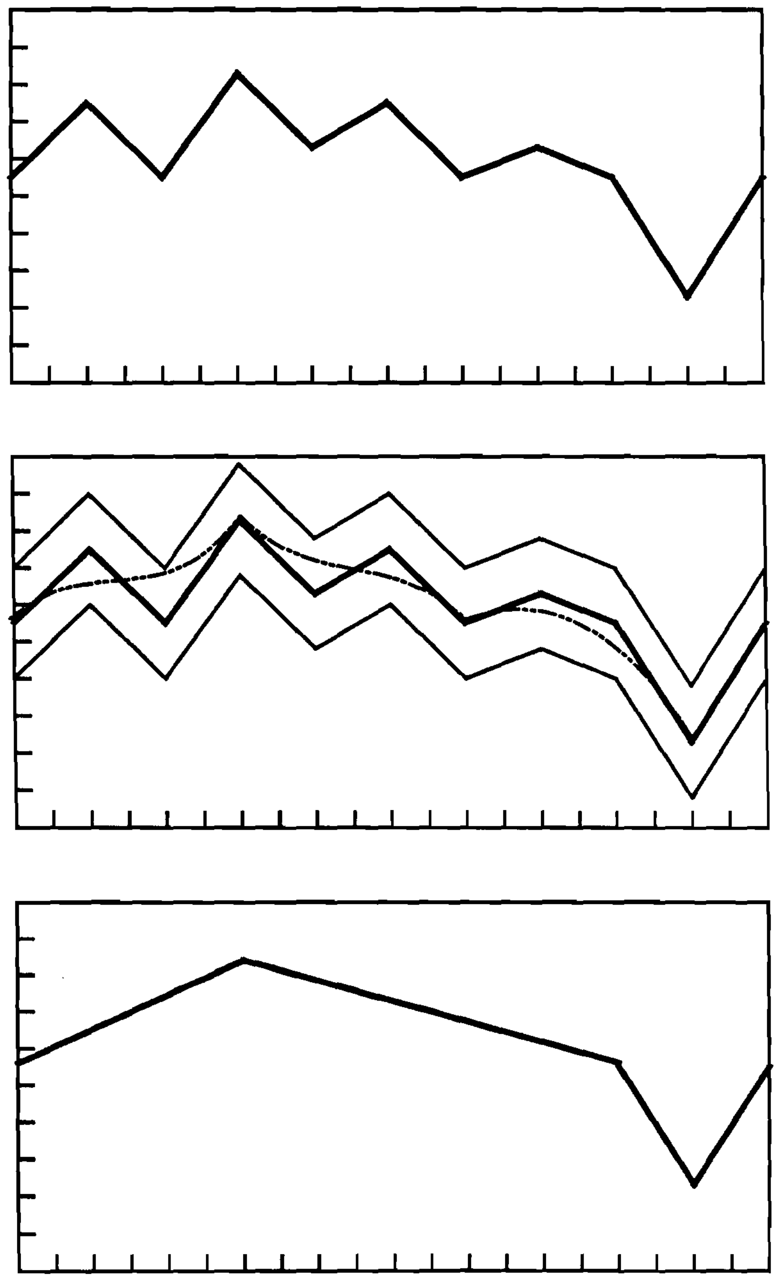


Figure 6-3. Graphical representation of the race-track cycle counting technique.

THEORETICAL BASIS FOR DAMAGE TRANSFORMS

From our present understanding of fatigue, it is not only the magnitude of the force that determines the extant of fatigue damage but also the rate at which it is applied and withdrawn. For cyclic loadings, that is equivalent to saying that damage would be a function of both the amplitude and the frequency of the load pattern.

When a force is applied to a body, the body deforms. As it moves from one configuration to the next, the applied forces (both internal and external) do some amount of work. Theoretically, it is possible to estimate the quantity of work done per unit time, i.e., rate of work done or power. This can be done by considering the kinetic energy of a unit volume of the body. For infinitesimal displacements over small time intervals, a sufficiently approximate formulation for the kinetic energy per unit volume could be derived. For a comprehensive study of this problem, the reader is referred to [Ref. 14].

The theory to derive a dimensionless number from a stress function to represent damage shall be discussed briefly in the next section. It should be pointed out that similar dimensionless numbers can also be derived here. Not all these dimensionless damage numbers may be independent of each other as will be seen in the next section.

DAMAGE AND STRESS FUNCTIONS

In exploring the different means whereby a hierarchical scale for different load cycles can be established, it is accepted that the time variable plays an important role. So, it is not just the magnitude of the applied force, stress or energy that matters in determining the extant of fatigue damage; what also matters is the rate at which it is applied. Therefore the rate of change of force, stress or energy intuitively seems to correspond better with our concepts of damage. If by some means, a number could be derived by means of a transfer function applied to the power expended on the pavement, a relevant damage number could be derived.

A technique of analysis based upon similitude study is a practical approach to address this complex engineering problem. In the classical sense, the similitude technique (also referred as dimension analysis) consists of a study of three stages. In the first stage, the predominant variables of the problem are recognized for grouping into meaningful dimensionless groups. The second stage in the study consists of setting the criteria for similitude by deciding the relative importance of the dimensionless groups of the variables of the problem. The third stage consists of the actual execution of the similarity criteria in making a mechanistic model, deciding the kinematic conditions, testing the process and predicting the behavior of the prototype in view of the dynamic parameters involved. An important theorem to be remembered in this regard is Buckingham's p theorem which provides that a set of n physical quantities with r base dimensions may always be arranged to form an infinite number of dimensionless groups, of which only (n - r) dimensionless parameters are independent.

An attempt is now made to address the damage issue by the use of dimensional analysis. As discussed earlier, the rate of application of the force (or the rate of change of energy which is power) has been intuitively identified as an effective parameter to represent damage. Using the notation L = the length dimension, T = the time dimension and M = the mass dimension, the following dimensional representation can be stated

$$\text{Power} = \frac{ML^2}{T^3} \tag{6-1}$$

A factor not included in our damage discussions so far concerns the size of the specimen being tested for damage. A larger specimen would more likely have a higher capacity to absorb and dissipate the energy of an impact as opposed to a smaller specimen with the same material properties. It would seem logical that the rate of change of applied energy, or power should be normalized by the volume of the specimen to account for the size factor. That leads to considering the power expended per unit volume for similitude analysis with damage.

$$\text{Power per unit volume} = \frac{M}{LT^3} \tag{6-2}$$

In this regard, a finite element program being developed by Roesset and his associates at the University of Texas at Austin was used to simulate the dynamics of vehicles moving on different pavement profiles. Results obtainable from the dynamic simulation model include the stress and displacement histories of a pavement system. These histories can be constructed not only across time at a particular point in space, but also for a particular time at a number of points. In other words, stresses and displacement can be plotted either against time for a constant distance, or they can be plotted against distance at a given value of time. This directly provides us with the following functions:

Stress = f (distance)constant time = f (time)constant distance, and

Displacement = f (distance)constant time = f (time)constant distance.

The advantage of representing these variables in their different functional forms is that the exact differential with respect to both time and distance can now be evaluated.

$$\lim_{\Delta t \to 0} \frac{\sigma(t) - \sigma(t + \Delta t)}{\Delta t} = \frac{\delta\sigma}{\delta t} \tag{6-3}$$

Examining the right hand side of equation, it can seen from equation 6-2 that it has the same dimensional form as equation 6-3, that is power / unit volume and the time rate of change of stress are dimensionally identical. e.g.,

$$\frac{d\sigma}{dt} = \frac{M}{LT^3} \tag{6-4}$$

This provides a theoretical basis for considering the rate of change of stress as one of the parameters used to derive a damage function. This damage function could also be represented as a stress-ratio dimensionless number by comparing the damage caused by one event with the damage caused by another event. [Ref. 8] and [Ref. 21] provides a detailed description of the programming techniques and methodologies as well as the theoretical formulation of the damage models. For the purpose of this report, the relevant equation for a single stress has been presented here. It should be noted that more complex stress diagrams can be treated as made up of a series of single peak stress diagrams and analyzed accordingly.

$$D = \int_0^T \frac{1}{T} \left| \frac{\delta\sigma}{\delta t} \right| dt \tag{6-5}$$

$$D_r = \frac{\sum_p (D_i)^n}{\sum_q (D_k)^n} \tag{6-6}$$

where D represents the damage caused by the event, T represents the time from start to finish of the stress peak, n is a power coefficient used for calibration. D_r denotes the relative damage caused by a stress event containing p stress peaks with respect to a stress event containing q stress peaks. For computing ESAL values of different single axle loads with respect to a 18 kip single axle, both p and q will have a value of 1 (only one stress peak for a single axle). However, for the purpose of computing ESAL values for different axle configurations in a tandem axle profile, with respect to a tandem axle of a standard axle space, the values of p and q will be equal to 2. If the ESAL value of the tandem axle are to be computed with respect to a single axle, the value of p will be two and the value of q will be 1.

CONCLUSIONS

The the damage transforms seen in equations 6-5 and 6-6 are conceptually similar in to ideas proposed by Palmgreen [Ref. 18] in his tretise on the phenomenon of fatigue failure in ball and roller bearings. The main difference between the two procedures is that Palmgreen used force as his choice of variable from which to derive a dimensionless ratio. His number corresponds to a dimensionless force ratio and depicts the ratio of the life L_1 as predicated by force F_1, with respect to the life L_2 as predicated by force F_2. Palmgren found that for his data, the best regression fit was obtained for $n = 3$.

The damage number D_r could be thought of in similar terms. The difference would be in that instead of Force ratios, Power per unit volume ratios are used. The value of n would be quite different in equations pertaining to pavements and ballbearings being dependent as it is on the interactions of a host of variables that are completely different for ball bearings and pavements.

This derived damage number may be more accurate in its representation of actual damage than the ESAL numbers due to reasons presented in [Ref. 8]. This methodology may be applied to determine the relative damage caused by an event with respect to the damage caused by another event. A limitation of the model may be the mathematical procedure which may limit the sensitivity of the final result only when sub-events are of the same order of magnitude. In fact, it is not certain whether this is a limitation or whether that is more or less the actual way it happens.

In the next chapter, this methodology shall be applied to determine the relative damage to pavement systems caused by different axle configurations and axle weights.

CHAPTER 7. FACTORS FOR PAVEMENT DAMAGE

INTRODUCTION

One of the primary concerns to be addressed here is the effect of axle configuration on damage. While the damage due to single axles placed far apart (of the order of 10 ft or more) from other single axles can be computed, the effect of tandem and tridem axles on pavements is not easily calculable with any precision. The spacing of axles within a tandem group is a crucial variable that would effect the amount of damage. This fact can be reasoned in the following manner.

Imagine a single axle approaching (from infinity) a point at which normal stresses are being recorded. When the distance separating the two are large, the stresses are zero. As a single axle nears the point, the normal stress increases steadily from its zero value. This stress reaches a maximum value when the axle is directly over the point of study. As the axle moves away from the point, the normal stress curve drops off steadily till it eventually (at a sufficiently large distance), it reduces to zero.

Next, consider the case of a tandem axle with both axles carrying an equal amount of load. Once again, as the tandem axle approaches from infinity, the stress starts off at zero and increases as the axles get closer to the point. When the first axle of the tandem group is directly over the point of interest, the stress reaches a maximum value. So far, the shape of the stress curve is similar to that seen produced by the single axle. The behavior of the stress curve during the next phase is quite different from that of the single axle. As the first axle of the tandem group moves away from the point, the stress starts to decrease. However, before the load of the first axle has been completely "unloaded" from the point and the stress curve dropped back to zero, the load of the second axle starts to effect the stress at the point and the curve starts to rise once again. By the time the second axle is directly over the point, the stress curve is back to its previous peak level experienced due to the passage of the first axle. When the second axle starts to moves away from the point, the stress curve for the point finally starts to decay back to zero.

The single axle stress curve can be likened to an inverted "V" with nonlinear sloping sides rising from zero while the axle aproaches the point, a peak occuring at the instant the axle is directly above the point and decaying to zero as the axle moves away. The stress curve for the tandem axle can be likened to the letter "M". Again, the sides are nonlinear and the two peaks occur at the instants at which each of the two axles are directly above the point. The distance between the two peaks is directly proportional to the space between the two axles of the set. The wider apart the axles, the wider apart are the peaks. Similarly, on a time scale, the slower the vehicle moves, the wider apart in time are the two peaks. The depth of the valley between the two peaks is a function of the depth of the point of interest below the pavement surface. If the depth of the point is greater than the axle spacing by orders of magnitude, there will be no discernable valley and the stress curve will have a general shape similar to that produced by a single axle. At depths of a similar magnitude as the axle spacing, there will be a a gentle valley between the two peaks. The lowest point in this valley would, however, be above the zero level. At points on (or very close to) the surface of the pavement the shape of the complete curve formed by the tandem axle could be duplicated by placing two similar single axle stress curves separated by the same distance as the distance seperating the two axles.

An example of both the curves for single axles and tandem axles can be found in the profiles generated by the truck type 3S2 - a tractor-semitrailer with 1 front single axle, 2 drive axles (tandem) and 2 semitrailer axles (tandem). Figures 7-1 to 7-20 show the stress and displacement profiles for this truck at various depths below the surface of the pavement. The space between the 5 wheels of the truck are - 17 feet, 4 feet, 40 feet and 4 feet respectively starting from the stearing axle. The axle loads are 12 kips, 34 kips (tandem) and 34 kips (tandem) respectively. The truck is assumed moving at 35 mph and the pavement profile is modelled after a flexible pavement to a depth of 128 feet of half-space below the surface of the pavement. It should be stated that the stress and displacement profiles compare with the general nature of the pore pressure curve measured under the pavement during the passage of a truck [Ref. 8].

VEHICLE SPEED

As can be judged from the preceeding discussion, the rate of loading and unloading plays an important role in determining the amount of damage caused. In fact, with particular reference to the formulation of the equations discussed in the previous chapter (equations 6-5 and 6-6), it can be seen that a high rate of change of stress will give us a larger damage number when compared to a low rate of change of stress. This is in conformity with our intuitive knowledge of fatigue damage discussed both in this study as well as in previous studies [Ref. 7].

This leads directly to the conclusion that all other things being equal, a vehicle moving at, say, 35 mph will cause less damage to the pavement than a similar vehicle moving at, say 65 mph. In essence, higher velocities would contribute to higher damage.

It may be possible to quantify the damage done to pavements by a 65 mph speed with respect to the damage caused by a 35 mph speed by a method similar to the one outlined in equations 6-5 and 6-6 . However, before such quantitative conclusions are made, further studies need to be conducted that would jointly examine the effect of other equally pertinent variables such as the elastic/plastic limits

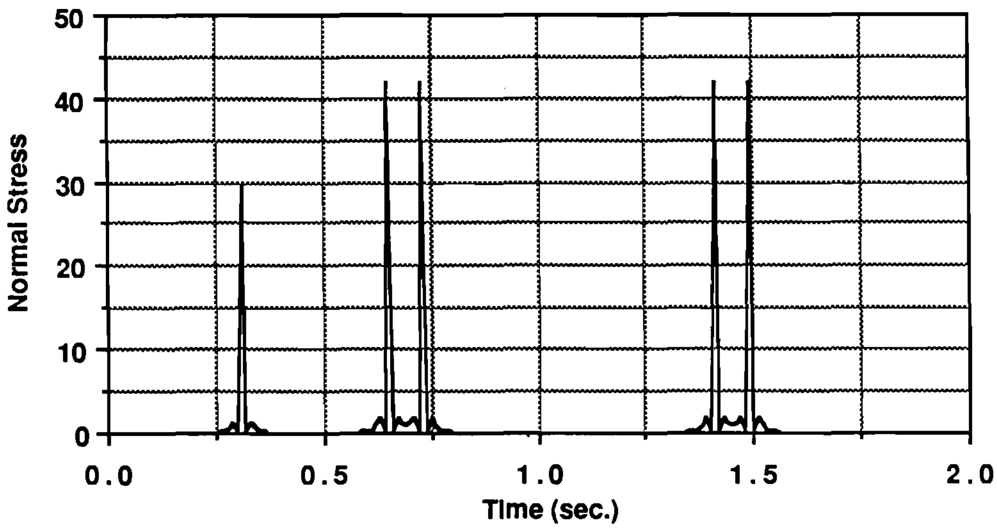


Figure 7-1. Stress profile for 3S2 truck six inches below pavement surface.

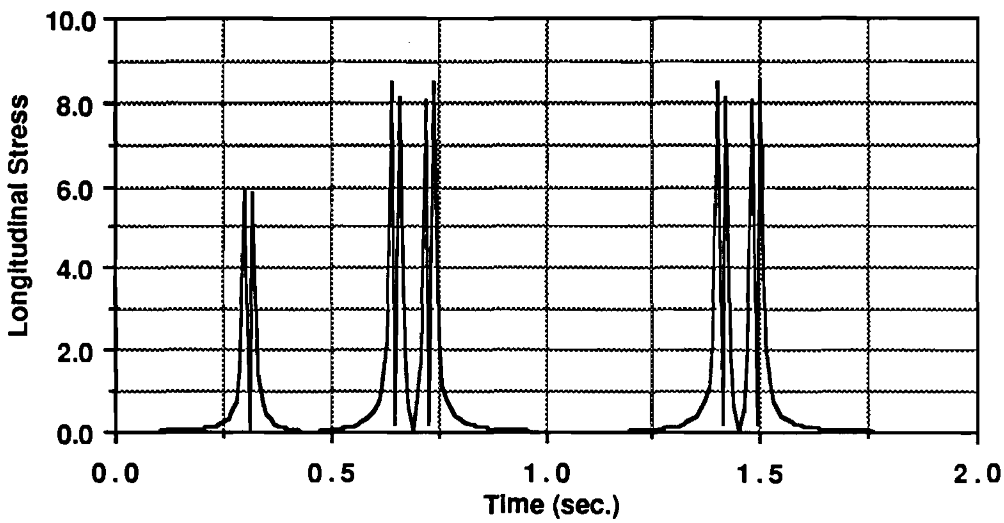


Figure 7-2. Stress profile for a 3S2 truck six inches below the pavement surface.

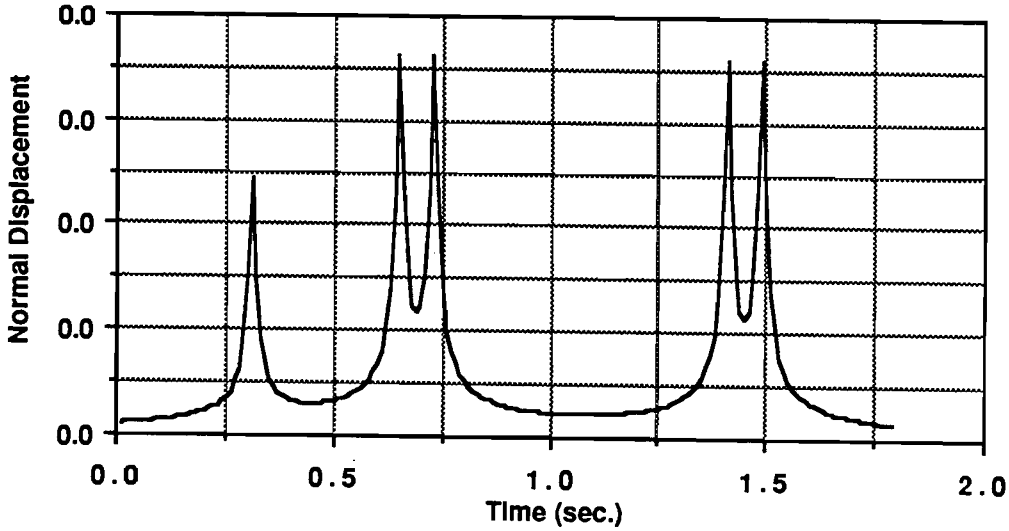


Figure 7-3. Displacement profile for a 3S2 truck six inches below pavement surface.

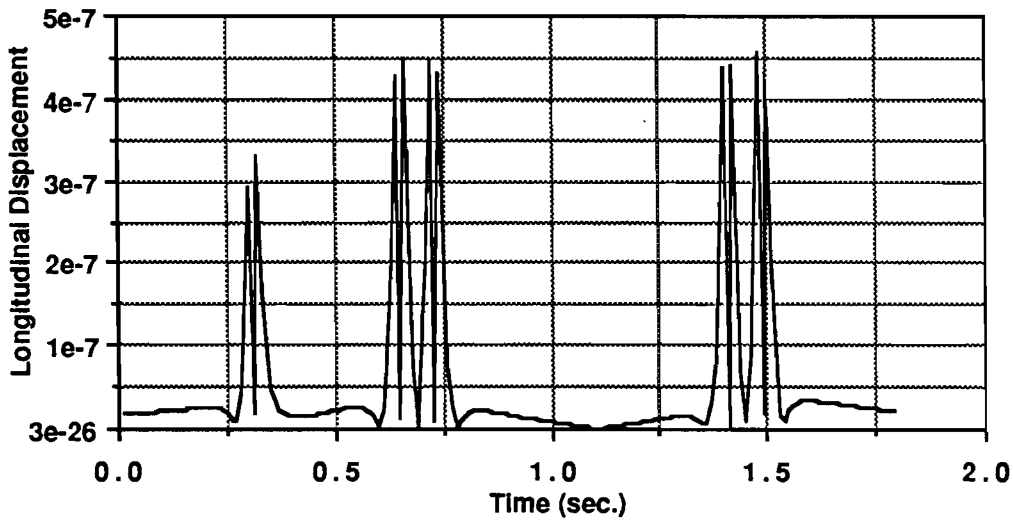


Figure 7-4. Displacement profile for a 3S2 truck six inches below pavement surface.

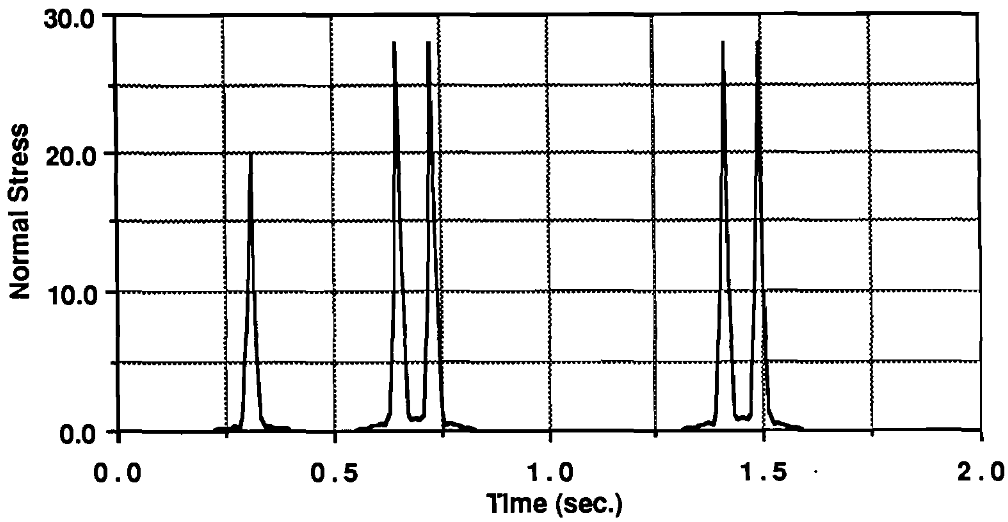


Figure 7-5. Stress profile for a 3S2 truck one foot below pavement surface.

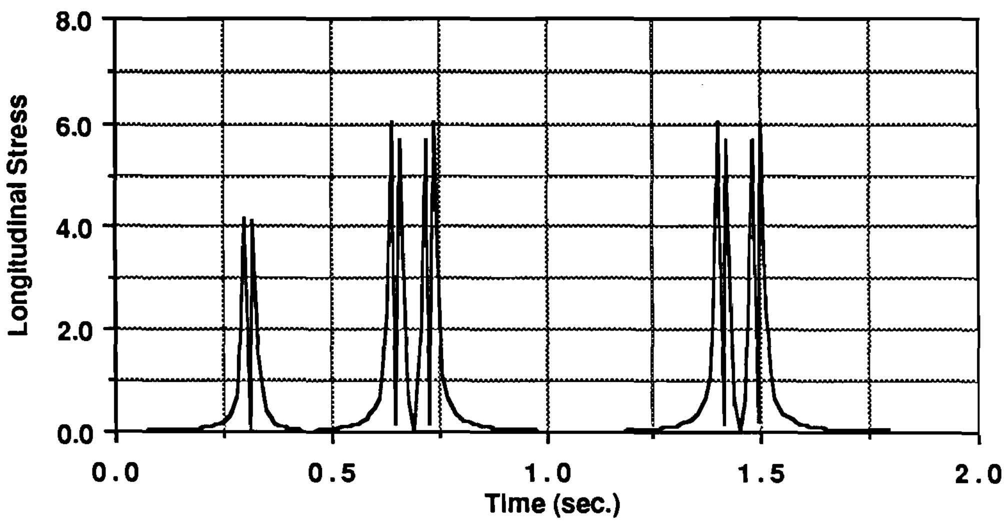


Figure 7-6. Stress profile for a 3S2 truck one foot below pavement surface.

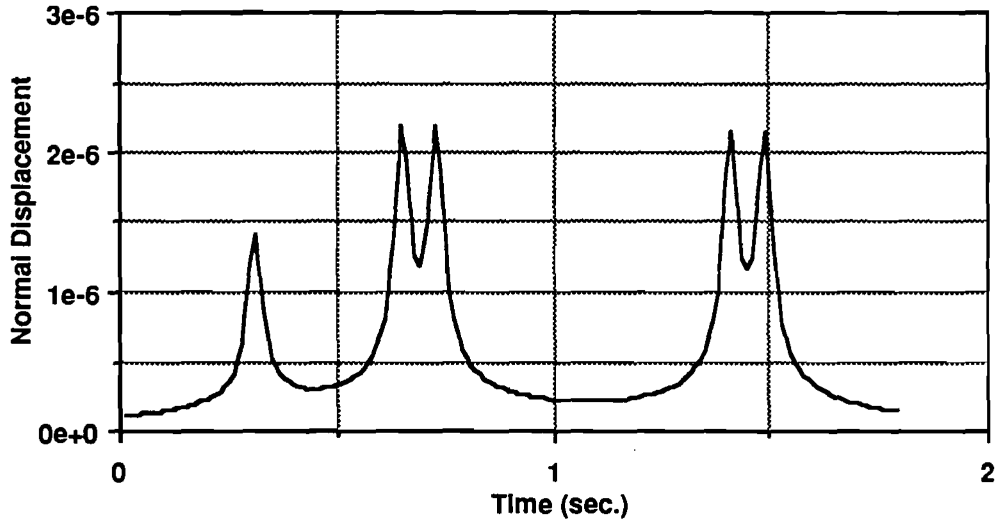


Figure 7-7. Displacement profile for a 3S2 truck one foot below pavement surface.

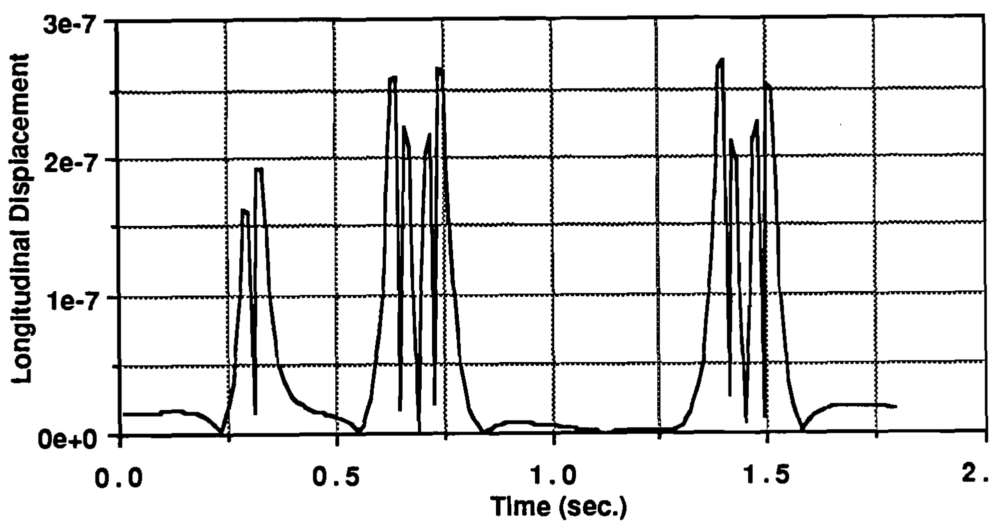


Figure 7-8. Displacement profile for a 3S2 truck one foot below pavement surface.

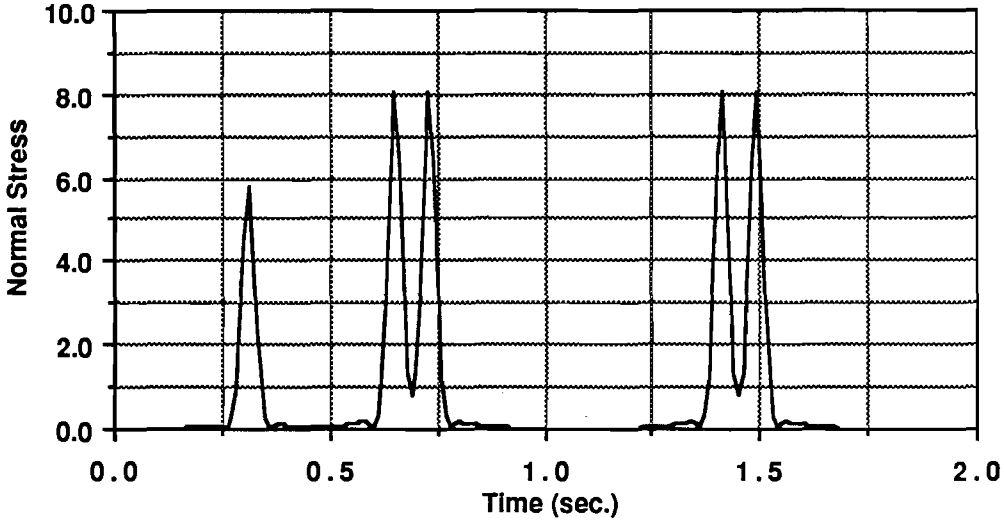


Figure 7-9. Stress profile for a 3S2 truck two feet below pavement surface.

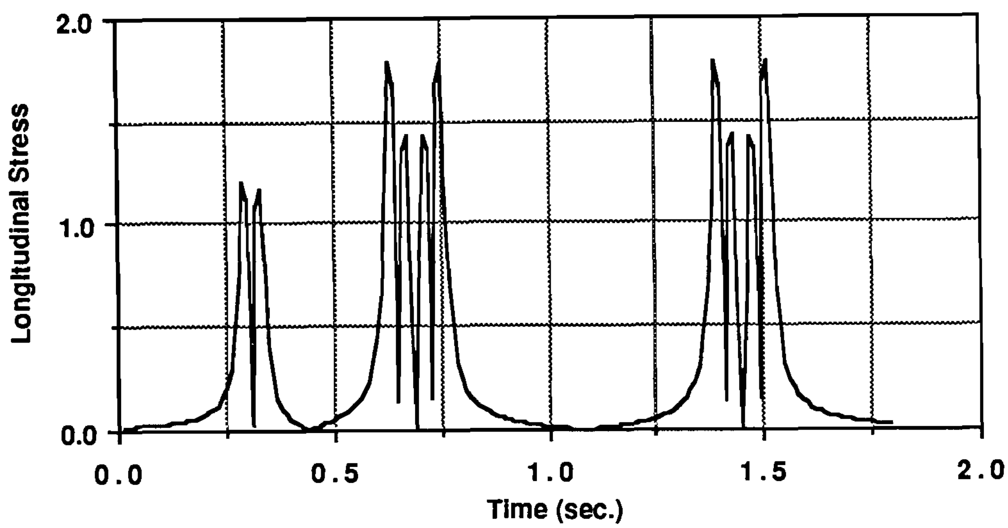


Figure 7-10. Stress profile for a 3S2 truck two feet below the pavement surface.

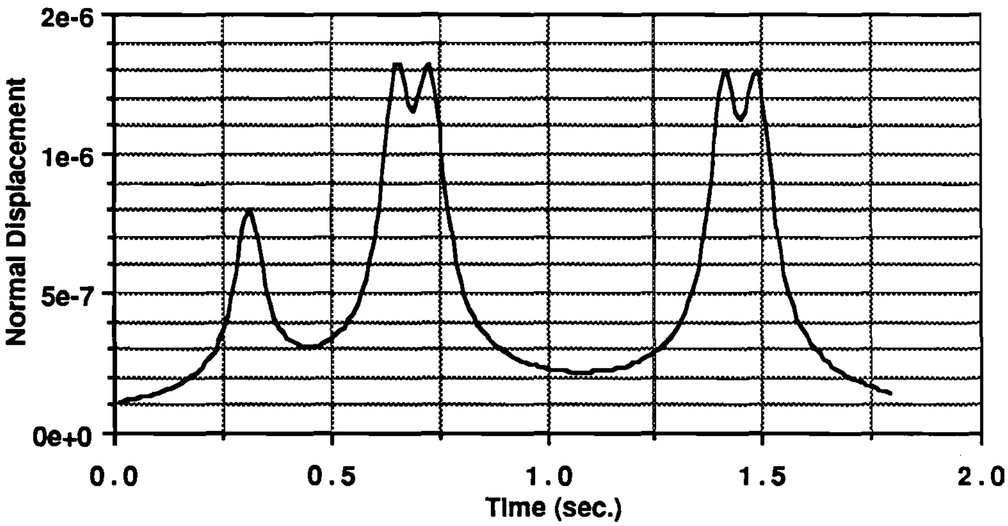


Figure 7-11. Displacement profile for a 3S2 truck two feet below pavement surface.

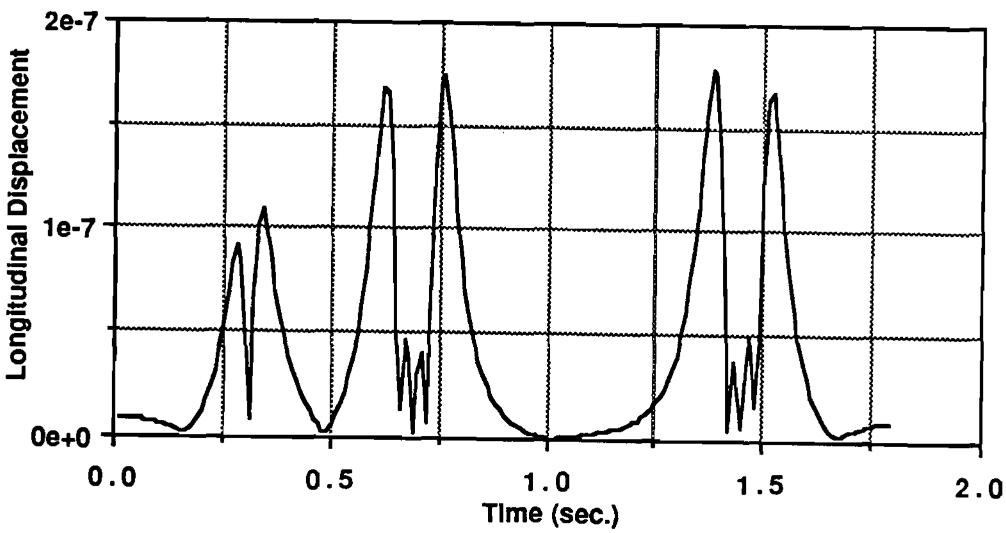


Figure 7-12. Displacement profile for a 3S2 truck two feet below pavement surface.

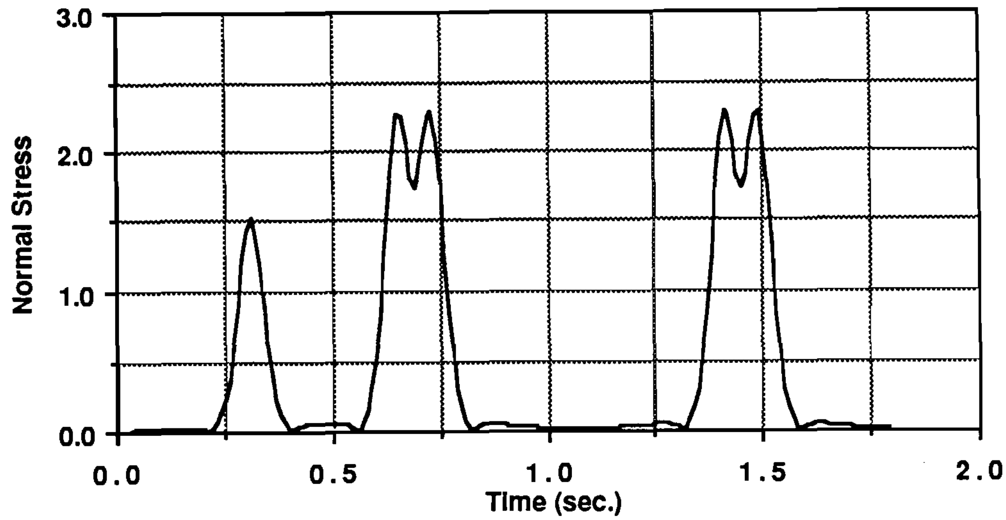


Figure 7-13. Stress profile for a 3S2 truck four feet below the pavement surface.

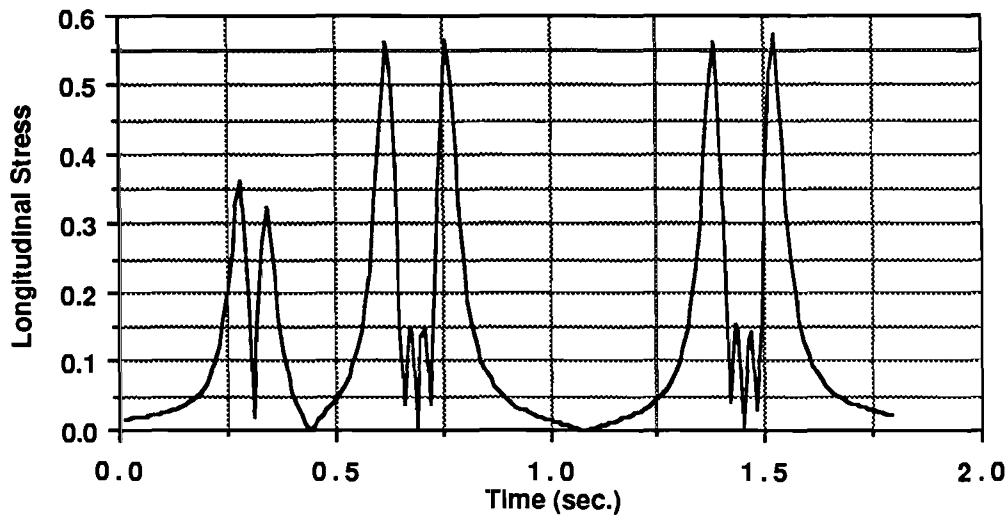


Figure 7-14. Stress profile for a 3S2 truck four feet below the pavement surface.

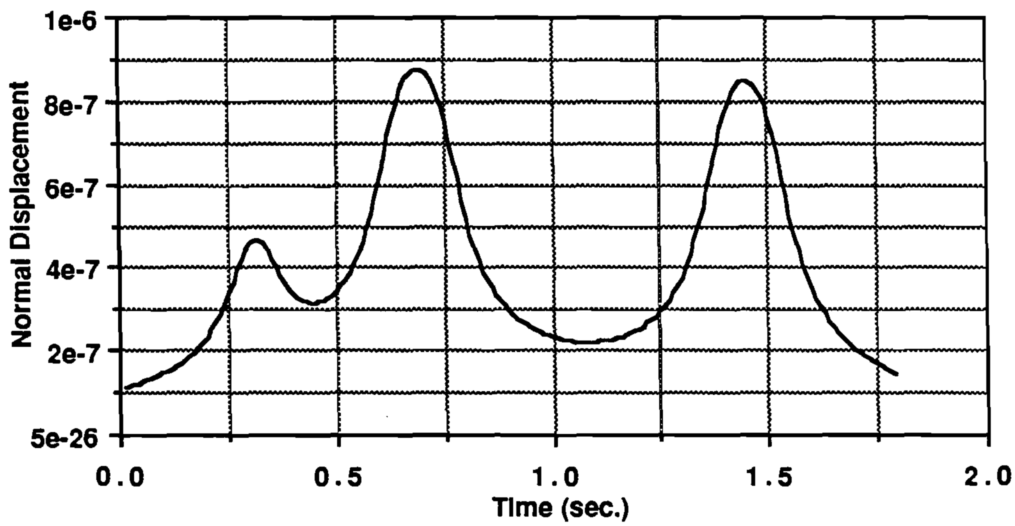


Figure 7-15. Displacement profile for a 3S2 truck four feet below pavement surface.

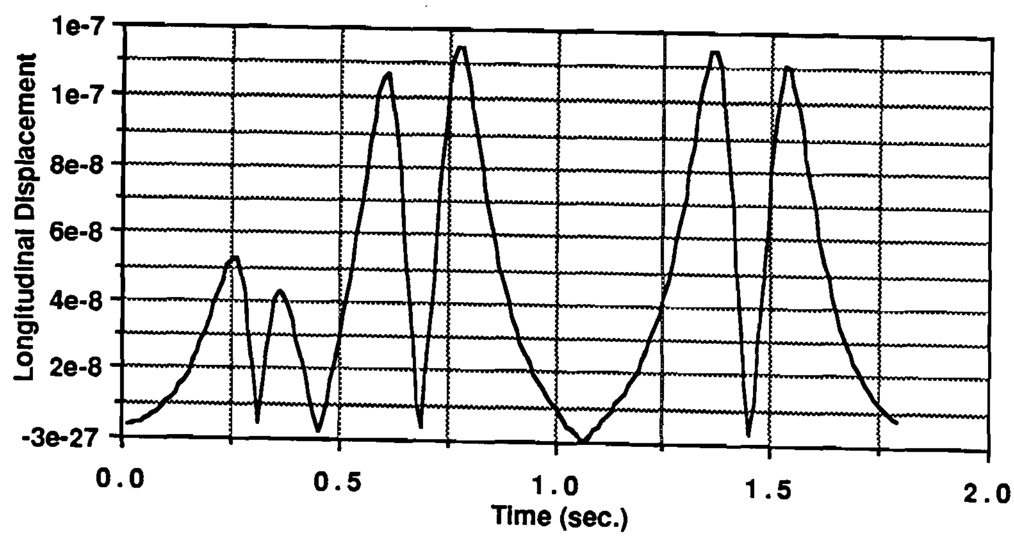


Figure 7-16. Displacement profile for a 3S2 truck four feet below pavement surface.

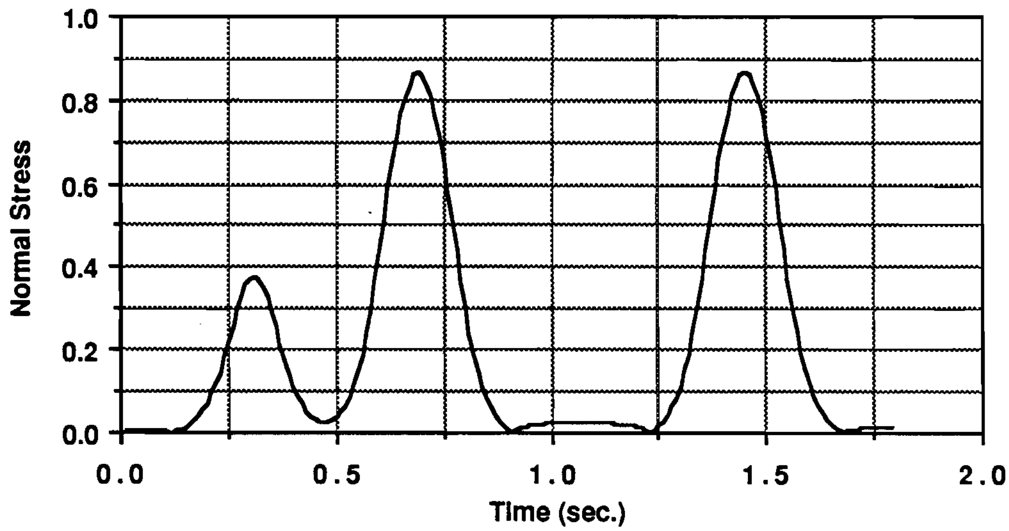


Figure 7-17. Stress profile for a 3S2 truck eight feet below pavement surface.

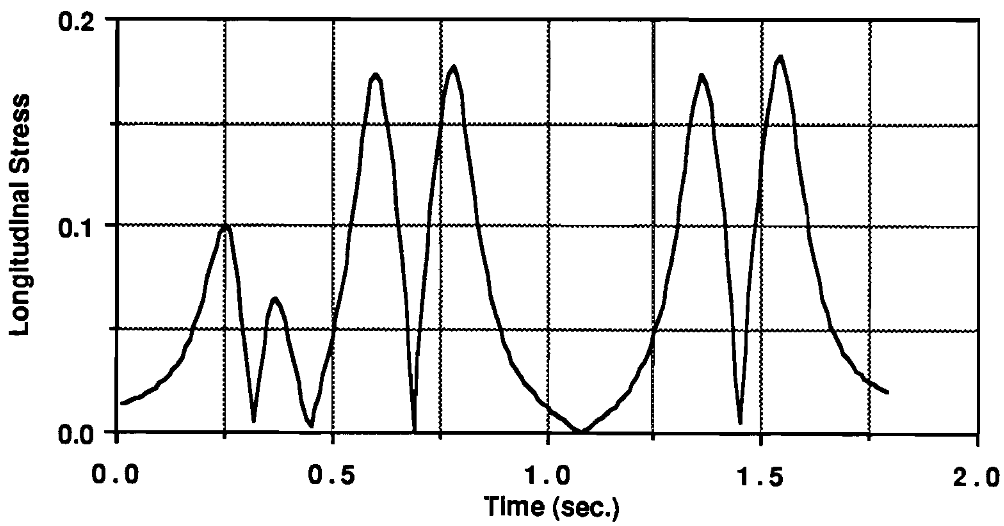


Figure 7-18. Stress profile for a 3S2 truck eight feet below pavement surface.

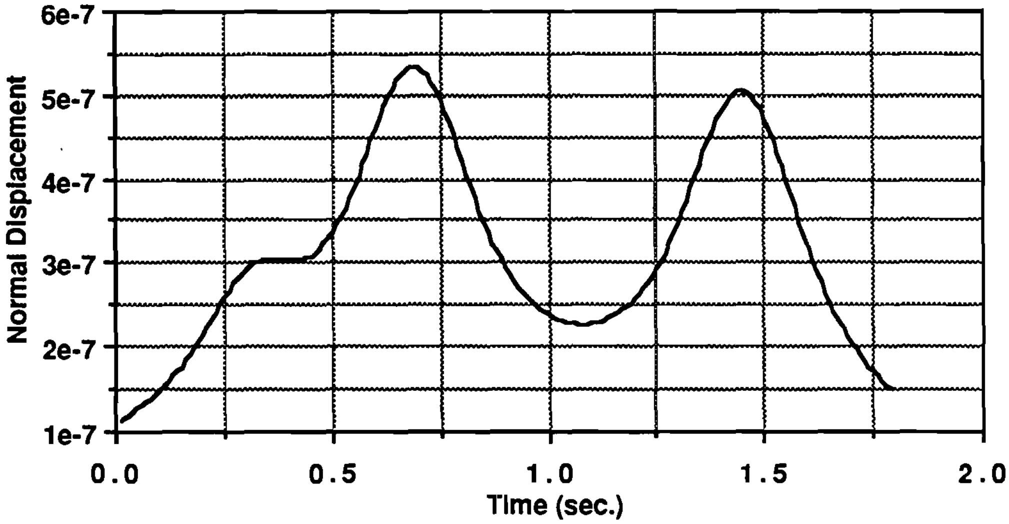


Figure 7-19. Displacement profile for a 3S2 truck eight feet below pavement surface.

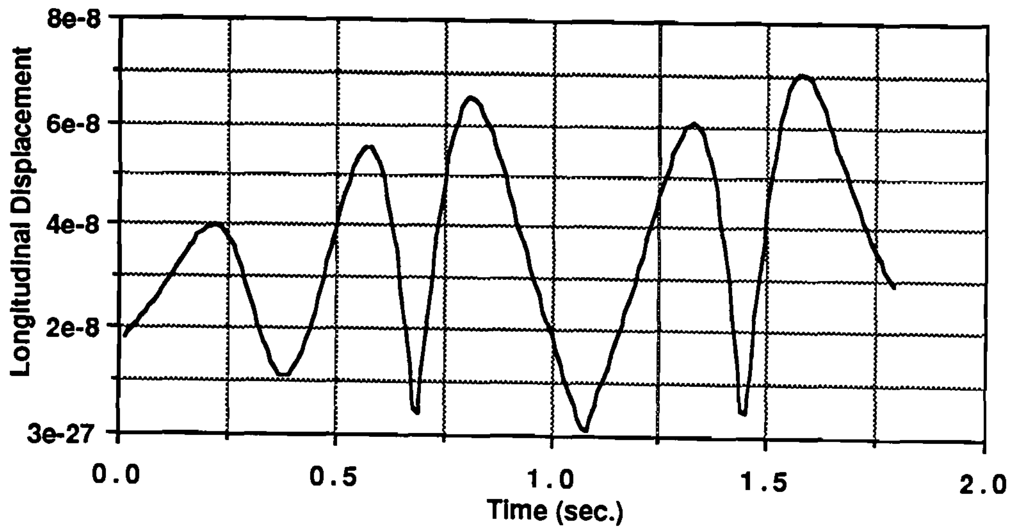


Figure 7-20. Displacement profile for a 3S2 truck eight feet below pavement surface.

of the pavement and the interrelationship between these limits and the magnitude of applied loads to fatigue damage and failure.

It has also been argued that the pavement system is stiffer for higher speeds. The implication is that faster trucks would cause less deflection and therefore less damage compared to slower trucks primarily due to the viscoelastic behavior of the pavement layers where a finite amount of time is required under loaded conditions for the pavement to attain its maximum deflection. This phenomenon will create higher deflections for a stationary load when compared to the same load if it is moving. However, for fatigue failure, it is not the magnitude of the load alone that is the governing factor (if the load is within the elastic limits of the material). If the applied load is greater than the ultimate strength of the specimen, the magnitude of the load would obviously determine failure. However, when the applied load is within the elastic limits of the specimen, the rate of change of the load governs for fatigue failure. It has been verified by observation that a large number of rapid applications and removals of a load significantly lower than the ultimate strength of the specimen, can cause the specimen to fail.

The preceding discussion would imply that a higher speed would cause less damage than a lower speed would because it results in smaller deflections. On the other hand, the former would also cause more fatigue damage than the latter due to the faster rate of the loading-unloading cycle. The final outcome (of whether high speeds cause more damage or less damage than low speeds) would depend not only on the differences in magnitude of the stresses in the two cases (due to viscoelastic effects), but also on other factors not included in the discussion here, e.g. impact loads. If the difference in the induced stress between the high speed and the low speed cases was of an order of magnitude comparable to the stress produced by the high speed case (i.e. the specimen was highly viscoelastic with an extremely low rate of change of strain and the stress produced by the high speed was significantly lower than the stress produced by the low speed), it is conceivable that the lower speed would cause more damage. However, purely from the point of view of fatigue failure, it would seem that high speeds would cause more damage than low speeds.

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PAVEMENT PROFILES

In the simulations run for different pavement profiles it has been observed that a bed of rock present beneath the pavement surface produces some dynamic amplification of stresses. On the other hand, when profiles containing a half space medium are modelled, no significant dynamic amplification is present. In terms of the program, the differences modeled in the two situations are simply their ability to reflect and refract Love waves. This is another arena where a considerable amount of further research is required before more definitive conclusions can be drawn. For the present, it can be stated that definite differences do exist between the behaviors of the half space and rock profiles.

AXLE CONFIGURATIONS

As discussed in the introduction section of this chapter, axle configurations play a vital role in determining the amount of damage caused to pavements. This comes about primarily due to the fact that two axles that are placed close to each other tend to produce stress curves that overlap. The unloading cycle of the first axle overlaps with the loading cycle of the second axle. If the two axles were separated and then run over a pavement, they would have the combined effect of making the pavement go through two complete loading and unloading cycles respectively. However, if placed close to each other, one unloading and loading cycle gets distorted due to the proximity of the axles and the nett effect produced is not two complete loading and unloading cycles, but only one complete loading cycle for the first axle and one complete unloading cycle for the second axle together with an incomplete unloading cycle for the first axle and an incomplete loading cycle for the second axle. This would lead us to the conclusion that axles that are spaced closer to each other would cause less damage compared to similar axles placed far apart. This fact is recognized implicitly in the AASHTO ESAL values. For example, a 36 kip tandem axle, each axle carrying 18 kip loads, is not equal to two 18 kip ESAL. AASHTO lists the ESAL value for this case as 1.36 . What AASHTO does not account for is the space between these two axles in the tandem group. Whether the space between the tandems is 4 feet or 5 feet or 6 feet, each of the three cases of tandem axles is assumed to have the same ESAL value [Ref 1].

From discussions in a previous section, it would seem as though it would be the most profitable from the point of reduction of fatigue damage to keep axles as close to each other as possible. Such is not the case as a number of other problems can confound the issue. First of all, there is the physical constraint of wheel radius that imposes a strict minimum on the distance that two axle may be placed relative to each other. Further, if two axles are placed too close to each other, the loading cycle of the second axle may start while the loading cycle of the first axle is still in progress instead of when the unloading cycle of the first axle is in progress. While this would reduce the loading unloading cycle to one, it would effectively increase the magnitude of the maximum stress attained and in all probability lead to disasterous consequences. Then there are intangible problems of compliance to Federal regulations as proposed in the Bridge Formula [Ref. 17]. This imposes restrictions on how close axles may be placed in any group of two or more axles. The purpose is to prevent overloading of short span bridges due to the simultaneous positioning of a large number of axles each carrying high loads. All this would lead us to believe that there would be a happy medium between placing axles too far apart to cause two complete loading and unloading cycles and having axles so close together that the magnitude of the maximum stress attained is increased. It is this factor that we will investigate in the next section.

DISCUSSION OF RESULTS

Two sets of results are presented and discussed here. The first set deals with the calibration of the model proposed in the previous chapter. The second set deals with applying the model to determining the ESAL for different tandem axle spacings.

Initially, a regression model was run to determine the bound of the value of n in the equation.

$$D = \int_{0}^{T} \frac{1}{T} \left| \frac{\delta \sigma}{\delta t} \right| dt \qquad (7\text{-}1)$$

$$D_r = \frac{\sum_{p} (D_i)^n}{\sum_{q} (D_k)^n} \qquad (7\text{-}2)$$

The data used to callibrate the model was obtained from the AASHTO Road Test Data set and is comprised of the data for flexible pavements (Tables 7-1 through 7-5). The data used corresponds to the life of the pavement sections while they were between a psi (present serviceability index) of 3.5 and 3.0 . The first two columns represent the section numbers for the 1st. lane and the 2nd lane respectively (L_1 and L_2). The next three columns in the tables are the life ratios of the number of loading and unloading cycles it took for the pavement to go from a p.s.i. value (present serviceability vale) of 3.5 to 3.0, from its initial p.s.i. value to 3.5 and from its initial p.s.i. value to 3.0 respectively for each lane. The traffic on each loop was simulated to obtain the damage transforms and the value of n was found to equate the life ratios to a power of the ratios of the damage transform. Table 7-6 lists the values of D from equation 7-1 for axle weight ranging from 2 kips to 50 kips.

A regression was performed on this data, more with the intent of finding the range of the value that the variable n could take and not just with the idea of fixing the best fit on the data with a particular value of n. Once a range had been determined for the variable, it would be easy to check how the equation behaved for different values of n within the bounds. In effect, a sensitivity analysis of the equation could now be performed and the results compared to the AASHTO ESAL values. Figure 7-21 shows the plot of the residual sum of squares plotted with respect to the values of n. From Figure 7-21, it can be seen that the value of the sum of squares is below 1 for n values between 3.0 and 7.0, with a smaller rate of change of slope observed toward the value of 3.0 compared with the rate of change toward 7.0, implying that the likelihood would be that the actual value of n would be more biased towards 3.0 than towards 7.0 . Clearly, this region should provide us with a value of n that would be a good fit for the AASHTO data. Table 7-6 provides the values computed for the value of the D not yet raised to the power n. Table 7-7 through table 7-12 shows how the ESAL values for different axle loads compute

TABLE 7-1. AASHTO ROAD TEST DATA FOR FLEXIBLE PAVEMENT SECTIONS IN LOOP 2

Lane 1 Section	Lane 2 Section	L2/L1 3.0-3.5	L2/L1 3.5	L2/L1 3.0
721	722	0.1	0.2	0.1
727	728	0.0	0.1	0.1
743	744	0.0	0.1	0.0
717	718	0.1	1.0	0.3
755	756	0.1	0.1	0.1
719	720	0.2	0.2	0.2
771	772	0.0	0.4	0.1
729	730	0.4	3.6	0.5
731	732	0.3	0.9	0.6
769	770	0.4	0.1	0.2
Mean		0.17	0.68	0.22
Std. Dev.		0.15	1.09	0.20

TABLE 7-2. AASHTO ROAD TEST DATA FOR FLEXIBLE PAVEMENT SECTIONS IN LOOP 3

Lane 1 Section	Lane 2 Section	L2/L1 3.0-3.5	L2/L1 3.5	L2/L1 3.0
127	128	0.1	2.1	0.9
157	158	0.2	0.9	0.3
121	122	0.3	1.5	0.7
131	132	0.3	0.1	0.2
119	120	0.4	0.8	0.5
133	134	0.4	1.3	0.9
147	148	0.5	2.5	1.5
149	150	0.5	1.3	0.8
137	138	0.5	0.9	0.6
129	130	0.5	0.5	0.5
123	124	0.5	1.1	0.9
151	152	0.6	2.6	1.1
153	154	0.8	0.9	0.9
155	156	0.8	0.2	0.4
161	162	0.9	1.0	0.9
135	136	0.9	0.4	0.5
113	114	1.0	0.7	0.7
141	142	1.6	0.6	1.1
109	110	1.9	0.2	0.7
117	118	1.9	0.3	0.9
163	164	2.0	0.6	0.9
143	144	2.0	0.9	1.0
107	108	2.2	0.2	0.7
111	112	3.9	0.1	0.7
145	146	178.5	0.7	1.1
Mean		8.13	0.89	0.77
Std. Dev.		35.52	0.68	0.29

TABLE 7-3. AASHTO ROAD TEST DATA FOR FLEXIBLE PAVEMENT SECTIONS IN LOOP 4

Lane 1 Section	Lane 2 Section	L2/L1 3.0-3.5	L2/L1 3.5	L2/L1 3.0
575	576	0.4	1.0	0.8
607	608	0.4	1.3	1.1
615	616	0.5	2.4	1.0
579	580	0.6	1.2	1.1
597	598	0.8	1.2	1.1
601	602	0.9	1.3	1.0
599	600	1.0	1.0	1.0
617	618	1.0	1.4	1.1
605	606	1.0	1.4	1.3
569	570	1.1	1.2	1.2
585	586	1.1	1.0	1.1
631	632	1.6	1.5	1.5
589	590	1.6	1.1	1.1
603	604	1.6	2.9	2.4
619	620	1.7	0.8	1.0
593	594	1.8	1.6	1.7
595	596	1.8	1.0	1.2
577	578	1.9	0.5	0.9
587	588	2.2	0.9	1.1
627	628	2.4	0.4	1.1
625	626	2.8	1.1	1.8
583	584	3.2	1.3	1.3
571	572	3.9	1.1	1.4
623	624	5.0	0.4	1.6
621	622	6.8	1.1	2.6
629	630	8.3	0.4	1.3
573	574	10.9	0.8	1.1
Mean		2.46	1.16	1.29
Std. Dev.		2.54	0.53	0.42

TABLE 7-4. AASHTO ROAD TEST DATA FOR FLEXIBLE PAVEMENT SECTIONS IN LOOP 5

Lane 1 Section	Lane 2 Section	L2/L1 3.0-3.5	L2/L1 3.5	L2/L1 3.0
485	486	0.1	0.3	0.2
473	474	0.2	2.2	1.2
441	442	0.3	0.5	0.3
425	426	0.4	0.5	0.4
451	452	0.5	3.5	1.3
411	412	0.5	1.3	1.1
479	480	0.7	0.8	0.7
475	476	0.7	0.4	0.5
445	446	0.8	2.2	1.1
469	470	0.8	1.0	0.9
477	478	0.9	1.0	0.9
417	418	1.0	0.9	1.0
443	444	1.0	1.8	1.4
447	448	1.1	0.7	0.9
455	456	1.1	0.6	0.8
471	472	1.2	1.1	1.2
415	416	1.4	0.5	0.7
423	424	1.4	0.9	1.0
413	414	1.4	0.8	1.0
439	440	1.5	1.8	1.7
437	438	1.5	1.4	1.4
429	430	1.6	1.9	1.7
421	422	1.7	0.8	1.0
449	450	2.0	1.1	1.2
453	454	2.3	0.6	1.3
481	482	2.8	0.7	1.0
483	484	3.0	0.3	1.0
419	420	6.0	0.7	1.2
487	488	17.7	4.8	12.3
Mean		1.92	1.22	1.40
Std. Dev.		3.25	0.98	2.13

TABLE 7-5. AASHTO ROAD TEST DATA FOR FLEXIBLE PAVEMENT SECTIONS IN LOOP 6

Lane 1 Section	Lane 2 Section	L2/L1 3.0-3.5	L2/L1 3.5	L2/L1 3.0
307	308	0.2	0.9	0.4
309	310	0.4	1.6	1.1
253	254	0.5	0.7	0.6
329	330	0.6	2.0	1.6
311	312	0.8	3.2	2.0
327	328	0.8	0.4	0.5
271	272	0.8	0.8	0.8
297	298	0.9	1.5	1.3
331	332	0.9	1.2	1.0
303	304	1.1	2.7	2.0
269	270	1.3	3.2	1.7
261	262	1.4	1.4	1.4
321	322	1.4	3.0	2.1
267	268	1.5	1.1	1.2
315	316	1.7	0.9	1.3
323	324	1.8	1.1	1.2
319	320	2.0	2.2	2.2
259	260	2.0	1.0	1.3
313	314	2.3	1.6	2.0
335	336	2.3	0.5	1.1
255	256	2.7	1.0	2.0
325	326	5.0	0.9	1.1
299	300	5.5	1.4	2.7
305	306	6.1	1.4	2.8
317	318	9.2	2.4	4.2
263	264	9.4	1.3	3.3
257	258	18.4	1.1	5.2
Mean		3.00	1.50	1.79
Std. Dev.		3.97	0.79	1.10

TABLE 7-6 COMPUTED VALUES OF
D FOR SINGLE AXLE LOADS

Load (kips)	D
2	1.51
4	3.04
6	4.54
8	6.08
10	7.58
12	9.08
14	10.62
16	12.12
18	13.62
20	15.14
22	16.64
24	18.18
26	19.68
28	21.22
30	22.62
32	24.20
34	25.78
36	27.38
38	28.76
40	30.36
42	31.84
44	33.20
46	34.82
48	36.36
50	37.76

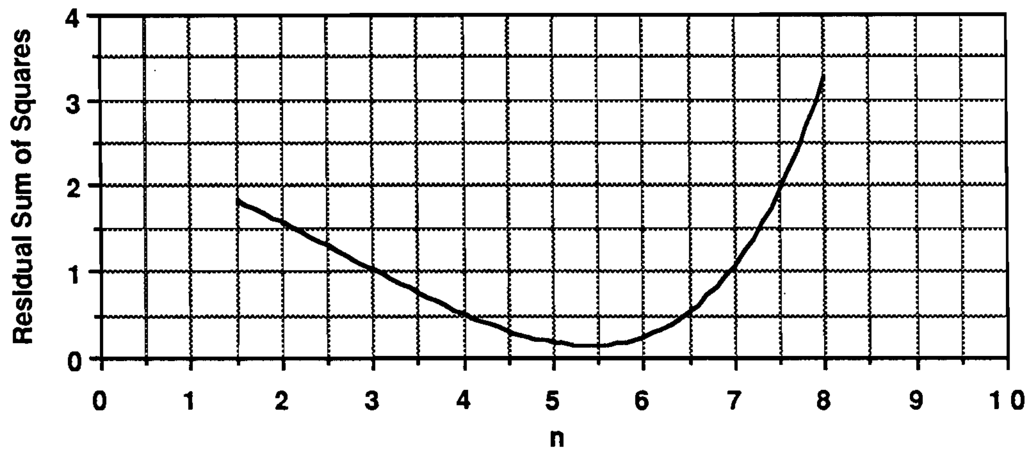

Figure 7-21. Residual sum of squares plotted against n for equations 7-1 and 7-2 using the AASHTO road test data for flexible pavements.

TABLE 7-7. SENSITIVITY ANALYSIS OF ESAL-LOAD TABLE WITH RESPECT TO n IN EQUATION 6-6

	n Values				
Load (kips)	4.70	4.65	4.60	4.55	4.50
2	0.00	0.00	0.00	0.00	0.00
4	0.00	0.00	0.00	0.00	0.00
6	0.01	0.01	0.01	0.01	0.01
8	0.02	0.02	0.02	0.03	0.03
10	0.06	0.07	0.07	0.07	0.07
12	0.15	0.15	0.15	0.16	0.16
14	0.31	0.31	0.32	0.32	0.33
16	0.58	0.58	0.58	0.59	0.59
18	1.00	1.00	1.00	1.00	1.00
20	1.64	1.64	1.63	1.62	1.61
22	2.56	2.54	2.51	2.49	2.46
24	3.89	3.83	3.78	3.72	3.67
26	5.64	5.54	5.44	5.34	5.24
28	8.04	7.86	7.69	7.52	7.35
30	10.85	10.58	10.31	10.06	9.80
32	14.90	14.48	14.07	13.67	13.29
34	20.06	19.43	18.82	18.23	17.66
36	26.63	25.71	24.83	23.98	23.16
38	33.55	32.32	31.13	29.99	28.89
40	43.27	41.57	39.94	38.37	36.86
42	54.12	51.87	49.71	47.65	45.66
44	65.87	63.00	60.26	57.63	55.12
46	82.41	78.63	75.02	71.58	68.30
48	100.99	96.16	91.55	87.16	82.99
50	120.62	114.62	108.93	103.51	98.37

TABLE 7-8. SENSITIVITY ANALYSIS OF ESAL-LOAD TABLE
WITH RESPECT TO n IN EQUATION 6-6

	n Values				
Load (kips)	4.45	4.40	4.35	4.30	4.25
2	0.00	0.00	0.00	0.00	0.00
4	0.00	0.00	0.00	0.00	0.00
6	0.01	0.01	0.01	0.01	0.01
8	0.03	0.03	0.03	0.03	0.03
10	0.07	0.08	0.08	0.08	0.08
12	0.16	0.17	0.17	0.17	0.18
14	0.33	0.33	0.34	0.34	0.35
16	0.59	0.60	0.60	0.61	0.61
18	1.00	1.00	1.00	1.00	1.00
20	1.60	1.59	1.58	1.58	1.57
22	2.44	2.41	2.39	2.37	2.34
24	3.61	3.56	3.51	3.46	3.41
26	5.14	5.05	4.96	4.87	4.78
28	7.19	7.04	6.88	6.73	6.58
30	9.56	9.32	9.09	8.86	8.64
32	12.91	12.54	12.19	11.84	11.51
34	17.10	16.57	16.05	15.54	15.06
36	22.36	21.59	20.85	20.14	19.45
38	27.83	26.81	25.83	24.88	23.97
40	35.41	34.02	32.68	31.40	30.17
42	43.77	41.95	40.20	38.53	36.93
44	52.72	50.42	48.23	46.12	44.11
46	65.17	62.18	59.33	56.61	54.02
48	79.01	75.23	71.62	68.19	64.92
50	93.48	88.83	84.41	80.22	76.23

TABLE 7-9. SENSITIVITY ANALYSIS OF ESAL-LOAD TABLE WITH RESPECT TO n IN EQUATION 6-6

	n Values				
Load (kips)	4.20	4.15	4.10	4.05	4.00
2	0.00	0.00	0.00	0.00	0.00
4	0.00	0.00	0.00	0.00	0.00
6	0.01	0.01	0.01	0.01	0.01
8	0.03	0.04	0.04	0.04	0.04
10	0.09	0.09	0.09	0.09	0.10
12	0.18	0.19	0.19	0.19	0.20
14	0.35	0.36	0.36	0.37	0.37
16	0.61	0.62	0.62	0.62	0.63
18	1.00	1.00	1.00	1.00	1.00
20	1.56	1.55	1.54	1.53	1.53
22	2.32	2.30	2.27	2.25	2.23
24	3.36	3.31	3.27	3.22	3.17
26	4.69	4.61	4.52	4.44	4.36
28	6.44	6.30	6.16	6.02	5.89
30	8.42	8.21	8.00	7.80	7.61
32	11.18	10.86	10.56	10.26	9.97
34	14.58	14.13	13.68	13.25	12.84
36	18.78	18.13	17.51	16.91	16.33
38	23.09	22.24	21.42	20.64	19.88
40	28.98	27.84	26.75	25.70	24.69
42	35.40	33.92	32.51	31.16	29.87
44	42.19	40.35	38.60	36.91	35.31
46	51.54	49.18	46.92	44.77	42.72
48	61.81	58.85	56.03	53.35	50.79
50	72.44	68.84	65.42	62.17	59.08

TABLE 7-10. SENSITIVITY ANALYSIS OF ESAL-LOAD TABLE WITH RESPECT TO n IN EQUATION 6-6.

	n Values				
Load (kips)	3.95	3.90	3.85	3.80	3.75
2	0.00	0.00	0.00	0.00	0.00
4	0.00	0.00	0.00	0.00	0.00
6	0.01	0.01	0.01	0.02	0.02
8	0.04	0.04	0.04	0.05	0.05
10	0.10	0.10	0.10	0.11	0.11
12	0.20	0.21	0.21	0.21	0.22
14	0.37	0.38	0.38	0.39	0.39
16	0.63	0.63	0.64	0.64	0.65
18	1.00	1.00	1.00	1.00	1.00
20	1.52	1.51	1.50	1.49	1.49
22	2.21	2.18	2.16	2.14	2.12
24	3.13	3.08	3.04	3.00	2.95
26	4.28	4.20	4.12	4.05	3.98
28	5.76	5.64	5.51	5.39	5.27
30	7.42	7.23	7.05	6.87	6.70
32	9.68	9.41	9.14	8.88	8.63
34	12.43	12.04	11.66	11.30	10.94
36	15.77	15.23	14.71	14.20	13.72
38	19.15	18.45	17.77	17.12	16.49
40	23.72	22.79	21.89	21.03	20.21
42	28.62	27.43	26.29	25.20	24.15
44	33.77	32.30	30.89	29.54	28.26
46	40.76	38.89	37.11	35.41	33.78
48	48.36	46.04	43.83	41.73	39.74
50	56.14	53.35	50.70	48.18	45.78

TABLE 7-11. SENSITIVITY ANALYSIS OF ESAL-LOAD TABLE WITH RESPECT TO n IN EQUATION 6-6

	n Values				
Load (kips)	3.70	3.65	3.60	3.55	3.50
2	0.00	0.00	0.00	0.00	0.00
4	0.00	0.00	0.00	0.00	0.01
6	0.02	0.02	0.02	0.02	0.02
8	0.05	0.05	0.05	0.06	0.06
10	0.11	0.12	0.12	0.12	0.13
12	0.22	0.23	0.23	0.24	0.24
14	0.40	0.40	0.41	0.41	0.42
16	0.65	0.65	0.66	0.66	0.66
18	1.00	1.00	1.00	1.00	1.00
20	1.48	1.47	1.46	1.46	1.45
22	2.10	2.08	2.06	2.04	2.02
24	2.91	2.87	2.83	2.79	2.75
26	3.90	3.83	3.76	3.69	3.63
28	5.16	5.05	4.93	4.83	4.72
30	6.53	6.37	6.21	6.06	5.90
32	8.39	8.15	7.92	7.70	7.48
34	10.60	10.27	9.94	9.63	9.33
36	13.24	12.79	12.35	11.93	11.52
38	15.89	15.30	14.74	14.20	13.68
40	19.41	18.65	17.92	17.21	16.54
42	23.15	22.19	21.27	20.38	19.53
44	27.02	25.85	24.72	23.64	22.61
46	32.23	30.76	29.35	28.00	26.72
48	37.83	36.02	34.29	32.65	31.09
50	43.51	41.34	39.29	37.34	35.48

TABLE 7-12. SENSITIVITY ANALYSIS OF ESAL-LOAD TABLE WITH RESPECT TO n IN EQUATION 6-6

	n Values			
Load (kips)	3.45	3.40	3.35	3.30
2	0.00	0.00	0.00	0.00
4	0.01	0.01	0.01	0.01
6	0.02	0.02	0.03	0.03
8	0.06	0.06	0.07	0.07
10	0.13	0.14	0.14	0.14
12	0.25	0.25	0.26	0.26
14	0.42	0.43	0.43	0.44
16	0.67	0.67	0.68	0.68
18	1.00	1.00	1.00	1.00
20	1.44	1.43	1.43	1.42
22	2.00	1.98	1.96	1.94
24	2.71	2.67	2.63	2.59
26	3.56	3.50	3.43	3.37
28	4.62	4.52	4.42	4.32
30	5.76	5.61	5.47	5.33
32	7.27	7.06	6.86	6.67
34	9.04	8.75	8.48	8.21
36	11.12	10.74	10.37	10.02
38	13.18	12.70	12.23	11.78
40	15.89	15.26	14.66	14.09
42	18.72	17.94	17.20	16.48
44	21.63	20.69	19.78	18.92
46	25.49	24.32	23.21	22.14
48	29.60	28.18	26.83	25.54
50	33.72	32.04	30.45	28.93

according to equation 7-1 and 7-2 with respect to a single 18 kip axle load for different values of n. In other words, it represents the D_r in equation 7-2 and uses the values in Table 7-6 for axle weights between 2 kip and 50 kip for different values of n. The denominator, D_K, would be the stress peak for a standard 18 kip single axle. Tables 7-13 through 7-15 list the ESAL values used by AASHTO. On inspection, it can be seen that the ESAL values for n in the range computed here are similar to the range of ESAL values provided by AASHTO. This exercise is done, again not with the intent of obtaining the "best" value of n, but more with the idea of fixing the range of the value of n to apply to the tandem axle study discussed later in this section and proving the fact that a family of curves similar to the AASHTO ESAL family of curves can be computed by this methodology.

The second set of results relate to the determination of ESAL values to axle spacing in tandem axle combinations. Once the value of n has been fixed by methods discussed above, the damage equation can now be rerun for tandem axles with differing axle spacings. The particular cases investigated were tandem axles with the legal maximum weight of 34 kips and the axle spacings varying from a minimum of 2 feet between axles to a maximum of 10.0 feet between axles. It should be noted that two axles placed further than 8 feet apart are in general, not considered to be tandem axles and are treated as two separate single axles. Further, axles that are placed closer than about 4.0 ft will fail the Bridge Formula for a 34 kip load.

Table 7-16, 7-17 and 7-18 list the relative damage caused by tandem axles for different axle spacing for three values of n (3.5, 4.0 and 4.5) which covers most of the range as provided by the single axle calibrations. The damage in table 7-16 is computed with respect to tandem axles placed 3.0 feet apart. The relative damage in Tables 7-17 and 7-18 correspond to axle spacing of 3.5 feet and 4.0 feet respectively. As can be noted from these tables, the ESAL values computed for a tandem axle with respect to a tandem axle are not as sensitive to the value n takes as the ESAL values are sensitive to n when single axle equivalents are computed.

CONCLUSIONS

As can be determined by comparing the ESAL values of AASHTO to the ESAL values computed by means of the simulation runs, they are same family of damage curves. What is noteworthy is the fact that the techniques used in this study can be applied to axle load ranges that were not covered by the AASHTO road test data and precise ESAL value obtained for those ranges. With respect to the physical significance of n, it can be said that it would represent the structural strength of the pavement. A higher value of n would imply a strong and well designed pavement with respect to the

TABLE 7-13. ESAL FACTORS FOR FLEXIBLE PAVEMENTS, SINGLE AXLES AND A PSI OF 3.0 [AFTER REF 1]

Axle Load (kips)	Pavement Structural Number (SN)					
	1	2	3	4	5	6
2	.0008	.0009	.0006	.0003	.0002	.0002
4	.004	.008	.006	.004	.002	.002
6	.014	.030	.028	.018	.012	.010
8	.035	.070	.080	.055	.040	.034
10	.082	.132	.168	.132	.101	.086
12	.173	.231	.296	.260	.212	.187
14	.332	.388	.468	.447	.391	.358
16	.594	.633	.695	.693	.651	.622
18	1.00	1.00	1.00	1.00	1.00	1.00
20	1.60	1.53	1.41	1.38	1.44	1.51
22	2.47	2.29	1.96	1.83	1.97	2.16
24	3.67	3.33	2.69	2.39	2.60	2.96
26	5.29	4.72	3.65	3.08	3.33	3.91
28	7.43	6.56	4.88	3.93	4.17	5.00
30	10.2	8.9	6.5	5.0	5.1	6.3
32	13.8	12.0	8.4	6.2	6.3	7.7
34	18.2	15.7	10.9	7.8	7.6	9.3
36	23.8	20.4	14.0	9.7	9.1	11.0
38	30.6	26.2	17.7	11.9	11.0	13.0
40	38.8	33.2	22.2	14.6	13.1	15.3
42	48.8	41.6	27.6	17.8	15.5	17.8
44	60.6	51.6	34.0	21.6	18.4	20.6
46	74.7	63.4	41.5	26.1	21.6	23.8
48	91.2	77.3	50.3	31.3	25.4	27.4
50	110.	94.	61.	37.	30.	32.

TABLE 7-14. ESAL FACTORS FOR FLEXIBLE PAVEMENTS, SINGLE AXLES AND A PSI OF 2.5 [AFTER REF 1]

Axle Load (kips)	Pavement Structural Number (SN)					
	1	2	3	4	5	6
2	.0004	.0004	.0003	.0002	.0002	.0002
4	.003	.004	.004	.003	.002	.002
6	.011	.017	.017	.013	.010	.009
8	.032	.047	.051	.041	.034	.031
10	.078	.102	.118	.102	.088	.080
12	.168	.198	.229	.213	.189	.176
14	.328	.358	.399	.388	.360	.342
16	.591	.613	.646	.645	.623	.606
18	1.00	1.00	1.00	1.00	1.00	1.00
20	1.61	1.57	1.49	1.47	1.51	1.55
22	2.48	2.38	2.17	2.09	2.18	2.30
24	3.69	3.49	3.09	2.89	3.03	3.27
26	5.33	4.99	4.31	3.91	4.09	4.48
28	7.49	6.98	5.90	5.21	5.39	5.98
30	10.3	9.5	7.9	6.8	7.0	7.8
32	13.9	12.8	10.5	8.8	8.9	10.0
34	18.4	16.9	13.7	11.3	11.2	12.5
36	24.0	22.0	17.7	14.4	13.9	15.5
38	30.9	28.3	22.6	18.1	17.2	19.0
40	39.3	35.9	28.5	22.5	21.1	23.0
42	49.3	45.0	35.6	27.8	25.6	27.7
44	61.3	55.9	44.0	34.0	31.0	33.1
46	75.5	68.8	54.0	41.4	37.2	39.3
48	92.2	83.9	65.7	50.1	44.5	46.5
50	112.	102.	79.	60.	53.	55.

TABLE 7-15. ESAL FACTORS FOR FLEXIBLE PAVEMENTS, SINGLE AXLES AND A PSI OF 2.0 [AFTER REF 1.]

Axle Load (kips)	Pavement Structural Number (SN)					
	1	2	3	4	5	6
2	.0002	.0002	.0002	.0002	.0002	.0002
4	.002	.003	.002	.002	.002	.002
6	.009	.012	.011	.010	.009	.009
8	.030	.035	.036	.033	.031	.029
10	.075	.085	.090	.085	.079	.076
12	.165	.177	.189	.183	.174	.168
14	.325	.338	.354	.350	.338	.331
16	.589	.598	.613	.612	.603	.596
18	1.00	1.00	1.00	1.00	1.00	1.00
20	1.61	1.59	1.56	1.55	1.57	1.59
22	2.49	2.44	2.35	2.31	2.35	2.41
24	3.71	3.62	3.43	3.33	3.40	3.51
26	5.36	5.21	4.88	4.68	4.77	4.96
28	7.54	7.31	6.78	6.42	6.52	6.83
30	10.4	10.0	9.2	8.6	8.7	9.2
32	14.0	13.5	12.4	11.5	11.5	12.1
34	18.5	17.9	16.3	15.0	14.9	15.6
36	24.2	23.3	21.2	19.3	19.0	19.9
38	31.1	29.9	27.1	24.6	24.0	25.1
40	39.6	38.0	34.3	30.9	30.0	31.2
42	49.7	47.7	43.0	38.6	37.2	38.5
44	61.8	59.3	53.4	47.6	45.7	47.1
46	76.1	73.0	65.6	58.3	55.7	57.0
48	92.9	89.1	80.0	70.9	67.3	68.6
50	113.	108.	97.	86.	81.	82.

TABLE 7-16. EQUIVALENT AXLE LOADS AS A FUNTION OF DISTANCE WITH RESPECT TO A 34 KIP TANDEM AXLE WITH 3 FEET AXLE SPACING

	n Values		
Distance (ft.)	3.50	4.00	4.50
2.0	0.68	0.65	0.61
2.5	0.88	0.86	0.85
3.0	1.00	1.00	1.00
3.5	1.08	1.09	1.10
4.0	1.12	1.14	1.16
4.5	1.15	1.17	1.20
5.0	1.17	1.20	1.23
5.5	1.19	1.21	1.24
6.0	1.19	1.22	1.25
6.5	1.19	1.22	1.25
7.0	1.19	1.23	1.26
7.5	1.20	1.23	1.26
8.0	1.20	1.23	1.26
8.5	1.20	1.24	1.27
9.0	1.20	1.24	1.27
9.5	1.21	1.24	1.27
10.0	1.21	1.24	1.27

TABLE 7-17. EQUIVALENT AXLE LOADS AS A FUNTION OF DISTANCE WITH RESPECT TO A 34 KIP TANDEM AXLE WITH 3.5 FEET AXLE SPACING

	n Values		
Distance (ft.)	3.50	4.00	4.50
2.0	0.63	0.59	0.56
2.5	0.82	0.80	0.77
3.0	0.93	0.92	0.91
3.5	1.00	1.00	1.00
4.0	1.04	1.05	1.06
4.5	1.07	1.08	1.09
5.0	1.09	1.10	1.12
5.5	1.10	1.12	1.13
6.0	1.10	1.12	1.14
6.5	1.11	1.12	1.14
7.0	1.11	1.13	1.14
7.5	1.11	1.13	1.15
8.0	1.12	1.13	1.15
8.5	1.12	1.14	1.16
9.0	1.12	1.14	1.16
9.5	1.12	1.14	1.16
10.0	1.12	1.14	1.16

TABLE 7-18. EQUIVALENT AXLE LOADS AS A FUNTION OF DISTANCE WITH RESPECT TO A 34 KIP TANDEM AXLE WITH 4 FEET AXLE SPACING

	n Values		
Distance (ft.)	3.50	4.00	4.50
2.0	0.61	0.57	0.53
2.5	0.78	0.76	0.73
3.0	0.89	0.88	0.86
3.5	0.96	0.95	0.95
4.0	1.00	1.00	1.00
4.5	1.02	1.03	1.03
5.0	1.05	1.05	1.06
5.5	1.06	1.06	1.07
6.0	1.06	1.07	1.08
6.5	1.06	1.07	1.08
7.0	1.06	1.07	1.08
7.5	1.07	1.08	1.09
8.0	1.07	1.08	1.09
8.5	1.07	1.08	1.09
9.0	1.07	1.08	1.09
9.5	1.08	1.09	1.10
10.0	1.08	1.09	1.10

load being carried on it, while a lower value of n would be used for pavements that are under designed for the load they experience.

Studying the tandem axle equivalencies, it can be seen that larger distances between axles would cause the load factor to increase. For example, when a tandem axle placed with a 5.5 feet gap is compared to a tandem axle with 4 feet gap, the damage caused by the former is roughly 6% to 7% (depending on the value of n) more than the damage caused by the later (Table 7-18). Moving the axles close to each other (say of the order of 2.0 feet) decreases the relative damage by about 39% (for n = 3.5) to 47% (for n=4.5). This would seem to imply that to reduce pavement damage, axles should be placed as close to each other as is practically feasible. However, as discussed earlier, other governing factors like the Bridge Formula, also have to be taken into account. In fact, in all the cases studied an listed here, the best configurations in terms of minimum pavement damage are those that are constrained by the minimum distance as imposed by the bridge formula.

The conclusion from the preceeding discussion is simply that what is good for the short span bridges of the nation (axles placed far apart) may not good for the nation's highways and what is good for the nation's highways (close placement of axles) may not be good for the short span bridges.

CHAPTER 8. SUMMARY AND RECOMMENDATIONS

Through the last several chapterss various procedures for the forecasting of Average Daily Traffic, truck weight distributions, vehicle classification data and pavement damage due to axle configurations and axle weights have been presented. Within this chapter the important concepts of each previous chapter and difficulties encountered (in the form of recommendations) during the course of the study are summarized.

SUMMARY

(1) Summary of Chapter 2: After having briefly reviewed some of the studies conducted over the past decade, one may summarize historical objectives and determine how best one can apply this knowledge to fulfill the project goals. Objectives of the previous Texas truck studies are summarized in the following phrases :

- Recommendations to improve the Texas vehicle weighing program.
- Recommendations to update the procedures involved in acquiring traffic data.
- Development of a methodology to forecast truck weight frequency distributions.
- Analysis of the effects of heavy trucks on the highway system.
- Assessment of changes in truck sizes and weights, and it's effects on the highway network.
- Assessment of changes in truck dimensions on the geometric design principles of the highway network.

The above studies recommend improved data acquisition procedures, and quantifies the effects of changes in truck weights and sizes. This study moves one step ahead in an attempt to evaluate the effects of changes in truck sizes and weights in order to better understand the relationship between these changes and their effects on pavement deterioration.

(2) Summary of Chapter 3: Since the percentage of vehicles at each of these WIM stations were similar, it was difficult to link the selected sites to any particular WIM station. Thus the weight data at each of the stations were combined to form a large data set, and the selected sites were all linked to this newly formed weight data set. This procedure was found to be the best available solution, due to the lack of sufficient weight stations in the State of Texas.

(3) Summary of Chapter 4: Choosing a specific method for forecasting any given data requires careful analysis of the data plotted, in the form of scatter plots, and then the selection of the right

methodology for the forecasting procedure. After having carefully studied the scatter plots of the historical ADT data collected at the various locations, the selection of forecasting procedures was limited to two general techniques, namely forces at work, and time series techniques. For the scatter plots that exhibited a linear trend, forces at work in the form of regression techniques were used. For the plots that exhibited cyclic variation, time series techniques were used.

(4) Summary of Chapter 5: In this chapter, the forecasting of Average Daily Traffic, and the trends of historical data for both the past and future (to the year 2005) were described. This was followed by the forecasting of truck weight distribution, with a brief history of the truck weighting program in Texas. The methodology of shifting the distribution involved the moving of the GVW cumulative frequency curve, in relation to the forecasted mean and variance values. The forecasting of vehicle classification data and a procedure for obtaining the "1974 equivalent number of trucks" from 1984 load data were also described. The methodology produced a marked increase in the loads carried by various classes of 1984 trucks as compared with 1974 trucks, which ranged from 11 percent to 150 percent.

(5) Summary of Chapter 6: This chapter deals primarily with providing an overview of the theoretical basis for constructing damage transforms from axle loads and and axle configurations. While it is not possible to discuss every aspect pertaining to this issue in its entirety in one chapter, an attempt has been made to provide the reader with as complete a discussion as is feasible given the physical constraints. While a more rigourous treatment of the subject can be found in various references mentioned throughout the chapter, the bulk of the basis of the actual theory has been developed in [Ref 8].

(6) Summary of Chapter 7: In this chapter, the damage model constructed in Chapter 6 is callibrated by use of the AASHTO Road Test Data applied to results obtained from dynamic simulation runs on a cumputer. The major findings are that the AASHTO ESAL values can be replacated by using a damage transform obtained through the use of the rate of change of stress in a pavement system. With respect to the axle configurations, it can be stated that axles placed close to each other lessen the amount of pavement damage compared to axles that are placed far apart. At distanceses of two feet, the damage is could be reduced by about 40 percent compared to the damage of four feet. At a distance of ten feet, the damage could increase by upto ten percent compared to the damage at four feet.

RECOMMENDATIONS

This report would not be complete if the number of obstacles that were faced in terms of inadequate data and the lack of substantial research in the area of highway deterioration. After a careful review of the facts collected this far, the following list of recommendations is provided.

- After analyzing the data of the number of trucks collected from the different WIM stations, it was observed that the currently-used sampling procedure produced a statistically biased number of 3S2 trucks when compared to classification counts. In other words, most of the trucks sampled were 3S2. Hence we recommend a revision of sampling procedures for truck weighing to remove selection bias. In order to collect a larger sample of truck weight data, an increase in the number is recommended of WIM stations which will help in the collection of a larger data base. A larger sample will better represent the types of trucks traversing the Texas highways.
- As mentioned in Chapter 3, ATR counts at locations that best represented typical Texas sections were unavailable. This however does not imply a dearth of ATR stations around Texas, but an imbalanced placing of the current recorders. In other words, a frequent validation of locations for ATR stations is suggested. This will help in the collection of data that is representative of the varied traffic pattems on the Texas highway network.
- As has been observed in Chapter 5, more trucks would be required to haul 1984 payloads under 1974 weight limits than actually operated in 1984. The percentages increase range from 11 percent to 150 percent depending on truck class. This suggests an ongoing increase in the loads imposed by trucks. This is bound to have a detrimental effect on the network, which is not constructed for ever increasing loads. Thus better surveillence for weight offenders, and the levying of strict penalties to those found guilty of abusing the weight laws, in direct proportion to the amount of over-weight they carry is suggested.
- During the literature search (Chapter 2), a lack of substantial research in the area of pavement deterioration was observed. Thus, in order to better understand the relationship between oversized trucks and pavement deterioration, continuous long-term research in this particular area is suggested. This will enable observation of a trend (if any) in the far reaching effects of overweight trucks.
- One way of tackling the overweight problem to some extent would be to design a new vehicle that will produce less damage per vehicle pass given current or future load limits. This, in turn, would improve the life of the pavements.

- While searching for data with regard to typical pavement cross-sections in Texas (Chapter 3), considerate difficulty was encountered in locating typical Texas sections that were best representative of the test sections represented in the survey. Thus a computer resilent data base of pavement cross-sections in the form of a computerized "control section log" is recommended. This would be an invaluable data source for future research.
- In studying the dynamics of the pavement system, a fair amount of dynamic load amplification was observed for certain pavement profiles. Currently, while other researchers have observed this phenomenon, [Ref 9], there is absolutely no study that provides a through understanding of this fact. In this respect, it would be worthwhile to further improve upon the modeling techniqes used for the simuulation runs to understand what precisely causes these dynamic load amplifications. This would prove to be invaluable in future pavement design enterprises by providing the designer with an understanding of which pavement profiles to avoid and how to best deal with the in situ soil profile to reduce dynamic load amplification, thereby increasing pavement life.

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APPENDIX A

TABLE A-1. EXAMPLE OF COMPLETED PAVEMENT SECTION SURVEY FORM

PROJECT 1	Layer Component (inches of depth)	Candidate Pavement Cross-Sections					
		A	B	C	D	E	F
Characteristics:	Surfacing	0.75	0.75	0.75	0.75	0.75	0.75
Roadway: FM (Rural) Lanes: 2	Flexible Base	9.5	8	11	14	19	26
Present ADT: 250 ADT @ 20 yrs: 700 20 yr 18 ESAL: 250000	Subbase	-	8	8	8	8	8
ATHWLD: 11000 Stiffness Coefficient: 0.20 Triaxial Class: 5.9	Overlay	1.5 @ 13 yrs	1.5 @ 13 yrs	1.5 @ 14 yrs	1.5 @ 15 yrs	1.5 @ 17 yrs	1.5 @ 18 yrs
Votes by Maintenance Engineer Panel		7	17	6	1	0	0

APPENDIX B

TABLE B-1. CLASSIFICATION DATA AT THE SELECTED SITES IN NUMBERS AND PERCENTAGES (1984)

VEHICLE TYPE	STATION NUMBER															
	1		2		3		3A		4		5		6		7	
	NO. OF VEHICLES	%	NO. OF VEHICLES	%	NO. OF VEHICLES	%	NO. OF VEHICLES	%	NO. OF VEHICLES	%	NO. OF VEHICLES	%	NO. OF VEHICLES	%	NO. OF VEHICLES	%
Cars	735	78.5	1329	80.0	3388	86.2	4039	87.0	77.86	94.8	6471	94.4	15142	92.7	11506	78.8
2D	49	5.2	99	6.0	183	4.7	173	3.7	214	2.6	192	2.8	702	4.3	525	3.6
3A()	0	0.0	12	0.7	21	0.5	64	1.4	29	0.4	14	0.2	195	1.2	67	0.5
2S1	0	0.0	3	0.2	7	0.2	10	0.2	10	0.1	8	0.1	20	.1	46	0.3
2S2	10	1.1	10	0.6	39	1.0	33	0.7	33	0.4	25	0.4	18	.1	130	0.9
3S1	0	0.0	0	0.0	1	0.0	0	0.0	0	0.0	0	0.0	8	.0	10	0.1
3S2	141	15.1	100	12.0	272	6.9	320	6.9	143	1.7	144	2.1	254	1.6	2295	15.7
2/3S12	1	0.1	9	0.5	18	0.5	1	0.0	0	0.0	3	0.0	3	.0	26	0.2
TOTAL	936	100	1662	100	3929	100	4640	100	8215	100	6857	100	16342	100	14605	100

continued

TABLE B-1. CONTINUED

VEHICLE TYPE	STATION NUMBER													
	8		9		10		11		12		13		14	
	NO. OF VEHICLES	%	NO. OF VEHICLES	%	NO. OF VEHICLES	%	NO. OF VEHICLES	%	NO. OF VEHICLES	%	NO. OF VEHICLES	%	NO. OF VEHICLES	%
Cars	9742	89.4	10770	80.5	7024	60.7	18506	85.0	7592	89.0	9974	71.2	48654	86.4
2D	359	3.3	551	4.1	471	4.1	806	3.7	222	2.6	467	3.3	1815	3.2
3A()	99	0.9	129	1.0	81	0.7	98	0.4	53	0.6	86	0.6	1406	2.5
2S1	12	0.1	17	0.1	63	0.5	68	0.3	16	0.2	43	0.3	119	0.2
2S2	78	0.7	133	1.0	213	1.8	110	0.5	29	0.3	156	1.1	245	0.4
3S1	4	0.0	2	0.0	5	0.0	3	0.0	3	0.0	4	0.0	10	0.0
3S2	592	5.4	1747	13.1	3605	31.2	2055	9.4	602	7.1	3104	22.2	4027	7.2
2/3S12	9	0.1	27	0.2	109	0.9	135	0.6	11	0.1	166	1.2	38	0.1
TOTAL	10895	100	13376	100	11571	100	21781	100	8528	100	14000	100	56314	100

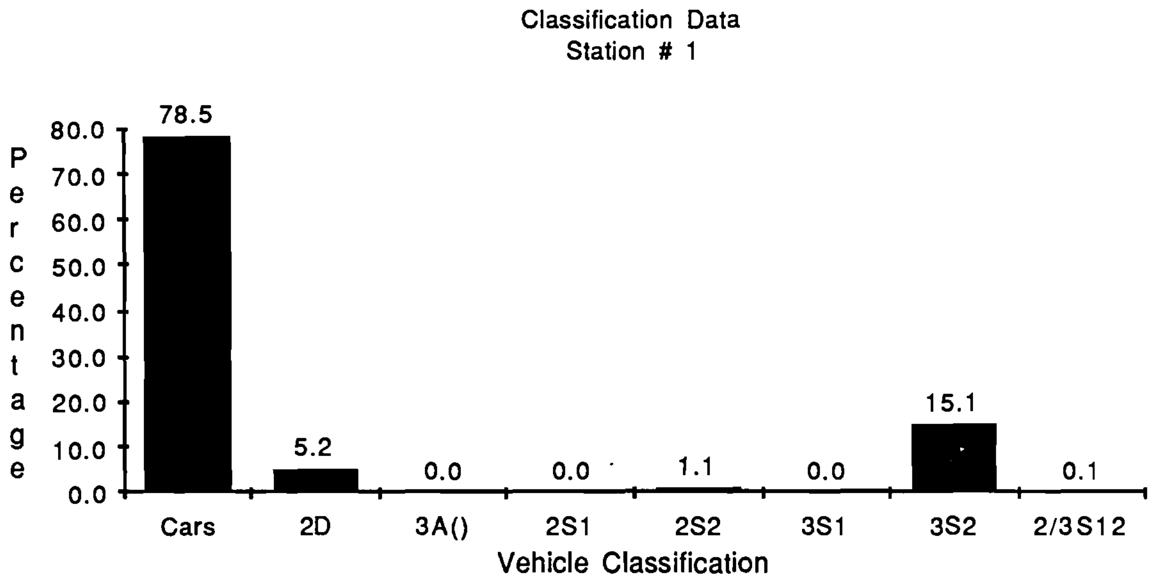

Figure B-1. Vehicle Classification Data, Station Number 1, 1984.

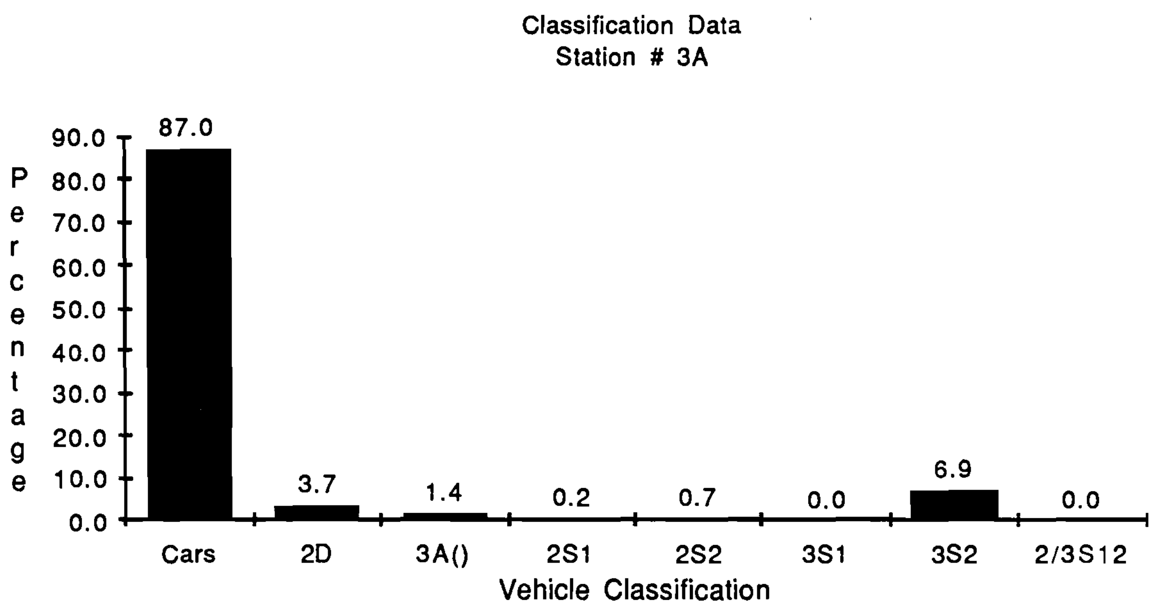

Figure B-2. Vehicle Classification Data, Station Number 3A, 1984.

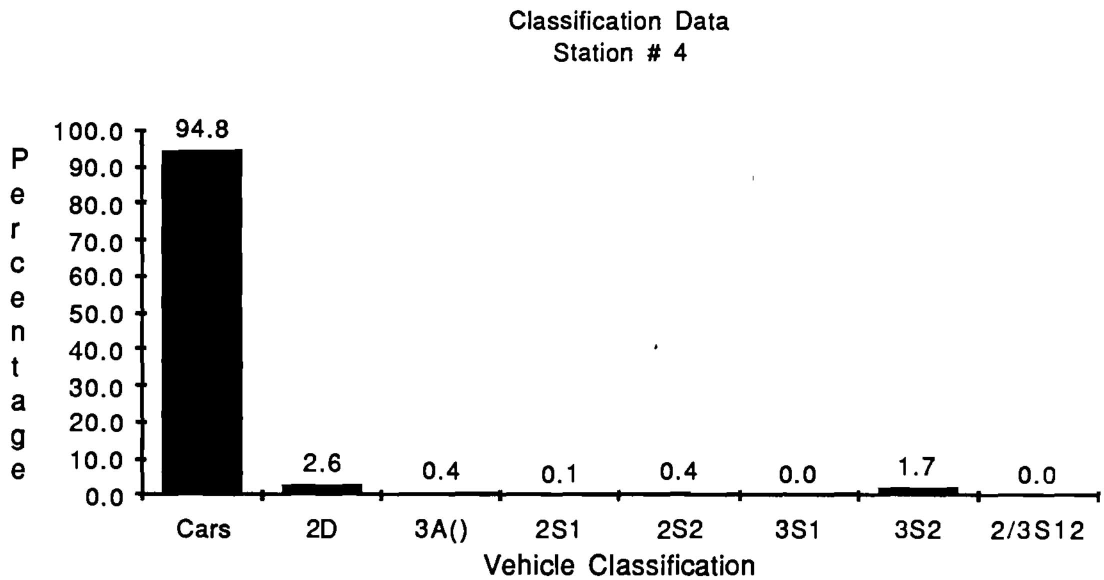

Figure B-3. Vehicle Classification Data, Station Number 4, 1984.

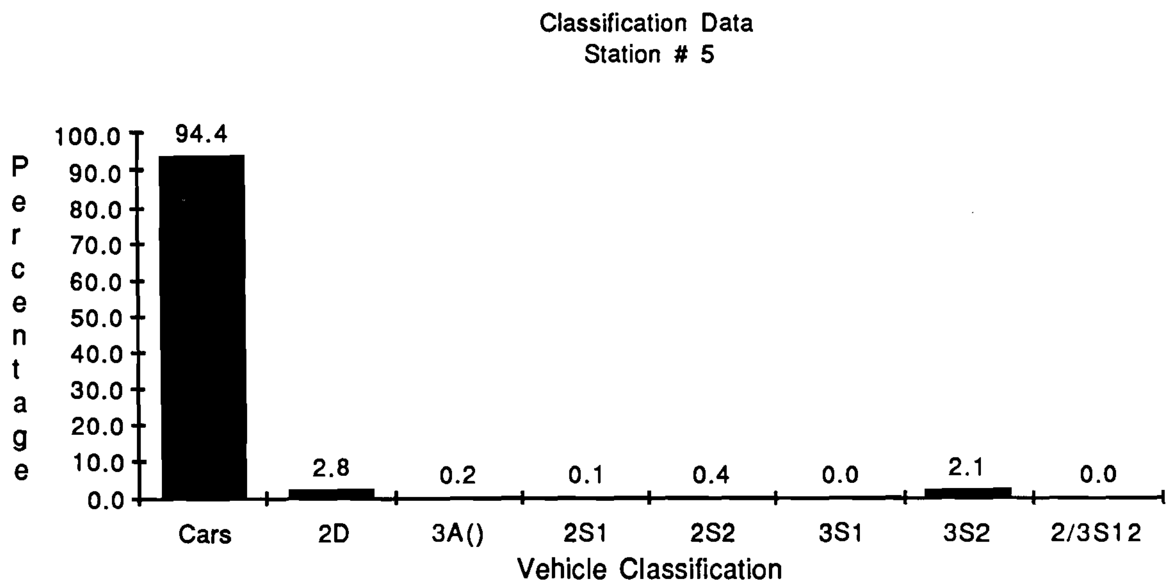

Figure B-4. Vehicle Classification Data, Station Number 5, 1984.

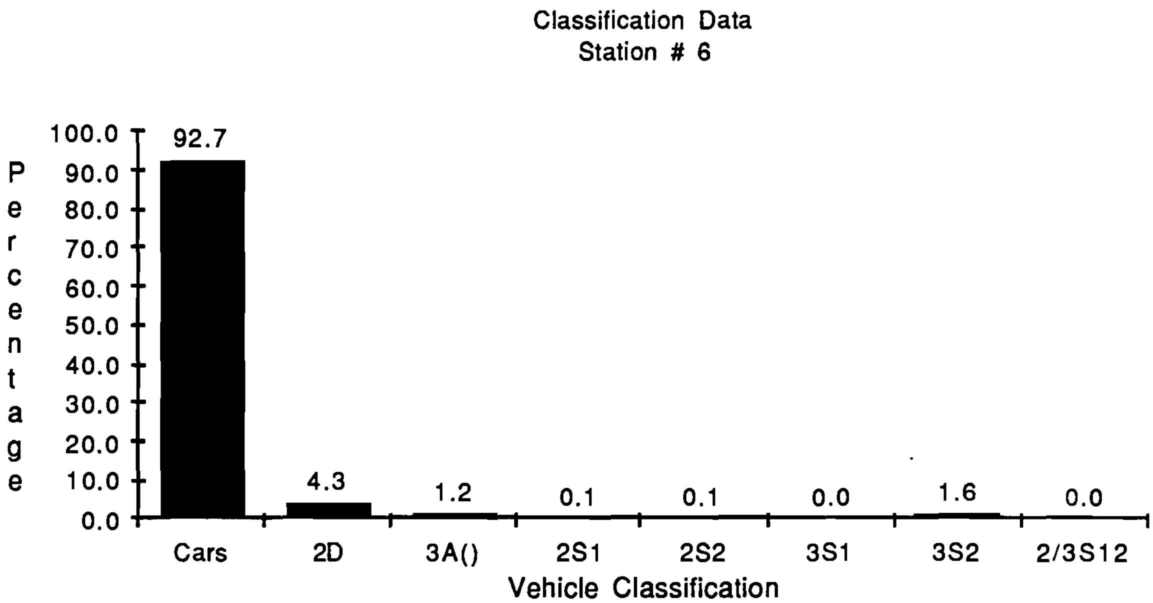

Figure B-5. Vehicle Classification Data, Station Number 6, 1984.

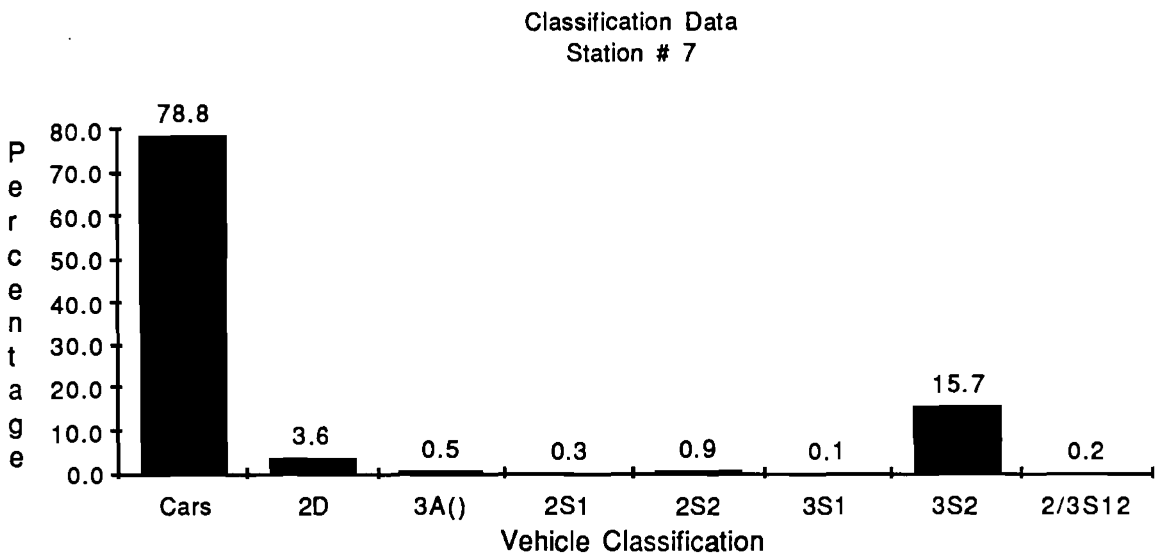

Figure B-6. Vehicle Classification Data, Station Number 7, 1984.

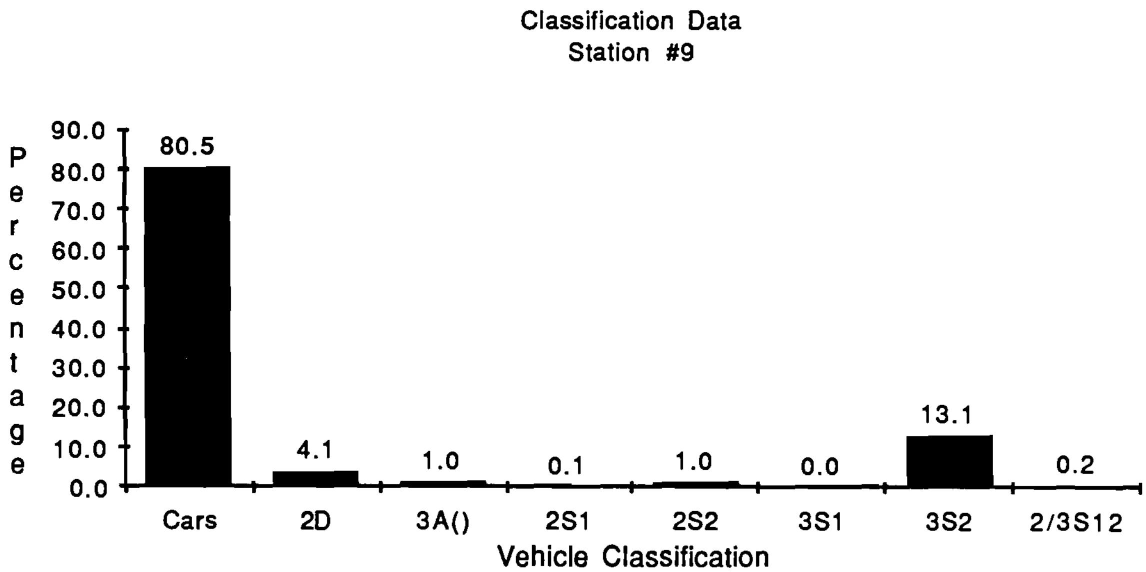

Figure B-7. Vehicle Classification Data, Station Number 9, 1984.

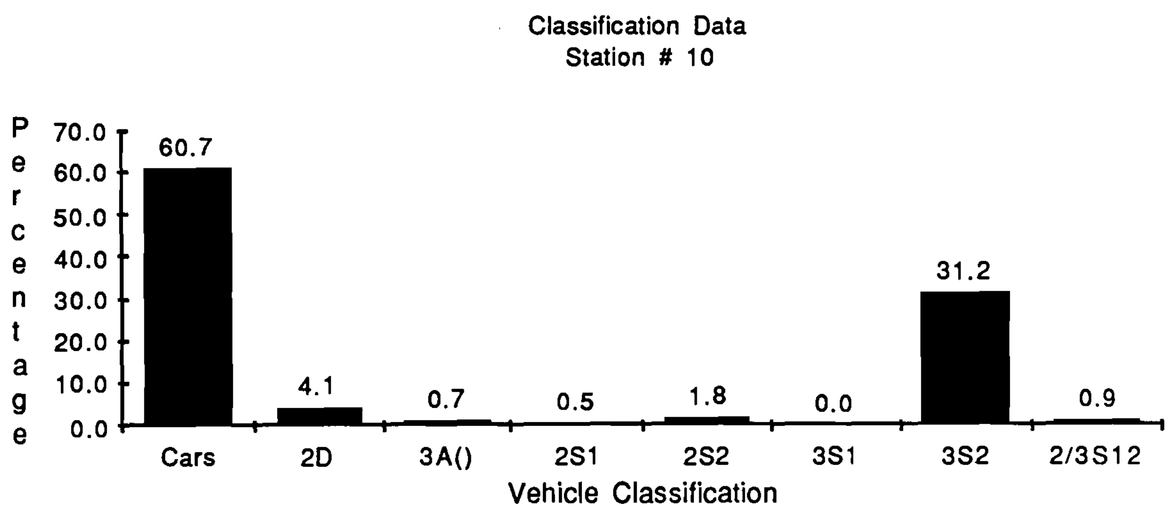

Figure B-8. Vehicle Classification Data, Station Number 10, 1984.

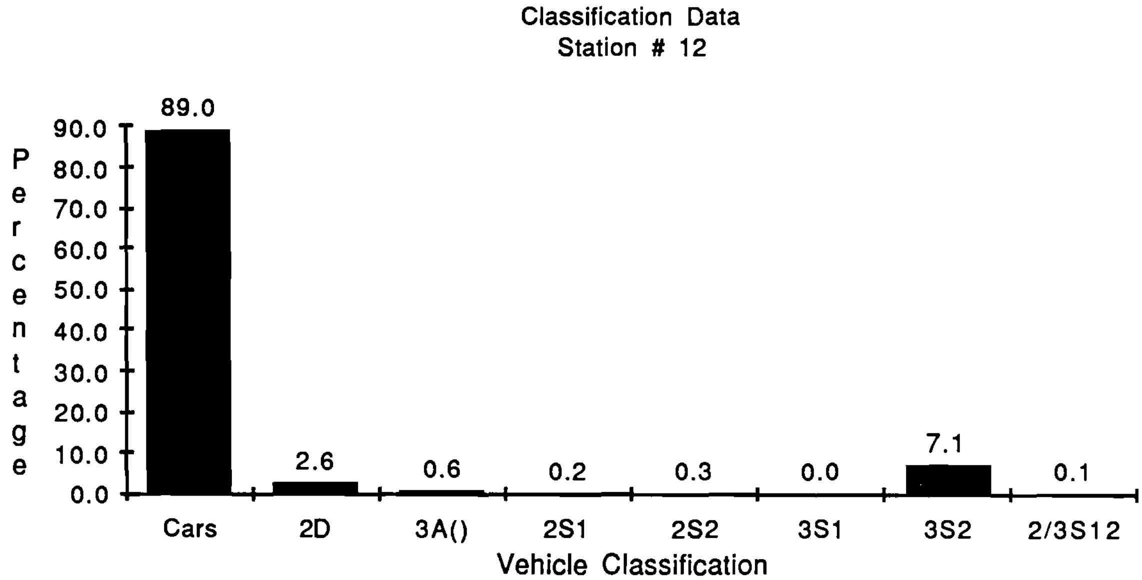

Figure B-9. Vehicle Classification Data, Station Number 12, 1984.

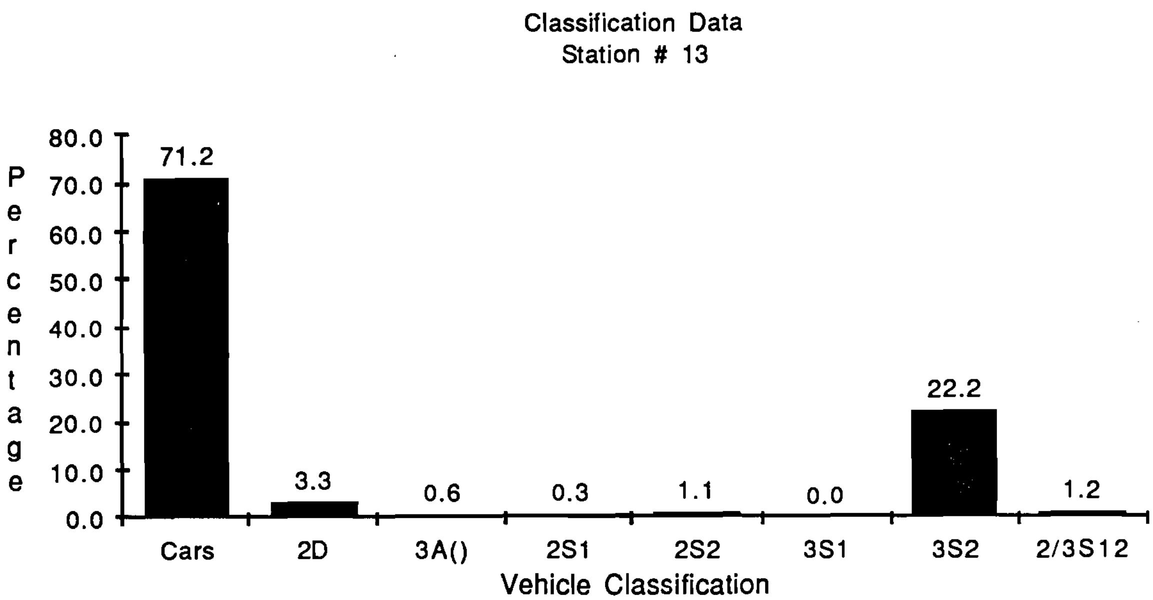

Figure B-10. Vehicle Classification Data, Station Number 13, 1984.

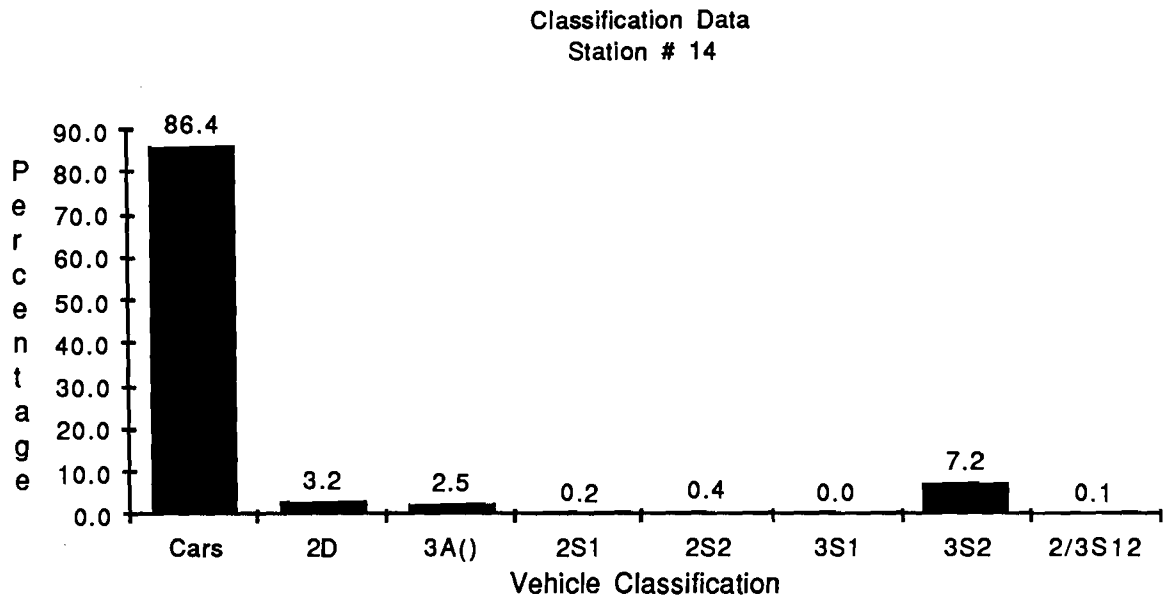

Figure B-11. Vehicle Classification Data, Station Number 14, 1984.

APPENDIX C

TABLE C-1. CHI-SQUARE TESTING, COMPARING WIM STATIONS 503 AND 502

Selection Number	Station Number 503	Station Number 502	$F_o - F_c$	$(F_o - F_c)^{**}2$	$(F_o - F_c)^{**}2/F_c$
	F_o	F_c	(B - C)	(D)**2	E/C
1	74	77.9	-3.9	15.21	0.20
2	0.8	0.1	0.7	0.49	4.90
3	1	0.8	0.2	0.04	0.05
4	0.2	0.4	-0.2	0.04	0.10
5	1.7	1.2	0.5	0.25	0.21
6	0	0	0	0	0.00
7	21.9	18.9	3	9	0.48
8	0.4	0.7	-0.3	0.09	0.13
TOTAL					6.06

TABLE C-2. CHI-SQUARE TESTING, COMPARING WIM STATIONS 504 AND 502

Selection Number 504/502	Station Number 504	Station Number 502	$F_o - F_c$	$(F_o - F_c)$**2	$(F_o - F_c)$**2/F_c
	F_o	F_c	(B - C)	(D)**2	E/C
1	71.1	77.9	-6.8	46.24	0.59
2	0.6	0.1	0.5	0.25	2.50
3	0.9	0.8	0.1	0.01	0.01
4	0.8	0.4	0.4	0.16	0.40
5	1.2	1.2	0	0	0.00
6	0.1	0	0.1	0.01	0.00
7	24.2	18.9	5.3	28.09	1.49
8	1.1	0.7	0.4	0.16	0.23
TOTAL					5.22

TABLE C-3. CHI-SQUARE TESTING, COMPARING WIM STATIONS 505 AND 502

Selection Number	Station Number 505	Station Number 502	$F_o - F_c$	$(F_o - F_c)^{**}2$	$(F_o - F_c)^{**}2/F_c$
	F_o	F_c	(B - C)	(D)**2	E/C
1	82.1	77.9	4.2	17.64	0.23
2	0.2	0.1	0.1	0.01	0.10
3	0.5	0.8	-0.3	0.09	0.11
4	0.3	0.4	-0.1	0.01	0.03
5	0.9	1.2	-0.3	0.09	0.08
6	0.1	0	0.1	0.01	0.00
7	15.7	18.9	-3.2	10.24	0.54
8	0.2	0.7	-0.5	0.25	0.36
TOTAL					1.44

TABLE C-4. CHI-SQUARE TESTING, COMPARING WIM STATIONS 506 AND 502

Selection Number	Station Number 506	Station Number 502	$F_o - F_c$	$(F_o - F_c)$**2	$(F_o - F_c)$**2/F_c
	F_o	F_c	(B - C)	(D)**2	E/C
1	72.7	77.9	-5.2	27.04	0.35
2	0.4	0.1	0.3	0.09	0.90
3	1.2	0.8	0.4	0.16	0.20
4	0.3	0.4	-0.1	0.01	0.03
5	1.2	1.2	0	0	0.00
6	0	0	0	0	0.00
7	22.8	18.9	3.9	15.21	0.80
8	1.4	0.7	0.7	0.49	0.70
TOTAL					2.98

TABLE C-5. CHI-SQUARE TESTING, COMPARING WIM STATIONS 503 AND 501

Selection Number	Station Number 503	Station Number 501	$F_o - F_c$	$(F_o - F_c)^{**}2$	$(F_o - F_c)^{**}2/F_c$
	F_o	F_c	(B - C)	(D)**2	E/C
1	74	85.5	11.5	132.25	1.55
2	0.8	0.7	-0.1	0.01	0.01
3	1	0.5	-0.5	0.25	0.50
4	0.2	0.3	0.1	0.01	0.03
5	1.7	0.6	-1.1	1.21	2.02
6	0	0	0	0	0.00
7	21.9	12.1	-9.8	96.04	7.94
8	0.4	0.3	-0.1	0.01	0.03
TOTAL					12.08

TABLE C-6. CHI-SQUARE TESTING, COMPARING WIM STATIONS 504 AND 501

Selection Number	Station Number 504	Station Number 501	$F_o - F_c$	$(F_o - F_c)$**2	$(F_o - F_c)$**2/F_c
	F_o	F_c	(B - C)	(D)**2	E/C
1	7.1	85.5	-14.4	207.36	2.43
2	0.6	0.7	-0.1	0.01	0.01
3	0.9	0.5	0.4	0.16	0.32
4	0.8	0.3	0.5	0.25	0.83
5	1.2	0.6	0.6	0.36	0.60
6	0.1	0	0.1	0.01	0.00
7	24.2	12.1	12.1	146.41	12.10
8	1.1	0.3	0.8	0.64	2.13
TOTAL					18.43

TABLE C-7. CHI-SQUARE TESTING, COMPARING WIM STATIONS 505 AND 501

Selection Number	Station Number 505	Station Number 501	$F_o - F_c$	$(F_o - F_c)^{**}2$	$(F_o - F_c)^{**}2/F_c$
	F_o	F_c	(B - C)	(D)**2	E/C
1	82.1	85.5	-3.4	11.56	0.14
2	0.2	0.7	-0.5	0.25	0.36
3	0.5	0.5	0	0	0.00
4	0.3	0.3	0	0	0.00
5	0.9	0.6	0.3	0.09	0.15
6	0.1	0	0.1	0.01	0.00
7	15.7	12.1	3.6	12.96	1.07
8	0.2	0.3	-0.1	0.01	0.03
TOTAL					1.75

TABLE C-8. CHI-SQUARE TESTING, COMPARING WIM STATIONS 506 AND 501

Selection Number	Station Number 506	Station Number 501	$F_o - F_c$	$(F_o - F_c)^{**2}$	$(F_o - F_c)^{**2}/F_c$
	F_o	F_c	(B - C)	(D)**2	E/C
1	72.7	85.5	-12.8	163.84	1.92
2	0.4	0.7	-0.3	0.09	0.13
3	1.2	0.5	0.7	0.49	0.98
4	0.3	0.3	0	0	0.00
5	1.2	0.6	0.6	0.36	0.60
6	0	0	0	0	0.00
7	22.8	12.1	10.7	114.49	9.46
8	1.4	0.3	1.1	1.21	4.03
TOTAL					17.12

TABLE C-9. CHI-SQUARE TESTING, COMPARING WIM STATIONS 504 AND 503

Selection Number	Station Number 504	Station Number 503	$F_o - F_c$	$(F_o - F_c)^{**}2$	$(F_o - F_c)^{**}2/F_c$
	F_o	F_c	(B - C)	(D)**2	E/C
1	71.1	74	-2.9	8.41	0.11
2	0.6	0.8	-0.2	0.04	0.05
3	0.9	1	-0.1	0.01	0.01
4	0.8	0.2	0.6	0.36	1.80
5	1.2	1.7	-0.5	0.25	0.15
6	0.1	0	0.1	0.01	0.00
7	24.2	21.9	2.3	5.29	0.24
8	1.1	0.4	0.7	0.49	1.23
TOTAL					3.59

TABLE C-10. CHI-SQUARE TESTING, COMPARING WIM STATIONS 505 AND 503

Selection Number	Station Number 505	Station Number 503	$F_o - F_c$	$(F_o - F_c)^{**2}$	$(F_o - F_c)^{**2}/F_c$
	F_o	F_c	(B - C)	(D)**2	E/C
1	82.1	74	8.1	65.61	0.89
2	0.2	0.8	-0.6	0.36	0.45
3	0.5	1	-0.5	0.25	0.25
4	0.3	0.2	0.1	0.01	0.05
5	0.9	1.7	-0.8	0.64	0.38
6	0.1	0	0.1	0.01	0.00
7	15.7	21.9	-6.2	38.44	1.76
8	0.2	0.4	-0.2	0.04	0.10
TOTAL					3.87

TABLE C-11. CHI-SQUARE TESTING, COMPARING WIM STATIONS 506 AND 503

Selection Number	Station Number 506	Station Number 503	$F_o - F_c$	$(F_o - F_c)$**2	$(F_o - F_c)$**2/F_c
	F_o	F_c	(B - C)	(D)**2	E/C
1	72.7	74	-1.3	1.69	0.02
2	0.4	0.8	-0.4	0.16	0.20
3	1.2	1	0.2	0.04	0.04
4	0.3	0.2	0.1	0.01	0.05
5	1.2	1.7	-0.5	0.25	0.15
6	0	0	0	0	0.00
7	22.8	21.9	0.9	0.81	0.04
8	1.4	0.4	1	1	2.50
TOTAL					3.00

TABLE C-12. CHI-SQUARE TESTING, COMPARING WIM STATIONS 505 AND 504

Selection Number	Station Number 505	Station Number 504	$F_o - F_c$	$(F_o - F_c)$**2	$(F_o - F_c)$**2/F_c
	F_o	F_c	(B - C)	(D)**2	E/C
1	82.1	71.1	11	121	1.70
2	0.2	0.6	-0.4	0.16	0.27
3	0.5	0.9	-0.4	0.16	0.18
4	0.3	0.8	-0.5	0.25	0.31
5	0.9	1.2	-0.3	0.09	0.08
6	0.1	0.1	0	0	0.00
7	15.7	24.2	-8.5	72.25	2.99
8	0.2	1.1	-0.9	0.81	0.74
TOTAL					6.26

TABLE C-13. CHI-SQUARE TESTING, COMPARING WIM STATIONS 506 AND 504

Selection Number	Station Number 506	Station Number 504	$F_o - F_c$	$(F_o - F_c)^{**2}$	$(F_o - F_c)^{**2}/F_c$
	F_o	F_c	(B - C)	(D)**2	E/C
1	72.7	71.1	1.6	2.56	0.04
2	0.4	0.6	-0.2	0.04	0.07
3	1.2	0.9	0.3	0.09	0.10
4	0.3	0.8	-0.5	0.25	0.31
5	1.2	1.2	0	0	0.00
6	0	0.1	-0.1	0.01	0.10
7	22.8	24.2	-1.4	1.96	0.08
8	1.4	1.1	0.3	0.09	0.08
TOTAL					0.78

TABLE C-14. CHI-SQUARE TESTING, COMPARING WIM STATIONS 506 AND 505

Selection Number	Station Number 506	Station Number 505	$F_o - F_c$	$(F_o - F_c)$**2	$(F_o - F_c)$**2/F_c
	F_o	F_c	(B - C)	(D)**2	E/C
1	72.7	82.1	-9.4	88.36	1.08
2	0.4	0.2	0.2	0.04	0.20
3	1.2	0.5	0.7	0.49	0.98
4	0.3	0.3	0	0	0.00
5	1.2	0.9	0.3	0.09	0.10
6	0	0.1	-0.1	0.01	0.10
7	22.8	15.7	7.1	50.41	3.21
8	1.4	0.2	1.2	1.44	7.20
TOTAL					12.87

APPENDIX D

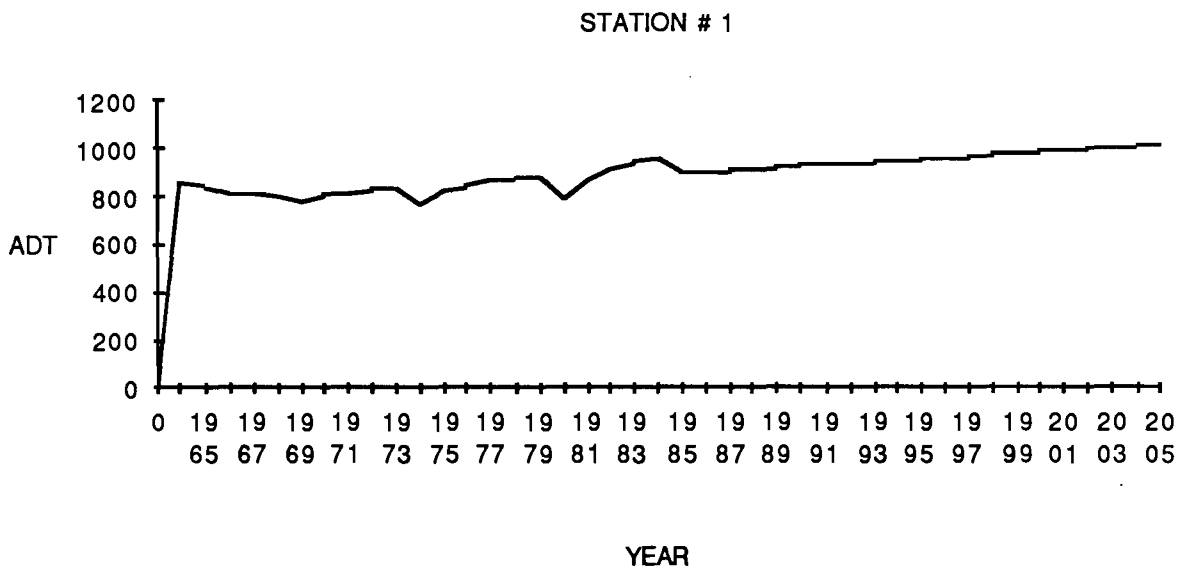

Figure D-1. Average Daily Traffic for Station Number 1 (1964-2005).

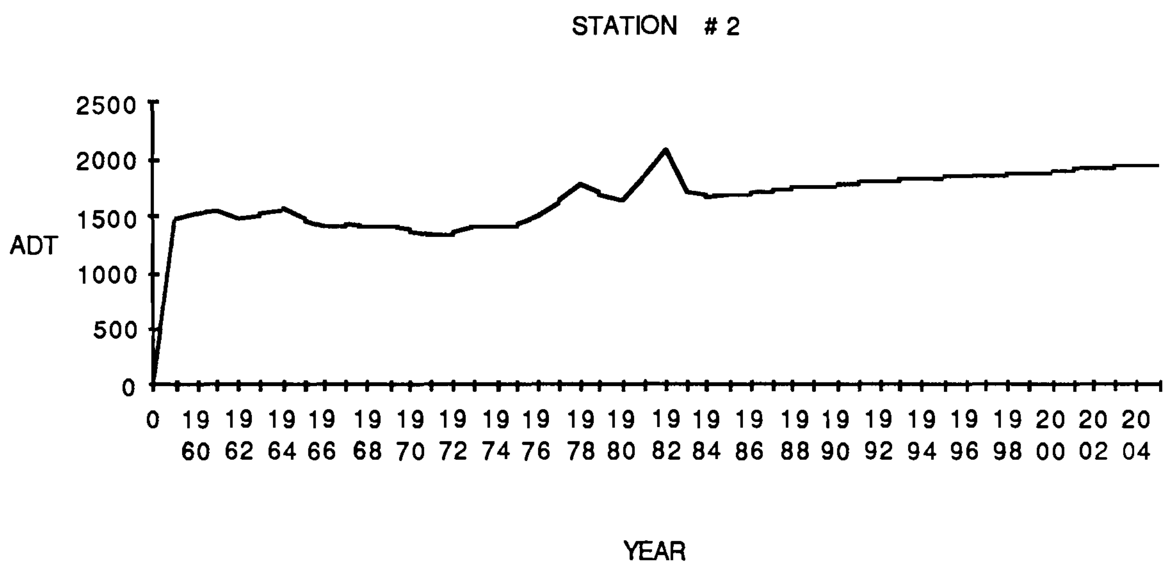

Figure D-2. Average Daily Traffic for Station Number 2 (1959-2005).

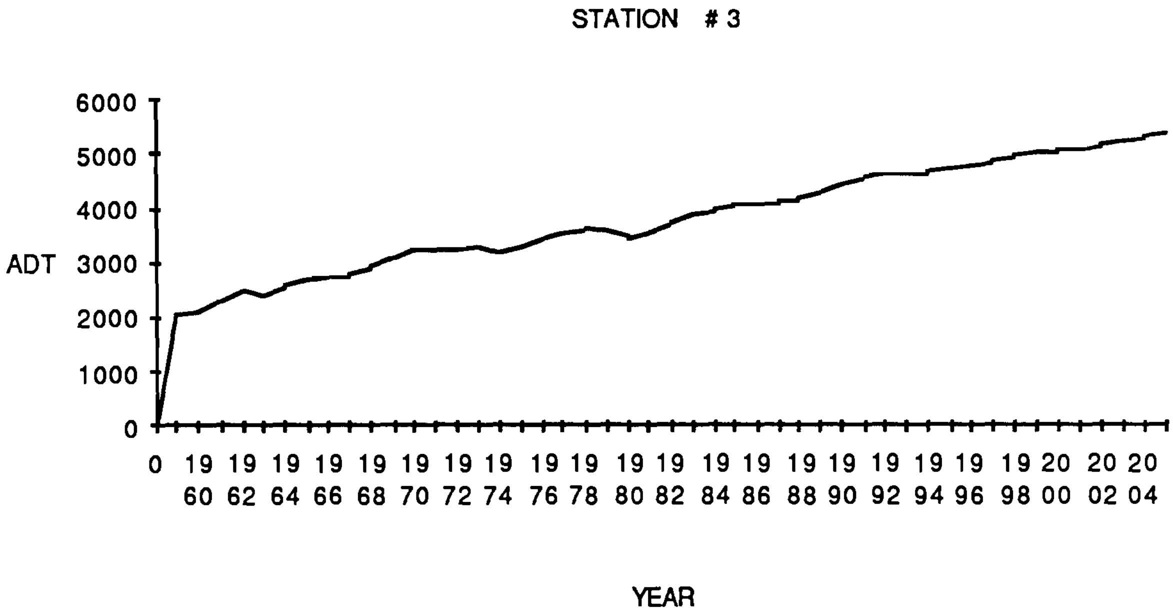

Figure D-3. Average Daily Traffic for Station Number 3 (1959-2005).

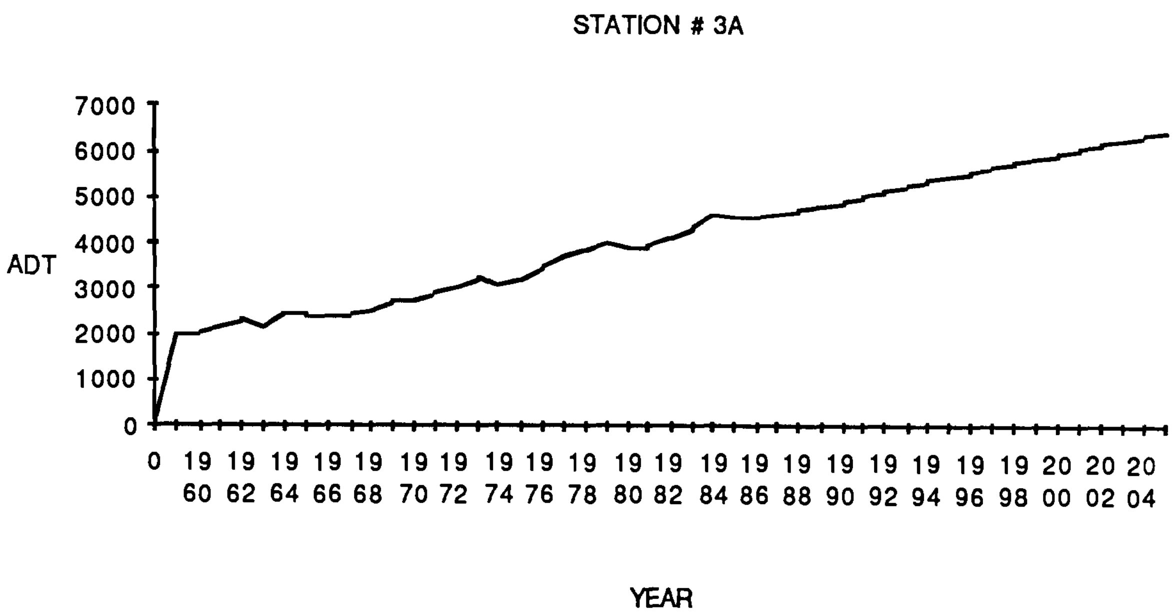

Figure D-4. Average Daily Traffic for Station Number 3A (1959-2005).

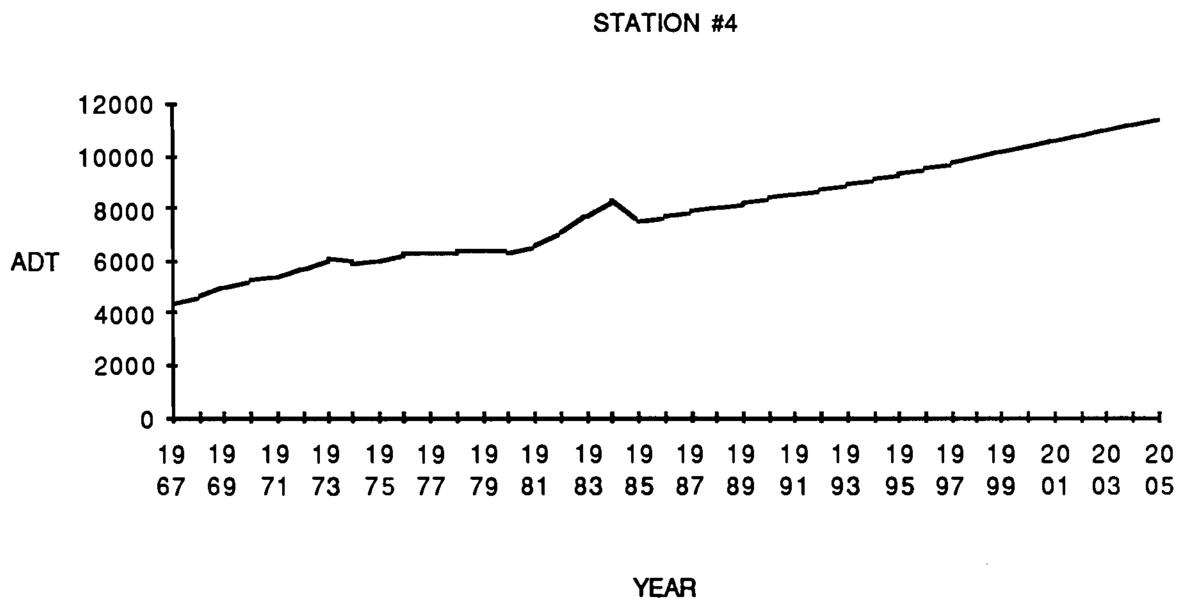

Figure D-5. Average Daily Traffic for Station Number 4 (1967-2005).

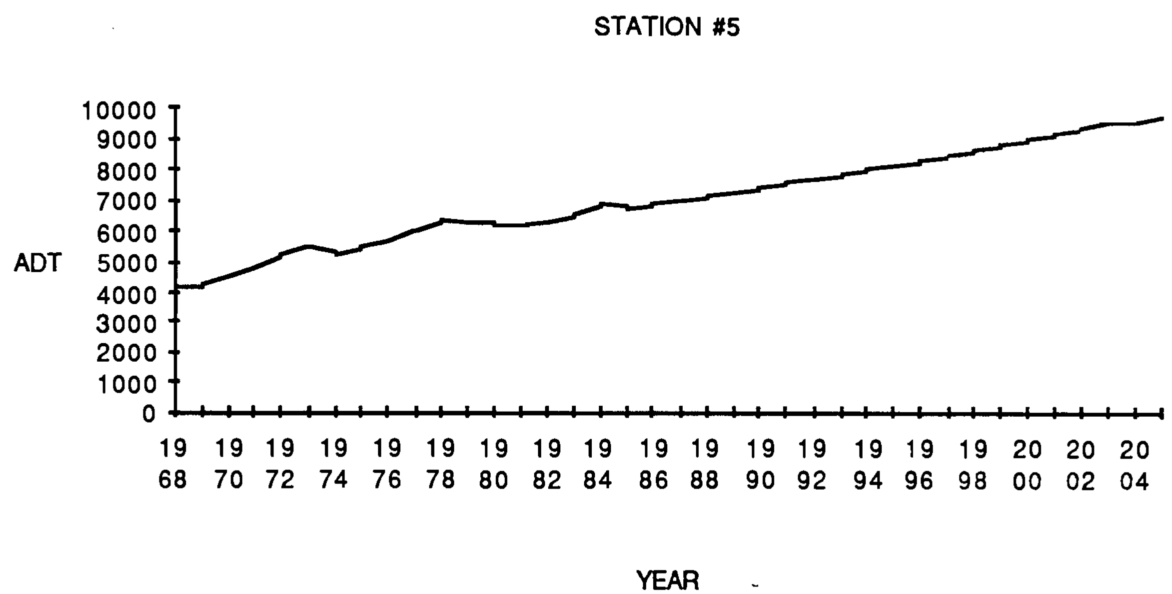

Figure D-6. Average Daily Traffic for Station Number 5 (1958-2005).

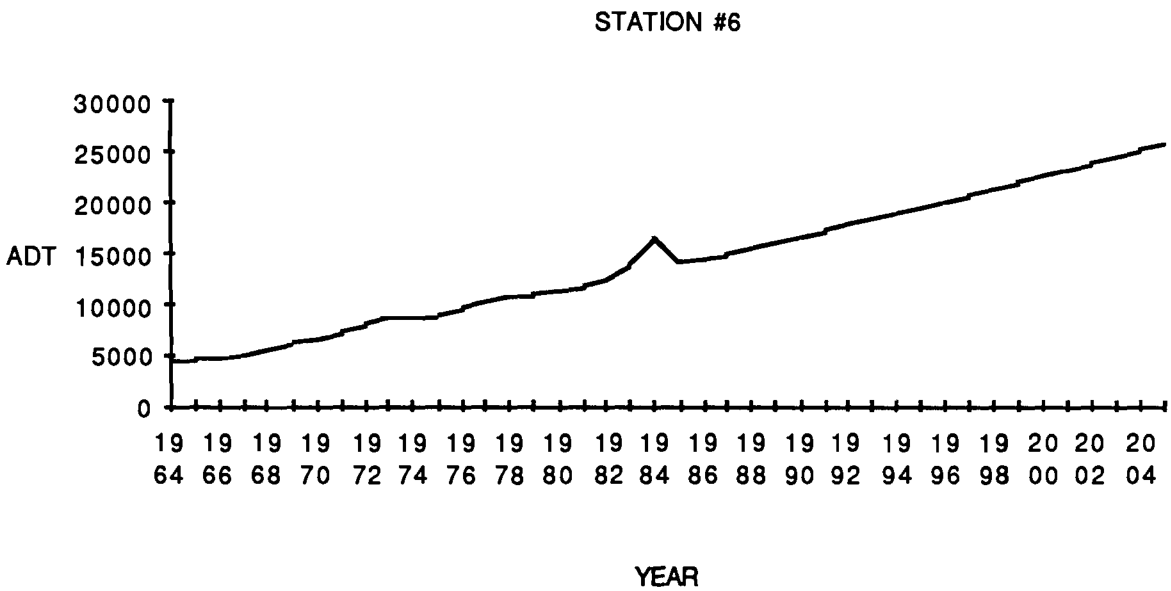

Figure D-7. Average Daily Traffic for Station Number 6 (1964-2005).

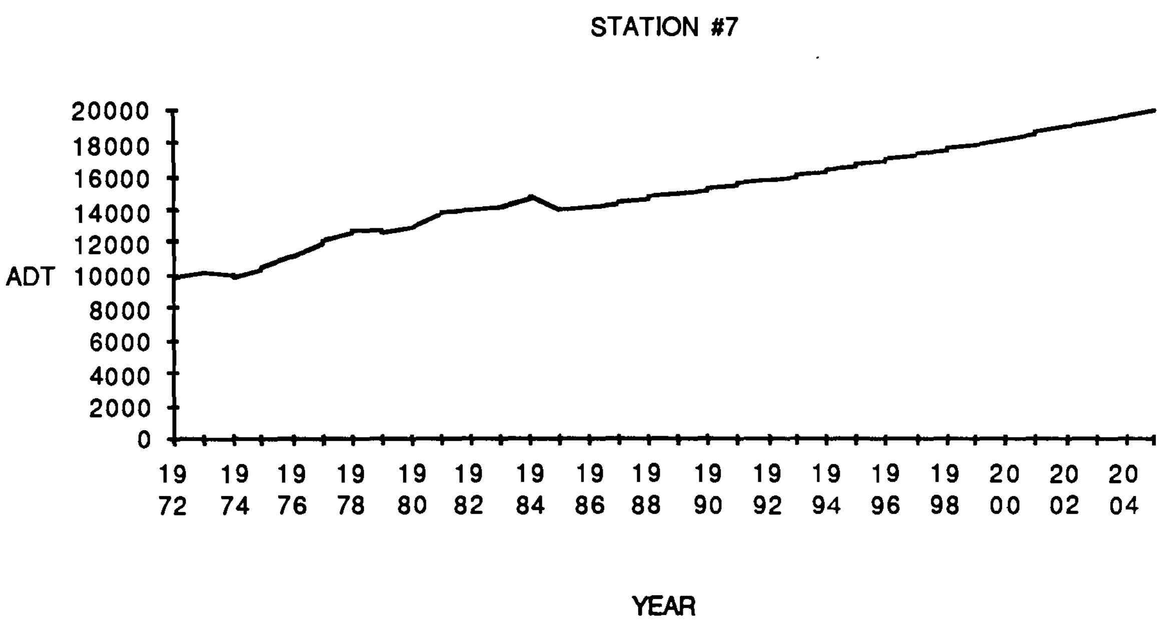

Figure D-8. Average Daily Traffic for Station Number 7 (1972-2005).

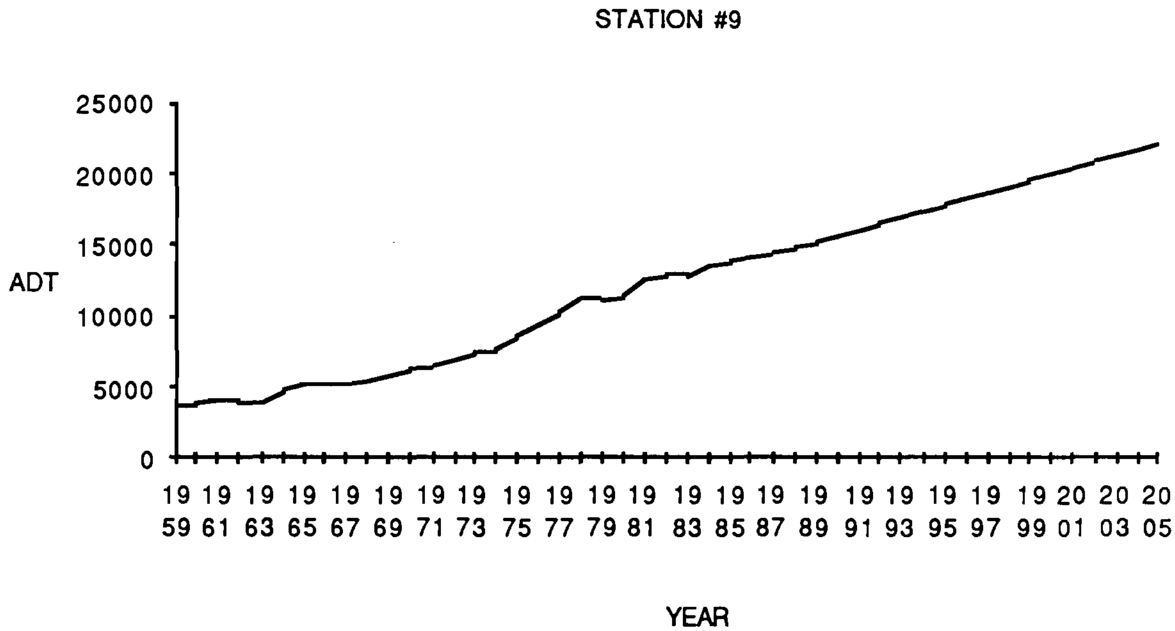

Figure D-9. Average Daily Traffic for Station Number 9 (1959-2005).

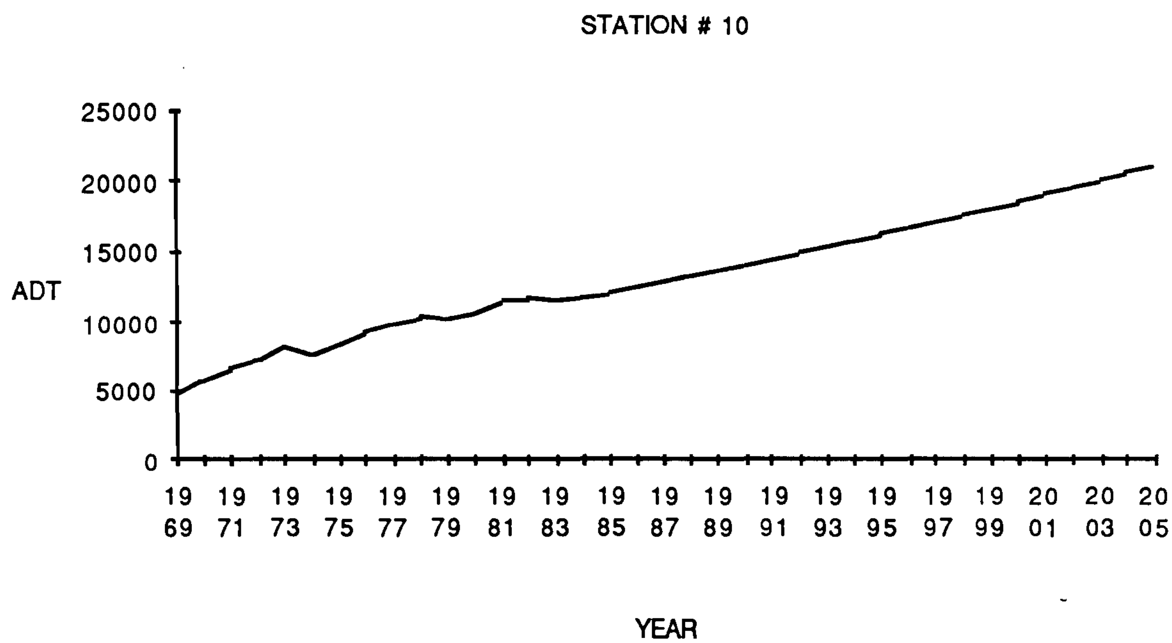

Figure D-10. Average Daily Traffic for Station Number 10 (1969-2005).

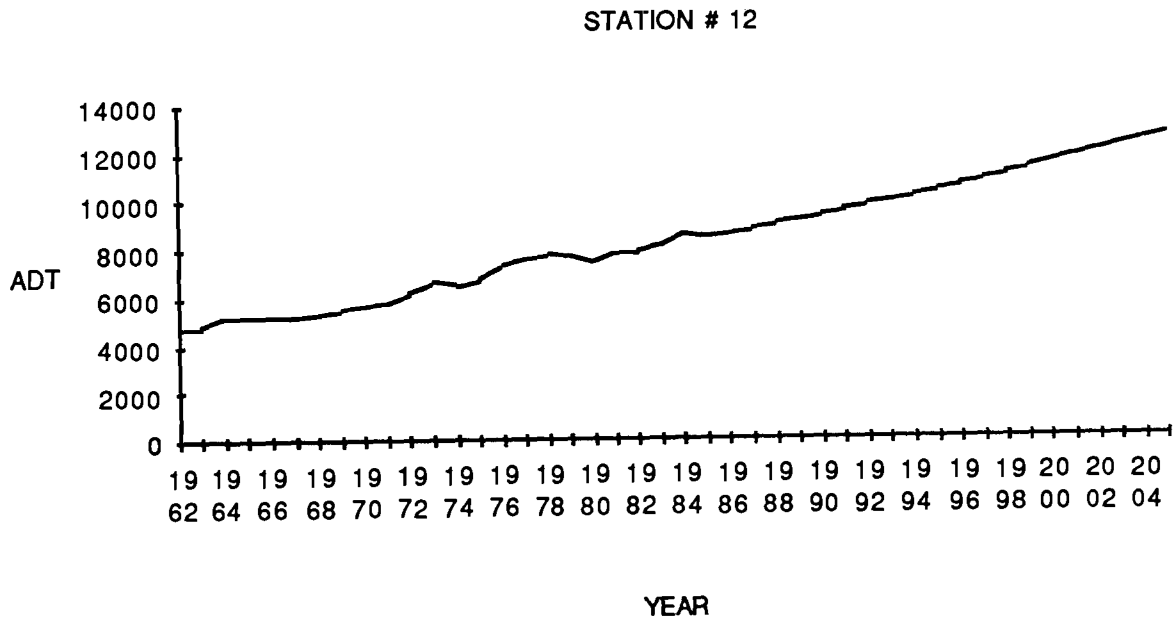

Figure D-11. Average Daily Traffic for Station Number 12 (1962-2005).

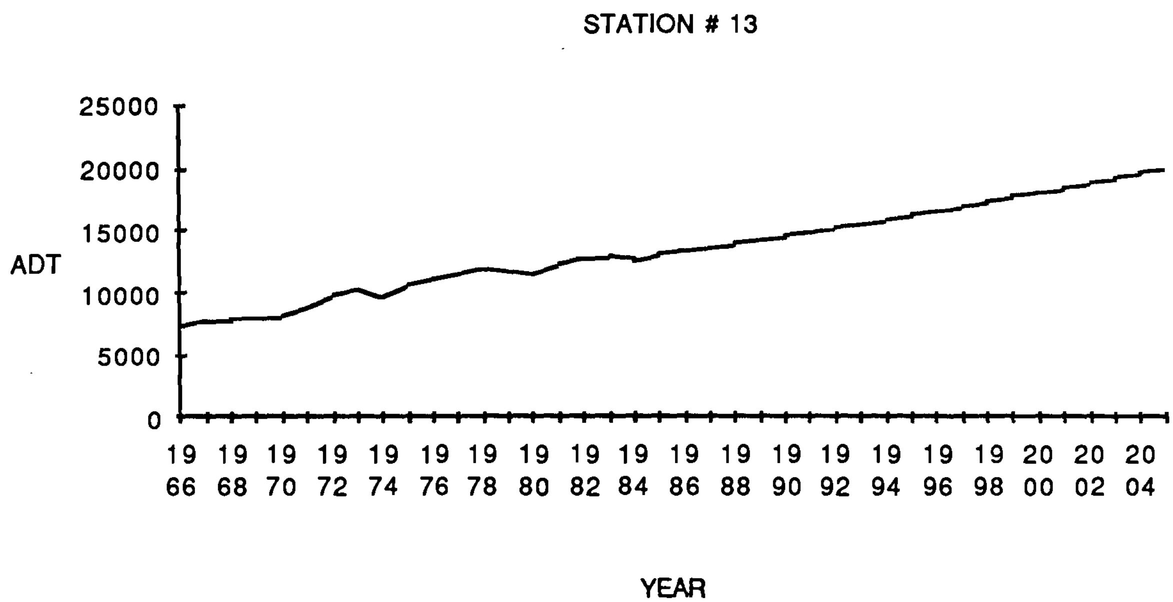

Figure D-12. Average Daily Traffic for Station Number 13 (1966-2005).

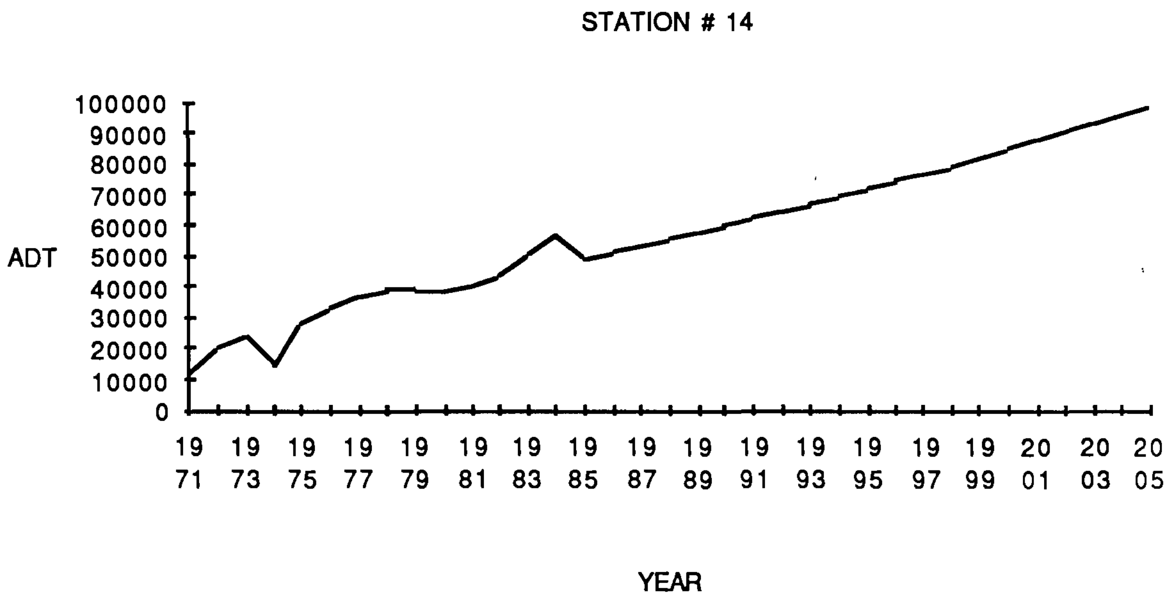

Figure D-13. Average Daily Traffic for Station Number 14 (1971-2005).

APPENDIX E

TABLE E-1. AVERAGE DAILY TRAFFIC AT THE SELECTED STATIONS (1959-2005)

YEAR	STATION														
	1	2	3	3A	4	5	6	7	8	9	10	11	12	13	14
1959	0	1472	2025	2002	0	0	0	0	0	3728	0	2966	0	0	0
1960	0	1526	2099	2020	0	0	0	0	0	3796	0	3006	0	0	0
1961	0	1560	2297	2161	0	0	0	0	0	3996	0	3330	0	0	0
1962	0	1486	2496	2327	0	0	0	0	0	3885	0	4260	4787	0	0
1963	0	1529	2379	2142	0	0	0	0	0	3953	0	4848	4822	0	0
1964	853	1570	2573	2465	0	0	4494	0	0	4765	0	6405	5206	0	0
1965	837	1469	2679	2409	0	0	4598	0	0	5211	0	6982	5137	0	0
1966	811	1421	2748	2379	0	0	4700	0	0	5137	0	7426	5140	7200	0
1967	817	1436	2790	2434	4354	0	5073	0	0	5204	0	7592	5182	7787	0
1968	801	1409	2926	2514	4609	4176	5595	0	4292	5414	0	8184	5350	7971	0
1969	777	1413	3093	2728	4974	4259	6273	0	4433	5796	4776	9034	5497	7947	0
1970	810	1371	3253	2741	5224	4530	6693	0	4623	6355	5722	10926	5617	8079	0
1971	812	1340	3231	2887	5397	4821	7396	0	4984	6471	6613	12092	5795	8745	11979
1972	834	1360	3229	3043	5704	5195	8100	9816	5526	6954	7297	13145	6198	9727	20012
1973	836	1414	3289	3270	6038	5457	8772	10178	6116	7465	8260	14179	6597	10250	23946
1974	766	1404	3170	3093	5877	5251	8767	9900	6248	7633	7605	13529	6436	9616	14660
1975	824	1425	3269	3221	5982	5493	8997	10464	6744	8542	8422	14566	6767	10575	28315
1976	846	1505	3427	3460	6229	5693	9628	11171	7216	9388	9268	15393	7294	11147	33241
1977	872	1641	3557	3725	6319	6041	10202	12015	7731	10301	9975	16902	7608	11589	36895
1978	880	1793	3650	3847	6414	6370	10809	12653	8133	11299	10501	17934	7806	12018	39049
1979	884	1703	3575	4023	6376	6244	10986	12534	8337	11173	10250	17775	7650	11749	38695
1980	789	1643	3457	3900	6302	6205	11417	12944	8517	11516	10620	17779	7419	11505	38564
1981	870	1855	3556	3941	6598	6188	11902	13872	9615	12705	11560	17781	7792	12295	40341
1982	915	2091	3715	4114	7059	6278	12454	14075	10013	12913	11706	18328	7869	12787	43767
1983	946	1705	3898	4344	7693	6527	13959	14247	10189	12771	11501	19328	8144	12883	50213
1984	964	1676	3972	4651	8275	6863	16507	14823	10959	13610	11715	20859	8609	12533	56838
1985	902	1702	4059	4587	7533	6727	14122	13996	11608	13895	12087	21929	8441	13089	49275
1986	907	1723	4106	4610	7697	6858	14622	14244	12041	14211	12466	22585	8619	13382	51335
1987	912	1742	4143	4673	7864	6992	15132	14497	12474	14552	12852	22940	8801	13682	53489
1988	916	1759	4187	4756	8035	7128	15653	14755	12906	14912	13246	23596	8987	13987	55585
1989	926	1774	4294	4849	8208	7267	16183	15019	13339	15287	13648	24534	9176	14298	57774
1990	931	1789	4450	4946	8385	7408	16724	15288	13771	15673	14057	25758	9368	14615	60006
1991	937	1801	4563	5059	8565	7552	17275	15562	41203	16113	14474	26893	9564	14937	62280
1992	941	1813	4618	5167	8749	7699	17835	15842	14635	16551	14899	27754	9763	15265	64598
1993	947	1826	4635	5273	8936	7848	18406	16127	15068	16987	15331	28176	9965	15599	66958
1994	952	1838	4676	5377	9126	8000	18987	16418	15499	17423	15770	28488	10171	15939	69361
1995	958	1851	4726	5480	9319	8155	19578	16713	15931	17857	16217	28943	10381	16284	71807
1996	964	1851	4795	5583	9515	8312	20178	17015	16363	18291	16672	29716	10594	16635	74296
1997	970	1864	4877	5685	9715	8471	20789	17321	16795	18725	17134	30628	10810	16992	76827
1998	976	1877	4963	5788	9918	8634	21410	17633	17227	19158	17604	31496	11030	17354	79401
1999	983	1890	5026	5890	10125	8798	22041	17950	17659	19591	18081	32152	11253	17722	82018
2000	989	1903	5057	5993	10335	8966	22682	18273	18091	20024	18566	32659	11480	18096	84678
2001	995	1916	5090	6095	10547	9136	23333	18601	18523	20456	19059	33171	11710	18476	87380
2002	1001	1928	5149	6197	10764	9309	23994	18934	18955	20889	19559	33894	11944	18861	90126
2003	1008	1941	5234	6293	10983	9484	24666	19273	19387	21321	20066	34835	12181	19252	92914
2004	1015	1954	5323	6402	11205	9531	25347	19617	19817	21753	20581	35868	12421	19649	95745
2005	1023	1967	5410	6505	11427	9706	26038	19966	20251	22185	21104	36798	12665	20052	98618

APPENDIX F

TABLE F-1. SHIFTING OF GVW DISTRIBUTION FOR TRUCK TYPE 2D (1984)

WEIGHT INTERVAL	MID-POINT	FREQUENCY	PERCENT	CUMULATIVE PERCENT
0 - 4.99	2.5	6	00.20	00.20
5 - 9.99	7.5	369	12.47	12.67
10 - 14.99	12.5	1087	36.74	49.41
15 - 19.99	17.5	845	28.56	77.97
20 - 24.99	22.5	445	15.04	93.00
25 - 29.99	27.5	190	06.42	99.43
30 - 34.99	32.5	16	00.54	99.97
35 - 39.99	37.5	1	00.03	100.00

TABLE F-2. SHIFTING OF GVW DISTRIBUTION FOR TRUCK TYPE 3A (1984)

WEIGHT INTERVAL	MID-POINT	FREQUENCY	PERCENT	CUMULATIVE PERCENT
0 - 4.99	2.5	0	00.00	00.00
5 - 9.99	7.5	9	01.07	01.07
10 - 14.99	12.5	46	05.47	06.54
15 - 19.99	17.5	196	23.31	29.85
20 - 24.99	22.5	178	21.17	51.01
25 - 29.99	27.5	134	15.93	66.94
30 - 34.99	32.5	89	10.58	77.53
35 - 39.99	37.5	77	09.16	86.68
40 - 44.99	42.5	55	06.54	93.22
45 - 49.99	47.5	35	04.16	97.38
50 - 54.99	52.5	17	02.02	99.41
55 - 59.99	57.5	3	00.36	99.76
60 - 64.99	62.5	2	00.24	100.00

TABLE F-3. SHIFTING OF GVW DISTRIBUTION FOR TRUCK TYPE 2S2 (1984)

WEIGHT INTERVAL	MID-POINT	FREQUENCY	PERCENT	CUMULATIVE PERCENT
5 - 9.99	7.5	0	00.00	00.00
10 - 14.99	12.5	3	00.21	00.21
15 - 19.99	17.5	28	01.99	02.20
20 - 24.99	22.5	118	08.38	10.58
25 - 29.99	27.5	293	20.81	31.39
30 - 34.99	32.5	291	20.67	52.06
35 - 39.99	37.5	190	13.49	65.55
40 - 44.99	42.5	210	14.91	80.47
45 - 49.99	47.5	145	10.30	90.77
50 - 54.99	52.5	77	05.47	96.24
55 - 59.99	57.5	26	01.85	98.08
60 - 64.99	62.5	17	01.21	99.29
65 - 69.99	67.5	4	00.28	99.57
70 - 74.99	72.5	4	00.28	99.86
75 - 79.99	77.5	1	00.07	99.93
80 - 84.99	82.5	1	00.07	100.00

TABLE F-4. SHIFTING OF GVW DISTRIBUTION FOR TRUCK TYPE 3S1 (1984)

WEIGHT INTERVAL	MID-POINT	FREQUENCY	PERCENT	CUMULATIVE PERCENT
5 - 9.99	7.5	0	00.00	00.00
10 - 14.99	12.5	10	03.60	03.60
15 - 19.99	17.5	29	10.43	14.03
20 - 24.99	22.5	84	30.22	44.24
25 - 29.99	27.5	76	27.34	71.58
30 - 34.99	32.5	32	11.51	83.09
35 - 39.99	37.5	16	05.76	88.85
40 - 44.99	42.5	6	02.16	91.01
45 - 49.99	47.5	12	04.32	95.32
50 - 54.99	52.5	3	01.08	96.40
55 - 59.99	57.5	3	01.08	97.48
60 - 64.99	62.5	4	01.44	98.92
65 - 69.99	67.5	1	00.36	99.28
70 - 74.99	72.5	2	00.72	100.00

TABLE F-5. SHIFTING OF GVW DISTRIBUTION FOR TRUCK TYPE 3S2 (1984)

WEIGHT INTERVAL	MID-POINT	FREQUENCY	PERCENT	CUMULATIVE PERCENT
5 - 9.99	7.5	0	00.00	00.00
10 - 14.99	12.5	17	00.07	00.07
15 - 19.99	17.5	131	00.54	00.61
20 - 24.99	22.5	269	01.11	01.73
25 - 29.99	27.5	1453	06.02	07.75
30 - 34.99	32.5	3491	14.46	22.20
35 - 39.99	37.5	2376	09.84	32.05
40 - 44.99	42.5	1507	06.24	38.29
45 - 49.99	47.5	1299	05.38	43.67
50 - 54.99	52.5	1267	05.25	48.91
55 - 59.99	57.5	1311	05.43	54.34
60 - 64.99	62.5	1533	06.35	60.69
65 - 69.99	67.5	2407	09.97	70.66
70 - 74.99	72.5	3117	12.91	83.57
75 - 79.99	77.5	2432	10.07	93.65
80 - 84.99	82.5	1006	04.17	97.81
85 - 89.99	87.5	324	01.34	99.16
90 - 94.99	92.5	139	00.58	99.73
95 - 99.99	97.5	41	00.17	99.90
100 - 104.99	102.5	12	00.05	99.95
105 - 109.99	107.5	9	00.04	99.99
110 - 114.99	112.5	2	00.01	100.00
115 - 119.99	117.5	1	00.00	100.00

TABLE F-6. SHIFTING OF GVW DISTRIBUTION FOR TRUCK TYPE 2/3S12 (1984)

WEIGHT INTERVAL	MID-POINT	FREQUENCY	PERCENT	CUMULATIVE PERCENT
5 - 9.99	7.5	0	00.00	00.00
10 - 14.99	12.5	0	00.00	00.00
15 - 19.99	17.5	0	00.00	00.00
20 - 24.99	22.5	0	00.00	00.00
25 - 29.99	27.5	19	03.69	03.69
30 - 34.99	32.5	31	06.02	09.71
35 - 39.99	37.5	43	08.35	18.06
40 - 44.99	42.5	39	07.57	25.63
45 - 49.99	47.5	43	08.35	33.98
50 - 54.99	52.5	56	10.87	44.85
55 - 59.99	57.5	61	11.84	56.70
60 - 64.99	62.5	62	12.04	68.74
65 - 69.99	67.5	44	08.54	77.28
70 - 74.99	72.5	51	09.90	87.18
75 - 79.99	77.5	44	08.54	95.73
80 - 84.99	82.5	12	02.33	98.06
85 - 89.99	87.5	8	01.55	99.61
90 - 94.99	92.5	2	00.39	100.00

APPENDIX G

TABLE G-1. PROCEDURE TO OBTAIN THE NET TOTAL LOAD CARRIED BY TRUCK TYPE 3A AT THE DIFFERENT SELECTED STATIONS (1984)

MIDPOINT KIPS	EMPTY WEIGHT 3A'	NET WEIGHT	NET WEIGHT NUMBERS	3A/GENERAL 1984	PERCENTAGE OF 1984	STATION 2		STATION 3		STATION 6	
						NEW TOTAL	NET LOAD	NEW TOTAL	NET LOAD	NEW TOTAL	NET LOAD
2.5	16.35	-13.85	0	0	0	0	0	0	0	0	0
7.5	16.35	-8.85	0	9	1	0	0	0	0	0	0
12.5	16.35	-3.85	0	46	5	0	0	0	0	0	0
17.5	16.35	1.15	1.15	196	23	5	6	11	13	48	55
22.5	16.35	6.15	6.15	178	21	4	25	8	49	35	215
27.5	16.35	11.15	11.15	134	16	3	33	6	67	26	290
32.5	16.35	16.15	16.15	89	11	2	32	4	65	18	291
37.5	16.35	21.15	21.15	77	9	2	42	4	85	15	317
42.5	16.35	26.15	26.15	55	7	1	26	3	78	12	314
47.5	16.35	31.15	31.15	35	4	1	31	2	62	7	218
52.5	16.35	36.15	36.15	17	2	0	0	1	36	3	108
57.5	16.35	41.15	41.15	3	0	0	0	0	0	0	0
62.5	16.35	46.15	46.15	2	0	0	0	0	0	0	0
Total				841	100	18	195.7	39	454.85	164	1808.6

continued

TABLE G-1. CONTINUED

MIDPOINT KIPS	EMPTY WEIGHT 3A'	NET WEIGHT	NET WEIGHT NUMBERS	3A/GENERAL 1984	PERCENTAGE OF 1984	STATION 8 NEW TOTAL	STATION 8 NET LOAD	STATION 9 NEW TOTAL	STATION 9 NET LOAD	STATION 10 NEW TOTAL	STATION 10 NET LOAD
2.5	16.35	-13.85	0	0	0	0	0	0	0	0	0
7.5	16.35	-8.85	0	9	1	0	0	0	0	0	0
12.5	16.35	-3.85	0	46	5	0	0	0	0	0	0
17.5	16.35	1.15	1.15	196	23	32	37	39	45	34	39
22.5	16.35	6.15	6.15	178	21	23	141	29	178	25	154
27.5	16.35	11.15	11.15	134	16	18	201	22	245	19	212
32.5	16.35	16.15	16.15	89	11	12	194	15	242	13	210
37.5	16.35	21.15	21.15	77	9	10	212	12	254	11	233
42.5	16.35	26.15	26.15	55	7	8	209	10	262	8	209
47.5	16.35	31.15	31.15	35	4	4	125	5	156	5	156
52.5	16.35	36.15	36.15	17	2	2	72	3	108	2	72
57.5	16.35	41.15	41.15	3	0	0	0	0	0	0	0
62.5	16.35	46.15	46.15	2	0	0	0	0	0	0	0
Total				841	100	109	1190.35	135	1490.25	117	1284.55

continued

TABLE G-1. CONTINUED

MIDPOINT KIPS	EMPTY WEIGHT 3A'	NET WEIGHT	NET WEIGHT NUMBERS	3A/GENERAL 1984	PERCENTAGE OF 1984	STATION 12 NEW TOTAL	12 NET LOAD	13 NEW TOTAL	13 NET LOAD	14 NEW TOTAL	14 NET LOAD
2.5	16.35	-13.85	0	0	0	0	0	0	0	0	0
7.5	16.35	-8.85	0	9	1	0	0	0	0	0	0
12.5	16.35	-3.85	0	46	5	0	0	0	0	0	0
17.5	16.35	1.15	1.15	196	23	25	29	39	45	330	380
22.5	16.35	6.15	6.15	178	21	18	111	28	172	239	1470
27.5	16.35	11.15	11.15	134	16	14	156	22	245	182	2029
32.5	16.35	16.15	16.15	89	11	9	145	15	242	125	2019
37.5	16.35	21.15	21.15	77	9	8	169	12	254	102	2157
42.5	16.35	26.15	26.15	55	7	6	157	9	235	80	2092
47.5	16.35	31.15	31.15	35	4	3	93	5	156	45	1402
52.5	16.35	36.15	36.15	17	2	2	72	3	108	23	831
57.5	16.35	41.15	41.15	3	0	0	0	0	0	0	0
62.5	16.35	46.15	46.15	2	0	0	0	0	0	0	0
Total				841	100	85	932.75	133	1457.95	1126	12379.9

TABLE G-2. PROCEDURE TO OBTAIN THE NET TOTAL LOAD CARRIED BY TRUCK TYPE 2S1 AT THE DIFFERENT SELECTED STATIONS (1984)

MIDPOINT KIPS	EMPTY WEIGHT	NET WEIGHT	NET WEIGHT/NO'S	2S1/GENERAL YEAR 1984	PERCENTAGE OF 1984	NET WEIGHT	NEW TOTAL NO. 10	NET WEIGHT NO. 10
2.5	23.85	-21.35	0	0	0	0	0	0
7.5	23.85	-16.35	0	0	0	0	0	0
12.5	23.85	-11.35	0	16	3	0	4	0
17.5	23.85	-6.35	0	49	10	0	12	0
22.5	23.85	-1.35	0	84	17	0	20	0
27.5	23.85	3.65	17.74	134	27	68	32	568
32.5	23.85	8.65	22.74	105	21	25	25	571
37.5	23.85	13.65	27.74	57	12	14	14	378
42.5	23.85	18.65	32.74	35	7	8	8	274
47.5	23.85	23.65	37.74	8	2	2	2	72
52.5	23.85	28.65	42.74	1	0	0	0	10
57.5	23.85	33.65	47.74	1	0	0	0	11
62.5	23.85	38.65	52.74		0	0	0	0
Total				490	100		117.15	

TABLE G-3. PROCEDURE TO OBTAIN THE NET TOTAL LOAD CARRIED BY TRUCK TYPE 2S2 AT THE DIFFERENT SELECTED STATIONS (1984)

MIDPOINT KIPS	EMPTY WEIGHT (2S2)	NET WEIGHT	NET WEIGHT/ NUMBERS	2S2/GENERAL YEAR 1984	PERCENTAGE OF 1984	1		2		3	
						NEW TOTAL	NET LOAD	NEW TOTAL	NET LOAD	NEW TOTAL	NET LOAD
2.5	19.55	-17.05	0	0	0	0	0	0	0	0	0
7.5	19.55	-12.05	0	0	0	0	0	0	0	0	0
12.5	19.55	-7.05	0	0	0	0	0	0	0	0	0
17.5	19.55	-2.05	0	0	0	0	0	0	0	0	0
22.5	19.55	2.95	2.95	149	11	1	3	2	5	4	12
27.5	19.55	7.95	7.95	293	21	2	16	3	28	8	66
32.5	19.55	12.95	12.95	291	21	2	26	3	45	8	106
37.5	19.55	17.95	17.95	190	13	1	23	2	41	5	96
42.5	19.55	22.95	22.95	210	15	1	33	2	57	6	136
47.5	19.55	27.95	27.95	145	10	1	28	2	48	4	114
52.5	19.55	32.95	32.95	77	5	1	17	1	30	2	72
57.5	19.55	37.95	37.95	26	2	0	7	0	12	1	28
62.5	19.55	42.95	42.95	17	1	0	5	0	9	0	21
67.5	19.55	47.95	47.95	4	0	0	1	0	2	0	5
72.5	19.55	52.95	52.95	4	0	0	1	0	3	0	6
77.5	19.55	57.95	57.95	1	0	0	0	0	1	0	2
82.5	19.55	62.95	62.95	1	0	0	0	0	1	0	2
87.5	19.55	67.95	67.95	0	0	0	0	0	0	0	0
92.5	19.55	72.95	72.95	0	0	0	0	0	0	0	0
97.5	19.55	77.95	77.95	0	0	0	0	0	0	0	0
102.5	19.55	82.95	82.95	0	0	0	0	0	0	0	0
107.5	19.55	87.95	87.95	0	0	0	0	0	0	0	0
112.5	19.55	92.95	92.95	0	0	0	0	0	0	0	0
117.5	19.55	97.95	97.95	0	0	0	0	0	0	0	0
Total				1408	99.2897727	9.57153409	161.56996	16.640966	280.90379	39.4379	665.72187

continued

TABLE G-3. CONTINUED

MIDPOINT KIPS	EMPTY WEIGHT (2S2)	NET WEIGHT	NET WEIGHT/ NUMBERS	2S2/GENERAL YEAR 1984	PERCENTAGE OF 1984	10		11		13	
						NEW TOTAL	NET LOAD	NEW TOTAL	NET LOAD	NEW TOTAL	NET LOAD
2.5	19.55	-17.05	0	0	0	0	0	0	0	0	0
7.5	19.55	-12.05	0	0	0	0	0	0	0	0	0
12.5	19.55	-7.05	0	0	0	0	0	0	0	0	0
17.5	19.55	-2.05	0	0	0	0	0	0	0	0	0
22.5	19.55	2.95	2.95	149	11	12	37	22	65	13	39
27.5	19.55	7.95	7.95	293	21	24	194	43	345	26	207
32.5	19.55	12.95	12.95	291	21	24	314	43	558	26	335
37.5	19.55	17.95	17.95	190	13	16	284	28	505	17	304
42.5	19.55	22.95	22.95	210	15	17	401	31	714	19	429
47.5	19.55	27.95	27.95	145	10	12	337	21	600	13	361
52.5	19.55	32.95	32.95	77	5	6	211	11	376	7	226
57.5	19.55	37.95	37.95	26	2	2	82	4	146	2	88
62.5	19.55	42.95	42.95	17	1	1	61	3	108	2	65
67.5	19.55	47.95	47.95	4	0	0	16	1	28	0	17
72.5	19.55	52.95	52.95	4	0	0	18	1	31	0	19
77.5	19.55	57.95	57.95	1	0	0	5	0	9	0	5
82.5	19.55	62.95	62.95	1	0	0	5	0	9	0	6
87.5	19.55	67.95	67.95	0	0	0	0	0	0	0	0
92.5	19.55	72.95	72.95	0	0	0	0	0	0	0	0
97.5	19.55	77.95	77.95	0	0	0	0	0	0	0	0
102.5	19.55	82.95	82.95	0	0	0	0	0	0	0	0
107.5	19.55	87.95	87.95	0	0	0	0	0	0	0	0
112.5	19.55	92.95	92.95	0	0	0	0	0	0	0	0
117.5	19.55	97.95	97.95	0	0	0	0	0	0	0	0
Total				1408	99.2897727	116.317969	1963.47727	207.10854	3496.0454	124.4399	2100.5771

continued

TABLE G-3. CONTINUED

MIDPOINT KIPS	EMPTY WEIGHT (2S2)	NET WEIGHT	NET WEIGHT/ NUMBERS	2S2/GENERAL YEAR 1984	PERCENTAGE OF 1984	7		8		9	
						NEW TOTAL	NET LOAD	NEW TOTAL	NET LOAD	NEW TOTAL	NET LOAD
2.5	19.55	-17.05	0	0	0	0	0	0	0	0	0
7.5	19.55	-12.05	0	0	0	0	0	0	0	0	0
12.5	19.55	-7.05	0	0	0	0	0	0	0	0	0
17.5	19.55	-2.05	0	0	0	0	0	0	0	0	0
22.5	19.55	2.95	2.95	149	11	16	46	12	34	14	42
27.5	19.55	7.95	7.95	293	21	31	245	23	181	28	225
32.5	19.55	12.95	12.95	291	21	31	397	23	293	28	364
37.5	19.55	17.95	17.95	190	13	20	359	15	265	18	330
42.5	19.55	22.95	22.95	210	15	22	507	16	375	20	466
47.5	19.55	27.95	27.95	145	10	15	427	11	315	14	392
52.5	19.55	32.95	32.95	77	5	8	267	6	197	7	245
57.5	19.55	37.95	37.95	26	2	3	104	2	77	3	95
62.5	19.55	42.95	42.95	17	1	2	77	1	57	2	71
67.5	19.55	47.95	47.95	4	0	0	20	0	15	0	19
72.5	19.55	52.95	52.95	4	0	0	22	0	16	0	20
77.5	19.55	57.95	57.95	1	0	0	6	0	5	0	6
82.5	19.55	62.95	62.95	1	0	0	7	0	5	0	6
87.5	19.55	67.95	67.95	0	0	0	0	0	0	0	0
92.5	19.55	72.95	72.95	0	0	0	0	0	0	0	0
97.5	19.55	77.95	77.95	0	0	0	0	0	0	0	0
102.5	19.55	82.95	82.95	0	0	0	0	0	0	0	0
107.5	19.55	87.95	87.95	0	0	0	0	0	0	0	0
112.5	19.55	92.95	92.95	0	0	0	0	0	0	0	0
117.5	19.55	97.95	97.95	0	0	0	0	0	0	0	0
Total				1408	99.2897727	147.17723	2484.38954	108.81166	1836.7689	135.1334	2281.0863

TABLE G-4. PROCEDURE TO OBTAIN THE NET TOTAL LOAD CARRIED BY TRUCK TYPE 3S2 AT THE DIFFERENT SELECTED STATIONS (1984)

MIDPOINT KIPS	EMPTY WT. (3S2)	NET WEIGHT	NET WEIGHT/ NUMBERS	3S2/GENERAL YR. 1984	PERCENTAGE OF 1984	1		2		3	
						NEW TOTAL	NET LOAD	NEW TOTAL	NET LOAD	NEW TOTAL	NET LOAD
2.5	26.75	-24.25	0	0	0	0	0	0	0	0	0
7.5	26.75	-19.25	0	0	0	0	0	0	0	0	0
12.5	26.75	-14.25	0	0	0	0	0	0	0	0	0
17.5	26.75	-9.25	0	0	0	0	0	0	0	0	0
22.5	26.75	-4.25	0	0	0	0	0	0	0	0	0
27.5	26.75	0.75	0.75	1870	8	11	8	16	12	22	16
32.5	26.75	5.75	5.75	3491	14	21	120	29	167	40	231
37.5	26.75	10.75	10.75	2376	10	14	153	20	213	27	294
42.5	26.75	15.75	15.75	1507	6	9	142	13	198	17	273
47.5	26.75	20.75	20.75	1299	5	8	161	11	225	15	310
52.5	26.75	25.75	25.75	1267	5	8	195	11	272	15	376
57.5	26.75	30.75	30.75	1311	5	8	241	11	336	15	464
62.5	26.75	35.75	35.75	1533	6	9	328	13	457	18	631
67.5	26.75	40.75	40.75	2407	10	14	587	20	817	28	1130
72.5	26.75	45.75	45.75	3117	13	19	854	26	1188	36	1642
77.5	26.75	50.75	50.75	2432	10	15	739	20	1028	28	1421
82.5	26.75	55.75	55.75	1006	4	6	336	8	467	12	646
87.5	26.75	60.75	60.75	324	1	2	118	3	164	4	227
92.5	26.75	65.75	65.75	139	1	1	55	1	76	2	105
97.5	26.75	70.75	70.75	41	0	0	17	0	24	0	33
102.5	26.75	75.75	75.75	12	0	0	5	0	8	0	10
107.5	26.75	80.75	80.75	9	0	0	4	0	6	0	8
112.5	26.75	85.75	85.75	2	0	0	1	0	1	0	2
117.5	26.75	90.75	90.75	1	0	0	1	0	1	0	1
Total				24144	60.694168	144.6	4068.1806	201.12	5658.3159	278.04	7822.3855

continued

TABLE G-4. CONTINUED

MIDPOINT KIPS	EMPTY WT. (3S2)	NET WEIGHT	NET WEIGHT/ NUMBERS	3S2/GENERAL YR. 1984	PERCENTAGE OF 1984	3A		4		5	
						NEW TOTAL	NET LOAD	NEW TOTAL	NET LOAD	NEW TOTAL	NET LOAD
2.5	26.75	-24.25	0	0	0	0	0	0	0	0	0
7.5	26.75	-19.25	0	0	0	0	0	0	0	0	0
12.5	26.75	-14.25	0	0	0	0	0	0	0	0	0
17.5	26.75	-9.25	0	0	0	0	0	0	0	0	0
22.5	26.75	-4.25	0	0	0	0	0	0	0	0	0
27.5	26.75	0.75	0.75	1870	8	11	8	13	10	11	8
32.5	26.75	5.75	5.75	3491	14	20	116	24	137	20	114
37.5	26.75	10.75	10.75	2376	10	14	148	16	174	14	145
42.5	26.75	15.75	15.75	1507	6	9	137	10	162	9	135
47.5	26.75	20.75	20.75	1299	5	8	156	9	184	7	153
52.5	26.75	25.75	25.75	1267	5	7	189	9	223	7	185
57.5	26.75	30.75	30.75	1311	5	8	233	9	275	7	229
62.5	26.75	35.75	35.75	1533	6	9	317	10	374	9	312
67.5	26.75	40.75	40.75	2407	10	14	567	16	670	14	558
72.5	26.75	45.75	45.75	3117	13	18	824	21	974	18	811
77.5	26.75	50.75	50.75	2432	10	14	713	17	843	14	702
82.5	26.75	55.75	55.75	1006	4	6	324	7	383	6	319
87.5	26.75	60.75	60.75	324	1	2	114	2	134	2	112
92.5	26.75	65.75	65.75	139	1	1	53	1	62	1	52
97.5	26.75	70.75	70.75	41	0	0	17	0	20	0	16
102.5	26.75	75.75	75.75	12	0	0	5	0	6	0	5
107.5	26.75	80.75	80.75	9	0	0	4	0	5	0	4
112.5	26.75	85.75	85.75	2	0	0	1	0	1	0	1
117.5	26.75	90.75	90.75	1	0	0	1	0	1	0	1
Total				24144	60.694168	139.53	3925.5411	164.92	4639.8641	137.26	3861.6768

continued

TABLE G-4. CONTINUED

MIDPOINT KIPS	EMPTY WT. (3S2)	NET WEIGHT	NET WEIGHT/ NUMBERS	3S2/GENERAL YR. 1984	PERCENTAGE OF 1984	6		7		8	
						NEW TOTAL	NET LOAD	NEW TOTAL	NET LOAD	NEW TOTAL	NET LOAD
2.5	26.75	-24.25	0	0	0	0	0	0	0	0	0
7.5	26.75	-19.25	0	0	0	0	0	0	0	0	0
12.5	26.75	-14.25	0	0	0	0	0	0	0	0	0
17.5	26.75	-9.25	0	0	0	0	0	0	0	0	0
22.5	26.75	-4.25	0	0	0	0	0	0	0	0	0
27.5	26.75	0.75	0.75	1870	8	26	19	184	138	42	32
32.5	26.75	5.75	5.75	3491	14	48	274	343	1972	79	456
37.5	26.75	10.75	10.75	2376	10	32	349	233	2509	54	580
42.5	26.75	15.75	15.75	1507	6	21	325	148	2332	34	539
47.5	26.75	20.75	20.75	1299	5	18	369	128	2648	29	612
52.5	26.75	25.75	25.75	1267	5	17	446	124	3205	29	740
57.5	26.75	30.75	30.75	1311	5	18	551	129	3960	30	915
62.5	26.75	35.75	35.75	1533	6	21	749	151	5384	35	1244
67.5	26.75	40.75	40.75	2407	10	33	1341	236	9635	55	2226
72.5	26.75	45.75	45.75	3117	13	43	1950	306	14008	71	3236
77.5	26.75	50.75	50.75	2432	10	33	1688	239	12124	55	2801
82.5	26.75	55.75	55.75	1006	4	14	767	99	5509	23	1273
87.5	26.75	60.75	60.75	324	1	4	269	32	1933	7	447
92.5	26.75	65.75	65.75	139	1	2	125	14	898	3	207
97.5	26.75	70.75	70.75	41	0	1	40	4	285	1	66
102.5	26.75	75.75	75.75	12	0	0	12	1	89	0	21
107.5	26.75	80.75	80.75	9	0	0	10	1	71	0	16
112.5	26.75	85.75	85.75	2	0	0	2	0	17	0	4
117.5	26.75	90.75	90.75	1	0	0	1	0	9	0	2
Total				24144	60.694168	330.14	9288.1684	2371.68	66724.914	547.95	15416.041

continued

TABLE G-4. CONTINUED

MIDPOINT KIPS	EMPTY WT. (3S2)	NET WEIGHT	NET WEIGHT/ NUMBERS	3S2/GENERAL YR. 1984	PERCENTAGE OF 1984	9		10		11	
						NEW TOTAL	NET LOAD	NEW TOTAL	NET LOAD	NEW TOTAL	NET LOAD
2.5	26.75	-24.25	0	0	0	0	0	0	0	0	0
7.5	26.75	-19.25	0	0	0	0	0	0	0	0	0
12.5	26.75	-14.25	0	0	0	0	0	0	0	0	0
17.5	26.75	-9.25	0	0	0	0	0	0	0	0	0
22.5	26.75	-4.25	0	0	0	0	0	0	0	0	0
27.5	26.75	0.75	0.75	1870	8	137	103	281	211	145	109
32.5	26.75	5.75	5.75	3491	14	256	1471	525	3019	271	1561
37.5	26.75	10.75	10.75	2376	10	174	1872	357	3842	185	1986
42.5	26.75	15.75	15.75	1507	6	110	1739	227	3570	117	1846
47.5	26.75	20.75	20.75	1299	5	95	1975	195	4054	101	2096
52.5	26.75	25.75	25.75	1267	5	93	2391	191	4907	99	2537
57.5	26.75	30.75	30.75	1311	5	96	2954	197	6064	102	3135
62.5	26.75	35.75	35.75	1533	6	112	4016	231	8244	119	4261
67.5	26.75	40.75	40.75	2407	10	176	7188	362	14754	187	7627
72.5	26.75	45.75	45.75	3117	13	228	10450	469	21450	242	11088
77.5	26.75	50.75	50.75	2432	10	178	9045	366	18565	189	9597
82.5	26.75	55.75	55.75	1006	4	74	4110	151	8436	78	4361
87.5	26.75	60.75	60.75	324	1	24	1442	49	2961	25	1530
92.5	26.75	65.75	65.75	139	1	10	670	21	1375	11	711
97.5	26.75	70.75	70.75	41	0	3	213	6	436	3	226
102.5	26.75	75.75	75.75	12	0	1	67	2	137	1	71
107.5	26.75	80.75	80.75	9	0	1	53	1	109	1	57
112.5	26.75	85.75	85.75	2	0	0	13	0	26	0	13
117.5	26.75	90.75	90.75	1	0	0	7	0	14	0	7
Total				24144	60.694168	1769.3	49777.538	3631.65	102172.95	1877.31	52816.294

continued

TABLE G-4. CONTINUED

MIDPOINT KIPS	EMPTY WT. (3S2)	NET WEIGHT	NET WEIGHT/ NUMBERS	3S2/GENERAL YR. 1984	PERCENTAGE OF 1984	12		13		14	
						NEW TOTAL	NET LOAD	NEW TOTAL	NET LOAD	NEW TOTAL	NET LOAD
2.5	26.75	-24.25	0	0	0	0	0	0	0	0	0
7.5	26.75	-19.25	0	0	0	0	0	0	0	0	0
12.5	26.75	-14.25	0	0	0	0	0	0	0	0	0
17.5	26.75	-9.25	0	0	0	0	0	0	0	0	0
22.5	26.75	-4.25	0	0	0	0	0	0	0	0	0
27.5	26.75	0.75	0.75	1870	8	47	35	214	160	308	231
32.5	26.75	5.75	5.75	3491	14	87	501	399	2292	575	3308
37.5	26.75	10.75	10.75	2376	10	59	638	271	2917	392	4209
42.5	26.75	15.75	15.75	1507	6	38	592	172	2711	248	3911
47.5	26.75	20.75	20.75	1299	5	32	673	148	3078	214	4442
52.5	26.75	25.75	25.75	1267	5	32	814	145	3726	209	5376
57.5	26.75	30.75	30.75	1311	5	33	1006	150	4604	216	6643
62.5	26.75	35.75	35.75	1533	6	38	1368	175	6259	253	9031
67.5	26.75	40.75	40.75	2407	10	60	2448	275	11201	397	16163
72.5	26.75	45.75	45.75	3117	13	78	3559	356	16285	514	23499
77.5	26.75	50.75	50.75	2432	10	61	3081	278	14095	401	20339
82.5	26.75	55.75	55.75	1006	4	25	1400	115	6405	166	9242
87.5	26.75	60.75	60.75	324	1	8	491	37	2248	53	3244
92.5	26.75	65.75	65.75	139	1	3	228	16	1044	23	1506
97.5	26.75	70.75	70.75	41	0	1	72	5	331	7	478
102.5	26.75	75.75	75.75	12	0	0	23	1	104	2	150
107.5	26.75	80.75	80.75	9	0	0	18	1	83	1	120
112.5	26.75	85.75	85.75	2	0	0	4	0	20	0	28
117.5	26.75	90.75	90.75	1	0	0	2	0	10	0	15
Total				24144	60.694168	602.63	16954.41	2757.26	77572.833	3978.66	111935.74

TABLE G-5. PROCEDURE TO OBTAIN THE NET TOTAL LOAD CARRIED BY TRUCK TYPE 2/3S12 AT THE DIFFERENT SELECTED STATIONS (1984)

MIDPOINT KIPS	EMPTY WEIGHT 2/3S12	NET WEIGHT	NO. 3S12'S OBSERVED 1984	PERCENTAGE 1984	STATION 10		STATION 11		STATION 13		STATION 14
					NEW TOTAL	NET WEIGHT	NEW TOTAL	NET WEIGHT	NEW TOTAL	NET WEIGHT	NEW TOTAL
2.5	31	0	0	0	0	0	0	0	0	0	0
7.5	31	0	0	0	0	0	0	0	0	0	0
12.5	31	0	0	0	0	0	0	0	0	0	0
17.5	31	0	0	0	0	0	0	0	0	0	0
22.5	31	0	0	0	0	0	0	0	0	0	0
27.5	31	0	0	0	0	0	0	0	0	0	0
32.5	31	1.5	10	8	9	13	16	24	9	14	0
37.5	31	6.5	10	8	9	57	16	102	9	61	0
42.5	31	11.5	11	8	10	111	17	198	10	119	0
47.5	31	16.5	8	6	7	116	13	207	8	124	0
52.5	31	21.5	11	8	10	208	17	371	10	223	0
57.5	31	26.5	14	11	12	327	22	582	13	350	0
62.5	31	31.5	18	14	16	499	28	889	17	534	0
67.5	31	36.5	15	11	13	482	24	859	14	516	0
72.5	31	41.5	13	10	11	475	20	846	12	508	0
77.5	31	46.5	16	12	14	655	25	1167	15	701	0
82.5	31	51.5	3	2	3	136	5	242	3	146	0
87.5	31	56.5	4	3	4	199	6	354	4	213	0
92.5	31	61.5	0	0	0	0	0	0	0	0	0
97.5	31	66.5	0	0	0	0	0	0	0	0	0
102.5	31	71.5	0	0	0	0	0	0	0	0	0
107.5	31	76.5	0	0	0	0	0	0	0	0	0
112.5	31	81.5	0	0	0	0	0	0	0	0	0
117.5	31	86.5	0	0	0	0	0	0	0	0	0
Total			133	61.654135338	117.15	3280.6404	208.59	5841.3042	125.33	3509.7112	0

TABLE G-6. PROCEDURE TO OBTAIN THE NET LOAD CARRIED BY TRUCK TYPE 3A (IN TERMS OF X), (1984)

MIDPOINT KIPS	TRUCK WT. (3A)	NET WEIGHT	NUMBER OF TRUCKS (percent of X)	NET TOTAL WEIGHT (in terms of X)
2.5	16.35	-13.85	0	0
7.5	16.35	-8.85	0	0
12.5	16.35	-3.85	0	0
17.5	16.35	1.15	0.55	0.6325
22.5	16.35	6.15	0.14	0.861
27.5	16.35	11.15	0.14	1.561
32.5	16.35	16.15	0.06	0.969
37.5	16.35	21.15	0.05	1.0575
42.5	16.35	26.15	0.03	0.7845
47.5	16.35	31.15	0	0
52.5	16.35	36.15	0.01	0.3615
57.5	16.35	41.15	0.02	0.823
62.5	16.35	46.15	0	0
67.5	16.35	51.15	0	0
72.5	16.35	56.15	0	0
77.5	16.35	61.15	0	0
82.5	16.35	66.15	0	0
87.5	16.35	71.15	0	0
Total			1	7.05

TABLE G-7. PROCEDURE TO OBTAIN THE NET LOAD CARRIED BY TRUCK TYPE 2S1 (IN TERMS OF X), (1984)

MIDPOINT KIPS	TRUCK WEIGHT (3A)	NET WEIGHT	NUMBER OF TRUCKS (percent of X)	NET TOTAL WEIGHT (in terms of X)
2.5	23.85	-21.35	0	0
7.5	23.85	-16.35	0	0
12.5	23.85	-11.35	0	0
17.5	23.85	-6.35	0	0
22.5	23.85	-1.35	0	0
27.5	23.85	3.65	0.66	2.409
32.5	23.85	8.65	0.22	1.903
37.5	23.85	13.65	0.08	1.092
42.5	23.85	18.65	0.03	0.5595
47.5	23.85	23.65	0.02	0.473
52.5	23.85	28.65	0	0
57.5	23.85	33.65	0	0
62.5	23.85	38.65	0	0
67.5	23.85	43.65	0	0
72.5	23.85	48.65	0	0
77.5	23.85	53.65	0	0
82.5	23.85	58.65	0	0
87.5	23.85	63.65	0	0
TOTAL			1.01	6.4365

TABLE G-8. PROCEDURE TO OBTAIN THE NET LOAD CARRIED BY TRUCK TYPE 2S2 (IN TERMS OF X), (1984)

MIDPOINT KIPS	TRUCK WEIGHT (2S2)	NET WEIGHT	NUMBER OF TRUCKS (percent of X)	NET TOTAL WEIGHT (in terms of X)
2.5	19.55	-17.05	0	0
7.5	19.55	-12.05	0	0
12.5	19.55	-7.05	0	0
17.5	19.55	-2.05	0	0
22.5	19.55	2.95	0.33	0.9735
27.5	19.55	7.95	0.13	1.0335
32.5	19.55	12.95	0.11	1.4245
37.5	19.55	17.95	0.11	1.9745
42.5	19.55	22.95	0.1	2.295
47.5	19.55	27.95	0.1	2.795
52.5	19.55	32.95	0.09	2.9655
57.5	19.55	37.95	0.03	1.1385
62.5	19.55	42.95	0.01	0.4295
67.5	19.55	47.95	0	0
72.5	19.55	52.95	0	0
77.5	19.55	57.95	0	0
82.5	19.55	62.95	0	0
87.5	19.55	67.95	0	0
Total			1.01	15.0295

TABLE G-9. PROCEDURE TO OBTAIN THE NET LOAD CARRIED BY TRUCK TYPE 3S2 (IN TERMS OF X), (1984)

MIDPOINT KIPS	TRUCK WEIGHT (3S2)	NET WEIGHT	NUMBER OF TRUCKS (percent of X)	NET TOTAL WEIGHT (in terms of X)
2.5	26.75	-24.25	0	0
7.5	26.75	-19.25	0	0
12.5	26.75	-14.25	0	0
17.5	26.75	-9.25	0	0
22.5	26.75	-4.25	0	0
27.5	26.75	0.75	0.33	0.2475
32.5	26.75	5.75	0.09	0.5175
37.5	26.75	10.75	0.04	0.43
42.5	26.75	15.75	0.04	0.63
47.5	26.75	20.75	0.06	1.245
52.5	26.75	25.75	0.07	1.8025
57.5	26.75	30.75	0.13	3.9975
62.5	26.75	35.75	0.12	4.29
67.5	26.75	40.75	0.06	2.445
72.5	26.75	45.75	0.03	1.3725
77.5	26.75	50.75	0.01	0.5075
82.5	26.75	55.75	0.01	0.5575
87.5	26.75	60.75	0.01	0.6075
TOTAL			1	18.65

TABLE G-10. PROCEDURE TO OBTAIN THE NET LOAD CARRIED BY TRUCK TYPE 2/3S12 (IN TERMS OF X), (1984)

MIDPOINT KIPS	TRUCK WEIGHT (2/3S12)	NET WEIGHT	NUMBER OF TRUCKS (percent of X)	NET TOTAL WEIGHT (in terms of X)
2.5	31	-28.5	0	0
7.5	31	-23.5	0	0
12.5	31	-18.5	0	0
17.5	31	-13.5	0	0
22.5	31	-8.5	0	0
27.5	31	-3.5	0	0
32.5	31	1.5	0.21	0.315
37.5	31	6.5	0.07	0.455
42.5	31	11.5	0.05	0.575
47.5	31	16.5	0.13	2.145
52.5	31	21.5	0.14	3.01
57.5	31	26.5	0.14	3.71
62.5	31	31.5	0.13	4.095
67.5	31	36.5	0.07	2.555
72.5	31	41.5	0.03	1.245
77.5	31	46.5	0.01	0.465
82.5	31	51.5	0.01	0.515
87.5	31	56.5	0	0
	0	0	0.01	0
TOTAL			1	19.085

APPENDIX H

TABLE H-1. PROCEDURE TO OBTAIN THE PERCENTAGE DIFFERENCE IN THE NUMBER OF TRUCKS CARRYING THE SAME GIVEN LOAD, BETWEEN THE YEARS 1974 AND 1984 FOR THE TRUCK TYPE 2D

STATION NUMBER YEAR TRUCK TYPE	NET TOTAL LOAD 1984 2D	TOTAL NUMBER TRUCKS 1984 2D	NET TOTAL LOAD 1974 2D	TOTAL NUMBER TRUCKS 1974 2D	DIFFERENCE IN TOTAL TRUCKS (%) COLUMNS((E - C) / C) 2D
1	325.93	48	5.6126X	58	20.83
2	564.67	84	5.6126X	101	20.24
3	1340.5	199	5.6126X	239	20.10
3A'	31366	4646	5.6126X	5588	20.28
4	1672.3	247	5.6126X	298	20.65
5	1390.8	206	5.6126X	248	20.39
6	4457.2	660	5.6126X	794	20.30
7	4003.2	593	5.6126X	713	20.24
8	2221.2	329	5.6126X	396	20.36
9	3675.6	544	5.6126X	655	20.40
10	3162.3	469	5.6126X	563	20.04
11	5635.2	834	5.6126X	1004	20.38
12	1744.8	258	5.6126X	311	20.54
13	2539.12	376	5.6126X	452	20.21
14	10920.1	1705	5.6126X	1946	14.13

TABLE H-2. PROCEDURE TO OBTAIN THE PERCENTAGE DIFFERENCE IN THE NUMBER OF TRUCKS CARRYING THE SAME GIVEN LOAD, BETWEEN THE YEARS 1974 AND 1984 FOR THE TRUCK TYPE 3A

STATION NUMBER YEAR TRUCK TYPE	NET TOTAL LOAD 1984 3A'	TOTAL NUMBER TRUCKS 1984 3A'	NET TOTAL LOAD 1974 3A'	TOTAL NUMBER TRUCKS 1974 3A'	DIFFERENCE IN TOTAL TRUCKS (%) COLUMNS((E - C) / C) 3A'
1	-	-	-	-	-
2	195.7	17	10.233X	19	11.76
3	454.85	39	10.233X	45	15.38
3A'	-	-	-	-	-
4	-	-	-	-	-
5	-	-	-	-	-
6	1808.6	164	10.233X	177	7.93
7	-	-	-	-	-
8	1190.35	109	10.233X	116	6.42
9	1490.25	135	10.233X	146	8.15
10	1284.55	117	10.233X	126	7.69
11	-	-	-	-	-
12	932.75	85	10.233X	91	7.06
13	1457.95	133	10.233X	143	7.52
14	12379.9	1126	10.233X	1211	7.55

TABLE H-3. PROCEDURE TO OBTAIN THE PERCENTAGE DIFFERENCE IN THE NUMBER OF TRUCKS CARRYING THE SAME GIVEN LOAD, BETWEEN THE YEARS 1974 AND 1984 FOR THE TRUCK TYPE 2S1

STATION NUMBER YEAR TRUCK TYPE	NET TOTAL LOAD 1984 2S1	TOTAL NUMBER TRUCKS 1984 2S1	NET TOTAL LOAD 1974 2S1	TOTAL NUMBER TRUCKS 1974 2S1	DIFFERENCE IN TOTAL TRUCKS (%) COLUMNS((E - C) / C) 2S1
1	-	-	-	-	-
2	-	-	-	-	-
3	-	-	-	-	-
3A'	-	-	-	-	-
4	-	-	-	-	-
5	-	-	-	-	-
6	-	-	-	-	-
7	-	-	-	-	-
8	-	-	-	-	-
9	-	-	-	-	-
10	1885	117	7.4X	255	117.95
11	-	-	-	-	-
12	-	-	-	-	-
13	-	-	-	-	-
14	-	-	-	-	-

TABLE H-4. PROCEDURE TO OBTAIN THE PERCENTAGE DIFFERENCE IN THE NUMBER OF TRUCKS CARRYING THE SAME GIVEN LOAD, BETWEEN THE YEARS 1974 AND 1984 FOR THE TRUCK TYPE 2S2

STATION NUMBER YEAR TRUCK TYPE	NET TOTAL LOAD 1984 2S2	TOTAL NUMBER TRUCKS 1984 2S2	NET TOTAL LOAD 1974 2S2	TOTAL NUMBER TRUCKS 1974 2S2	DIFFERENCE IN TOTAL TRUCKS (%) COLUMNS((E - C) / C) 2S2
1	161.57	10	13.10X	12	20.00
2	280.904	17	13.10X	21	23.53
3	665.72	40	13.10X	51	27.50
3A'	-	-	-	-	-
4	-	-	-	-	-
5	-	-	-	-	-
6	-	-	-	-	-
7	2484	148	13.10X	190	28.38
8	1837	110	13.10X	140	27.27
9	2281	136	13.10X	174	27.94
10	1964	117	13.10X	150	28.21
11	3496	209	13.10X	267	27.75
12	-	-	-	-	-
13	2101	125	13.10X	160	28.00
14	-	-	-	-	-

TABLE H-5. PROCEDURE TO OBTAIN THE PERCENTAGE DIFFERENCE IN THE NUMBER OF TRUCKS CARRYING THE SAME GIVEN LOAD, BETWEEN THE YEARS 1974 AND 1984 FOR THE TRUCK TYPE 3S2

STATION NUMBER YEAR TRUCK TYPE	NET TOTAL LOAD 1984 2S2	TOTAL NUMBER TRUCKS 1984 2S2	NET TOTAL LOAD 1974 2S2	TOTAL NUMBER TRUCKS 1974 2S2	DIFFERENCE IN TOTAL TRUCKS (%) COLUMNS((E - C) / C) 2S2
1	4068	145	18.2425X	223	53.79
2	5658	201	18.2425X	310	54.23
3	7822	278	18.2425X	429	54.32
3A'	3926	140	18.2425X	215	53.57
4	4640	165	18.2425X	254	53.94
5	3862	137	18.2425X	212	54.74
6	9288	330	18.2425X	509	54.24
7	66725	2372	18.2425X	3658	54.22
8	15146	548	18.2425X	845	54.20
9	49778	1769	18.2425X	2729	54.27
10	102173	3632	18.2425X	5601	54.21
11	52816	1877	18.2425X	2895	54.24
12	16954	603	18.2425X	929	54.06
13	77573	2757	18.2425X	4252	54.23
14	111936	3979	18.2425X	6136	54.21

TABLE H-6. PROCEDURE TO OBTAIN THE PERCENTAGE DIFFERENCE IN THE NUMBER OF TRUCKS CARRYING THE SAME GIVEN LOAD, BETWEEN THE YEARS 1974 AND 1984 FOR THE TRUCK TYPE 23S12

STATION NUMBER YEAR TRUCK TYPE	NET TOTAL LOAD 1984 23S12	TOTAL NUMBER TRUCKS 1984 23S12	NET TOTAL LOAD 1974 23S12	TOTAL NUMBER TRUCKS 1974 23S12	DIFFERENCE IN TOTAL TRUCKS (%) COLUMNS((E - C) / C) 23S12
1	-	-	-	-	-
2	-	-	-	-	-
3	-	-	-	-	-
3A'	-	-	-	-	-
4	-	-	-	-	-
5	-	-	-	-	-
6	-	-	-	-	-
7	-	-	-	-	-
8	-	-	-	-	-
9	-	-	-	-	-
10	3281	117	23.5625X	139	18.80
11	5841	209	23.5625X	248	18.66
12	-	-	-	-	-
13	3510	125	23.5625X	149	19.20
14	-	-	-	-	-

TABLE H-7. PERCENTAGE DIFFERENCE IN THE NUMBER OF TRUCKS BY CLASS REQUIRED TO CARRY THE SAME NET LOAD UNDER LEGAL LOAD LIMITS OF 1974 VERSUS 1984

COLUMN	A	B	C	D	E	F
YEAR	1984	1974	(C-B)/B*100	1984	1974	(F-E)/E*100
TRUCK TYPE	2D	2D	2D	3A'	3A'	3A'
STATION NUMBER	NUMBER TRUCKS	NUMBER TRUCKS	% DIFFERENCE	NUMBER TRUCKS	NUMBER TRUCKS	% DIFFERENCE
1	48	58	20.83	-	-	-
2	84	101	20.24	17	19	11.76
3	199	239	20.10	39	45	15.38
3A'	4646	5588	20.28	-	-	-
4	247	298	20.65	-	-	-
5	206	248	20.39	-	-	-
6	660	794	20.30	164	177	7.93
7	593	713	20.24	-	-	-
8	329	396	20.36	109	116	6.42
9	544	655	20.40	135	146	8.15
10	469	563	20.04	117	126	7.69
11	834	1004	20.38	-	-	-
12	258	311	20.54	85	91	7.06
13	376	452	20.21	133	143	7.52
14	1705	1946	14.13	1126	1211	7.55

continued

TABLE H-7. CONTINUED

COLUMN	G	H	I	J	K	L
YEAR	1984	1974	(I-H)/H*100	1984	1974	(K-J)/J*100
TRUCK TYPE	2S1	2S1	2S1	2S2	2S2	2S2
STATION NUMBER	NUMBER TRUCKS	NUMBER TRUCKS	% DIFFERENCE	NUMBER TRUCKS	NUMER TRUCKS	% DIFFERENCE
1	-	-	-	10	12	20.00
2	-	-	-	17	21	23.53
3	-	-	-	40	51	27.50
3A'	-	-	-	-	-	-
4	-	-	-	-	-	-
5	-	-	-	-	-	-
6	-	-	-	-	-	-
7	-	-	-	148	190	28.38
8	-	-	-	110	140	27.27
9	-	-	-	136	174	27.94
10	117	255	117.95	117	150	28.21
11	-	-	-	209	267	27.75
12	-	-	-	-	-	-
13	-	-	-	125	160	28.00
14	-	-	-	-	-	-

continued

TABLE H-7. CONTINUED

COLUMN	M	N	O	P	Q	R
YEAR	1984	1974	(O-N)/N*100	1984	1974	(R-Q)/Q*100
TRUCK TYPE	3S2	3S2	3S2	2/3S12	2/3S12	2/3S12
STATION NUMBER	NUMBER TRUCKS	NUMBER TRUCKS	ERCENT DIFFERENC	NUMBER TRUCKS	NUMBER TRUCKS	% DIFFERENCE
1	145	223	53.79	-	-	-
2	201	310	54.23	-	-	-
3	278	429	54.32	-	-	-
3A'	140	215	53.57	-	-	-
4	165	254	53.94	-	-	-
5	137	212	54.74	-	-	-
6	330	509	54.24	-	-	-
7	2372	3658	54.22	-	-	-
8	548	845	54.20	-	-	-
9	1769	2729	54.27	-	-	-
10	3632	5601	54.21	117	139	18.80
11	1877	2895	54.24	209	248	18.66
12	603	929	54.06	-	-	-
13	2757	4252	54.23	125	149	19.20
14	3979	6136	54.21	-	-	-