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16. Abstract

In order to forecast highway pavement performance and to design adequate pavement structures, detailed traffic loading information is essential. Traffic data collected by two unique weigh-in-motion (WIM) systems located in the southbound lanes of US 59 in east Texas have been analyzed and used to develop a methodology for forecasting future traffic loading patterns. The WIM systems, which have been in service continually since late 1992, have collected such data as the date, time, speed, lateral lane position, axle spacings, and wheel loads for about 7,500 individual vehicles per day. Thermocouples in the air and embedded in the pavement have measured and recorded hourly air and pavement temperatures, respectively.

Data from the WIM systems, which are located approximately 5 km apart, allow cross-referencing of individual vehicle characteristics, as well as comparison of aggregated data between sites. Data analysis has included trends for speed, lateral lane position, traffic volume, lane utilization, and vehicle classification. Historical traffic classification counts and annual average daily traffic maps provided by the Texas Department of Transportation (TxDOT) have been used to estimate growth rates for overall traffic volume and for various traffic classes within the study corridor. Analysis of the above data has been used to develop a procedure for predicting future traffic loading patterns. Thus, included in this report is a sample forecast for a typical pavement structure located within the 250-km project corridor.

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PRELIMINARY RESEARCH FINDINGS ON TRAFFIC-LOAD FORECASTING USING WEIGH-IN-MOTION DATA

by

Clyde E. Lee
Jeffrey W. Pangburn

Research Report Number 987-5

Research Project 7-987

A Long-Range Plan for the Rehabilitation of US 59 in the Lufkin District

conducted for the

TEXAS DEPARTMENT OF TRANSPORTATION

by the

CENTER FOR TRANSPORTATION RESEARCH
Bureau of Engineering Research
THE UNIVERSITY OF TEXAS AT AUSTIN

June 1996

IMPLEMENTATION STATEMENT

The long-range rehabilitation plan developed as part of this project will be directed toward the needs of US 59 within the Lufkin District. Although this long-range plan is being developed specifically for the Lufkin District, the framework of this plan may be utilized for the cost-effective rehabilitation of pavements throughout Texas.

Prepared in cooperation with the Texas Department of Transportation

DISCLAIMERS

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 Texas Department of Transportation. This report does not constitute a standard, specification, or regulation.

ACKNOWLEDGMENTS

The continuing research study upon which the content of this report is based is being conducted by the Center for Transportation Research at The University of Texas at Austin and is sponsored by the Texas Department of Transportation (TxDOT), specifically by its Lufkin District. J. L. Beaird (former District Engineer, now retired), initiated the research, and David Justice, Lufkin District Engineer, now guides the study. Technical coordination of the construction of the research pavement test sections and the installation of the WIM systems were provided by Harry Thompson, Livingston Area Engineer. Eric Starnater, formerly Assistant Area Engineer and now Lufkin District Pavement Engineer, was responsible for engineering support during site construction and continues to assist in field operations, especially WIM data collection. Personnel from TxDOT's Transportation Planning and Programming (TPP) Division in Austin, including Dean Barrett, Dayton Grumbles, Brian St. John, Willard Peavy, Alan Grahman, and others, have provided essential expert guidance and skills to make the WIM systems function. Sincere appreciation is expressed to all these dedicated people and to others not mentioned who have contributed to the success of the phase of the research project reported herein.

NOT INTENDED FOR CONSTRUCTION, BIDDING, OR PERMIT PURPOSES

Clyde E. Lee, P.E. (Texas No. 20512)
Research Supervisor

TABLE OF CONTENTS

SUMMARY

Detailed traffic loading information is essential in forecasting highway pavement performance and in designing adequate pavement structures. Traffic data collected by two unique weigh-in-motion (WIM) systems located in the southbound lanes of US 59 in east Texas have been analyzed and used to develop a methodology for forecasting future traffic loading patterns. The WIM systems, which have been in service continually since late 1992, have collected such data as the date, time, speed, lateral lane position, axle spacings, and wheel loads for about 7,500 individual vehicles per day. Thermocouples in the air and embedded in the pavement have measured and recorded hourly air and pavement temperatures, respectively.

Data from the WIM systems, which are located approximately 5 km apart, allow cross-referencing of individual vehicle characteristics, as well as comparison of aggregated data between sites. Data analysis has included trends for speed, lateral lane position, traffic volume, lane utilization, and vehicle classification. Historical traffic classification counts and annual average daily traffic maps provided by the Texas Department of Transportation (TxDOT) have been used to estimate growth rates for overall traffic volume and for various traffic classes within the study corridor. Analysis of the above data has been used to develop a procedure for predicting future traffic loading patterns. Thus, included in this report is a sample forecast for a typical pavement structure located within the 250-km project corridor.

CHAPTER 1. INTRODUCTION

The damaging effects of traffic loading on the performance and expected life of highway pavements have been a concern for many years. Ever since the American Association of State Highway Officials (AASHO) conducted the AASHO Road Test (1956 through 1960), the damaging effects of vehicle loads on pavements have received continuous attention by both researchers and legislators. The collection of statistical traffic loading data has become much more common with the advent of weigh-in-motion (WIM) technology.

WIM systems measure the real-time dynamic force of a vehicle tire at both high and low speeds (with little or no interruption of vehicle movement). Using these forces, the system is capable of estimating static vehicle axle loads and weight. This technology provides a much greater capacity for sampling the weight of vehicles — and does so without placing a direct cost on the road user. At a minimum, WIM systems can measure and store axle loads and axle spacings for individual vehicles; they can also typically store such supplementary data as the date, time, speed, lane of travel, station identification, and vehicle class. Some WIM systems are capable as well of storing data in statistical bins in order to facilitate the determination of such traffic trends as average speed and vehicle classification information.

In addition to the data mentioned above, the WIM systems installed for this project also record the lateral position of each vehicle within the lane of travel and the number of tires (single or dual) at the end of each axle. While the initial WIM installations used a thru-beam infrared source and receiver in each lane for this purpose, current replacements consist of a reflex infrared sensor and a retro-reflector in each lane.

BACKGROUND

As part of Center for Transportation Research (CTR) and Texas Department of Transportation (TxDOT) Research Project 987, WIM data have been recorded and stored continually from two sites on southbound US 59 in the Lufkin District since late 1992. Highway US 59 is a four-lane, divided primary arterial highway originating in Texas at the Texas-Mexico border in Laredo, traversing northeast through Houston and then north through Lufkin, and exiting Texas in Texarkana at the Texas-Arkansas border. The northern WIM site is located approximately 8 km to the north of Corrigan, Texas, while the southern WIM site is located approximately 3 km to the south of Corrigan. Both sites are operated in conjunction with five pavement test sections, each 300 m long, that are designed and constructed to represent the various pavement structures existing along US 59 within the Lufkin District. In addition to the WIM data, pavement performance measures, such as condition surveys and pavement surface profiles, have been recorded for use in related aspects of the research project. The focus of this research project is to develop a long-range pavement rehabilitation plan for the approximately 250 centerline km of US 59 within the Lufkin District.

In the development of such a plan, the importance of accurate traffic data must be stressed, as the cost of both the over- and under-design of a pavement structure should be avoided. The primary cause of both rigid and flexible pavement fatigue can be attributed to the loads applied to the road surface by individual vehicle tires. Pavement thickness is the most important means of achieving successful pavement performance for rigid pavements that experience large numbers of heavy truck axle loads. Fatigue damage varies by a factor of 20 between thin and thick rigid pavements (Ref 15). Rutting in flexible pavements has been determined to be proportional to the cumulative axle loads applied to the pavement. Therefore, both the magnitude of axle loads and the frequency of the loads must be determined in order to make an adequate estimate of the pavement structure required to successfully carry the projected traffic load.

Predicted traffic loading, a key variable in pavement design, is used in economic considerations as well. Analyses have shown that economies of scale apply to pavement design. For example, for a given increase in equivalent 80-kN (18-kip) single axle loads (ESALs), say 10 percent, only a 1.5-percent increase in pavement thickness would be required to sufficiently support the additional traffic loading (Ref 16). Therefore, the most accurate forecast of traffic loading should be conducted for any pavement design to avoid either the over- or under-design of the pavement structure. Where an over-design may result in the inefficient allocation of scarce resources, an under-design could result in the premature failure of the pavement and may require an extensive and costly rehabilitation program.

Given the ongoing implementation of the North American Free Trade Agreement (NAFTA), increased attention is being directed towards the Lufkin District as a link in a new interstate highway corridor, or NAFTA-Superhighway, that can possibly serve traffic moving between Canada, the U.S., and Mexico. When President Clinton signed the bill that eliminated the national 55 mph speed limit, he also approved an attached provision (included by U.S. Representative Tom DeLay, R-Sugar Land) that effectively allows US 59 to be designated a future interstate highway, namely, I-69 (Ref 5). The fact that the proposed route will generally follow the US 59 corridor in the Lufkin District underscores the usefulness of the WIM data being collected as part of this research project.

OBJECTIVE

The US 59 arterial highway, originally constructed in the 1950s, continues to serve as an important traffic corridor through east Texas 50 years later. More than a dozen seaports located along the Gulf Coast can easily access US 59, including the Houston shipping ports (Ref 14). The associated traffic loads applied to the highway have contributed to the deterioration of the pavements along the route. The resulting periodic pavement maintenance generally has consisted of an asphalt concrete pavement overlay on the existing pavement structure, rigid or flexible. Such resurfacing has left a large assortment of hybrid pavement structures that historically have not performed satisfactorily.

The primary goal of Project 987 is to recommend long-range pavement rehabilitation plans for the portion of US 59 within the Lufkin District. In order to achieve this goal, the objective of this report is to utilize the WIM data collected as part of the research project to characterize past traffic loading patterns and to forecast future traffic loads that will occur along this highway.

CHAPTER 2. PROJECT HISTORY

The weigh-in-motion (WIM) effort on US 59 near Corrigan, Texas, was initiated during the summer of 1991 under the guidance of Dr. Clyde E. Lee. Mr. Joseph Garner, a doctoral student in civil engineering, conducted much of the early research and has fully documented the installation and design efforts of this unique WIM system. The primary goal of this portion of the overall research project was to provide detailed traffic information in order to facilitate the development of a long-term pavement rehabilitation program for US 59 in the Lufkin District.

PROJECT CONFIGURATION

Since the later months of 1992, detailed vehicle records have been recorded by two modified PIETZSCH Automatisierungs-Technik GmbH (PAT) DAW100 WIM systems. Each system records traffic data in the two southbound lanes of US 59 at the respective sites. While a brief summary of the equipment required follows, the reader is encouraged to consult Garner's report (Ref 8) for a detailed description of the system installation, commissioning, and specifications. Each travel lane contains two bending-plate wheel-force transducers (weighpads, one per wheel path), an inductance loop detector, and an infrared beam sensing device (Figs 2.1, 2.2, and 2.3). Two thermocouples are also utilized at each site to measure and record the hourly ambient air and pavement temperatures.

Figure 2.1 Southbound view of US 59 at north site

Figure 2.2 Southbound view of US 59 at south site

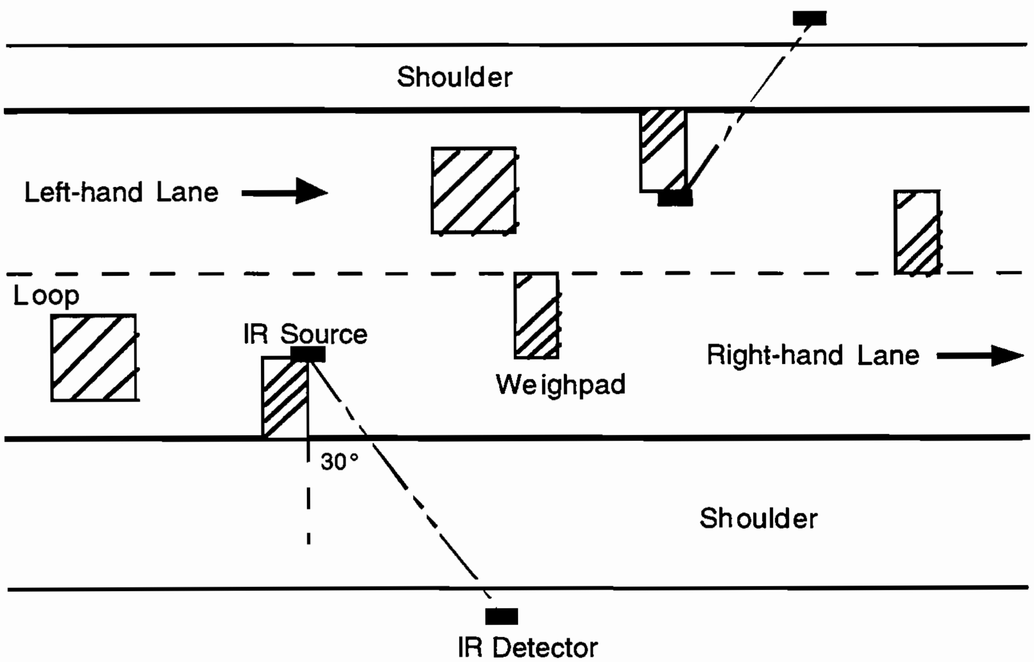

Figure 2.3 Schematic of typical US 59 WIM system layout

Weighpads

By staggering the two weighpads 4.5 m longitudinally in the respective wheel paths of each lane, the need for multiple vehicle-presence sensors is eliminated and measurement problems encountered with an accelerating vehicle are overcome (Ref 8). Utilizing the known distance between weighpads and the travel time for the front axle from the leading to the trailing weighpad, vehicle speed is calculated. For a representative vehicle speed measurement, the speed of the front axle is a reasonable estimate. However, if the vehicle is accelerating or decelerating, the average speed of successive axles is needed to provide a basis for calculation of axle spacing. The staggered weighpads make it possible to measure the speed of each axle.

Inductance Loop Detectors

Prior to the installation of the WIM systems on US 59, various geometries for the inductance loop detector were considered. A 6-turn, 1.8-m square loop arrangement was determined to provide the best vehicle-presence detection for the traffic encountered in this area. Of primary concern were the 5-axle semi-trailer trucks utilized for log-hauling operations throughout the area. The trailers consist of a tandem axle at the end of a retractable, telescoping steel pole that either extends to the length of the load being carried or is fully retracted and carried over the drive tandem of the tractor. The mass of metal in the extended steel pole is insufficient for reliable presence detection by conventional inductance loop arrangements, but it was usually detected by the 6-turn, 1.8-m square loop.

The inductance loop detector activates the DAW100 once the presence of a vehicle is sensed, and maintains the signal until presence is no longer detected, at which time the DAW100 provides a programmable time extension. It is important to calibrate this extension to allow the complete recording of all axles on a vehicle, without providing an excessive extension that will indicate the presence of the next successive vehicle.

Infrared Beam Sensing System

Incorporation of a modulated infrared beam sensing system is what makes this WIM system unique. Located on the leading weighpad was an OPCON 1173A-100 infrared thru-beam source housed inside a specially constructed aluminum housing of the same approximate size of a typical raised pavement marker (Fig 2.4). The infrared beam originated in the center of each lane and was aimed downstream towards the respective shoulder at a 30-degree angle just above the pavement surface. The housing was bolted onto a solid steel mounting plate that had been welded onto the weighpad steel frame before installation in the pavement. The thru-beam infrared detector was located off the shoulder approximately 0.6 m and along the 30-degree beam projection. A separate OPCON 70 Series control unit placed inside the cabinet modulates (switches on-and-off in a coded pattern) the source unit, and receives and interprets the reflected infrared beam signals via the infrared thru-beam receiver and parlays the conditioned signals to the DAW100. The received signal is conditioned to respond only to the modulated signal from the source, not to other sources of infrared light (e.g., sunlight).

An annoying difficulty in operating the infrared sensor system was the lack of durability of the infrared source that was bolted onto the corner of the leading weighpads. Although the source units were located in the middle of each lane, they received numerous blows, apparently from objects either dragged under or trailing behind vehicles, which left various deep impressions in the aluminum ramps on each housing. On a couple of occasions such blows sheared off the bolts that held the source housing to the mounting plate. We added shear pins to each housing to increase the strength of the bolted connections, a modification that extended the service life of the source housings considerably. At the south site a weighpad frame cracked at the two welded connection points with the mounting plate when the supporting pavement deformed excessively after approximately three years of use. The integrity of the weighpad frame was deemed to be adequate for continued use; the void left by the removed mounting plate was filled with asphalt concrete.

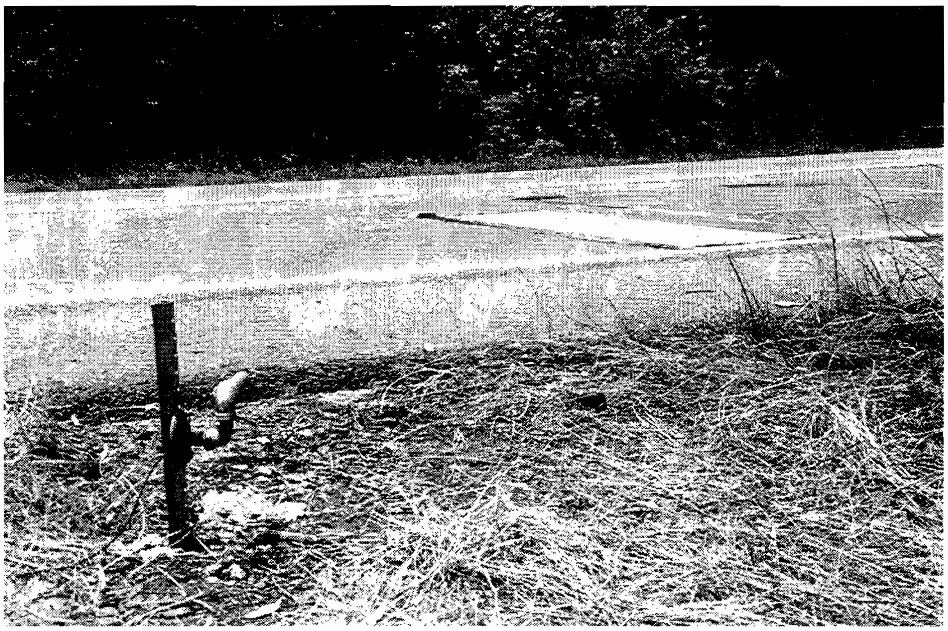

Figure 2.4 Left-hand lane infrared thru-beam setup

Another problem with the infrared sensors was water leakage, which ruined two infrared sources on separate occasions and required their replacement. Replacement often required the extraction of frozen or sheared bolts, which was both time consuming and required traffic control measures to close the affected travel lane. The infrared receivers offset from the shoulders also had problems, as mowers knocked over two posts and vehicular accidents bent three receiver posts during a three-year period. Water seeping into the receivers also caused electrical shorts on two occasions and required their replacement.

The infrared sensor system allowed the calculation of the lateral placement of each vehicle and a measure of whether an axle consisted of dual tires or a single tire. The methodology behind the physical layout and the calculations is detailed in Chapter 4.

Thru-beam vs. Reflex Sensors: In the summer of 1995, an OPCON 80 Series infrared reflex sensor and control unit was substituted for a malfunctioning thru-beam sensor system. Near where the infrared thru-beam source had been located on the pavement surface was placed a specially designed low-profile, aluminum reflective button. The button consisted of a 1-on-4 upstream ramp to deflect any impacts and a side that consisted of 3M reflective sheeting. The reflective area was approximately 130 mm by 25 mm and was inset approximately 6 mm to avoid direct tire contact with the reflective sheeting. The button was held in place by applying hot asphalt to the bottom surface of the button (approximately 100 mm by 200 mm) and to the roadway surface, and then joining the two and allowing sufficient curing time. An integral reflex source-receiver-control unit was substituted in place of the thru-beam detector off the shoulder.

A major advantage of the reflex sensor is the ability for one person to align the sensor, as compared with the need for at least two people to align the thru-beam sensors. The infrared reflex sensor unit incorporates a beam-status LED that indicates proper beam alignment. This LED for the thru-beam infrared sensors is located inside the equipment cabinet. Thus, the LED cannot be viewed by the person adjusting the receiver unit. Another advantage of the reflex sensors is the absence of electronic equipment on the pavement surface. By utilizing a simple reflector in the roadway, replacement can be accomplished in about 5 minutes (with only minimal impact to traffic) by a TxDOT road crew. In addition, the possible detrimental effects of water are reduced significantly, since there are fewer exposed electrical connections.

Instrument Cabinet

An aluminum NEMA-type traffic equipment cabinet equipped with a thermostatically controlled exhaust fan houses the DAW100, a short-term back-up power supply, the infrared thru-beam sensor control units, the ambient air thermocouple, a 6-outlet surge suppresser, a phone line surge suppresser, and a 14,400 baud rate US Robotics Sportster external modem. In addition to the above, the south site also contains a 230 mm electric box fan to facilitate air circulation within that cabinet.

DATA ACQUISITION

Both WIM systems are fully automated and require no direct user inputs during normal operation. Each system is capable of storing roughly 4Mb of WIM data, equivalent to approximately fifteen days of data at these locations. The actual number of days that can be handled by on-site storage is a direct function of the number and length of the traffic records, with an average day near Corrigan having about 7,500 vehicle records. At the end of each day (midnight), the DAW100 system saves the just-expired day's data file internally and deletes the oldest file still in storage, whether the file has been downloaded or not.

Using the CC200 software provided by PAT, it is possible to communicate electronically with each system on a real-time basis. A US Robotics external modem is used at the roadside on the DAW100 and at the remote computer used for communications. These modems have performed exceptionally well, considering the weather extremes to which they are subjected, including temperatures that range from -7° C (20° F) to over 38° C (100° F) during the summer months. In-line telephone surge suppressers have been used on occasion at the roadside and have performed well (only one burn-out during operation). It is important to note the importance of surge protection for the system electric power, as numerous electronic circuits depend on an uninterrupted power supply.

Owing to the approximately 400-km distance between Austin and Corrigan, phone calls made between the sites are charged at long-distance rates. Experience has shown an effective downloading rate of approximately 4200 bps, resulting in a phone-connect time of approximately 12 minutes per daily file. For this reason, an IBM compatible microcomputer was placed at the TxDOT maintenance facility in Corrigan for routine downloading of WIM data at local phone rates. Currently, the data are stored on the microcomputer hard drive to be transferred at a later time. Software (CC200) incorporates an automatic call log that allows the automatic scheduled downloading of data at times specified by the user for a 15-day period. Once the 15-day period has expired, it is necessary for an individual (Mr. Eric Starnater of the TxDOT Lufkin District in this case) to reset the software call log to prevent the DAW100 system from overwriting current data (typically an additional fifteen days for data files of this traffic volume). Ideally, the downloading software could be programmed to operate on a specified basis (such as every four days) for an indefinite period of time, thus eliminating the requirement of manually resetting the automatic call log.

Once the binary data are transferred from the remote microcomputer's hard drive to floppy disks, the disks are mailed to The University of Texas at Austin for permanent storage and analysis. Using the WIMFTP program developed by Mr. Liren Huang of The University of Texas at Austin, the original data files are translated from binary format into two ASCII text files, one consisting of hourly temperature data and the other consisting of the individual vehicle records. A schematic of the data acquisition and analysis procedure is provided in Figure 2.5.

OPERATIONAL PROBLEMS

Throughout the data collection effort, various problems arose; some had minimal impact while others resulted in the loss of data. The problems ranged from malfunctioning infrared sensors and loop detectors, to inoperable electronic circuit boards and modems. The following comments focus on events occurring after September 1994.

Data inconsistency problems were encountered during October 1994, when over 500 mm of rain fell within a 48-hour period. With flooding throughout the project area, US 59 was closed for a period of time and various equipment problems began to arise. The communications board at the north site was the first obvious problem, as remote communication with the DAW100 was not possible. Once the communications board was replaced, communication was still sporadic; the DAW100 was consequently shipped to PAT in Pennsylvania for service. Once returned from PAT, the DAW100 was reinstalled but failed to operate and was subsequently returned to PAT for service again. This extended service period required three trips to the site and lasted until March of

1995. Once the system had become functional again, it was apparent that the loop detector wires in the pavement at the north site were damaged; these were replaced late in March of 1995. Since then, the system has continued to function properly.

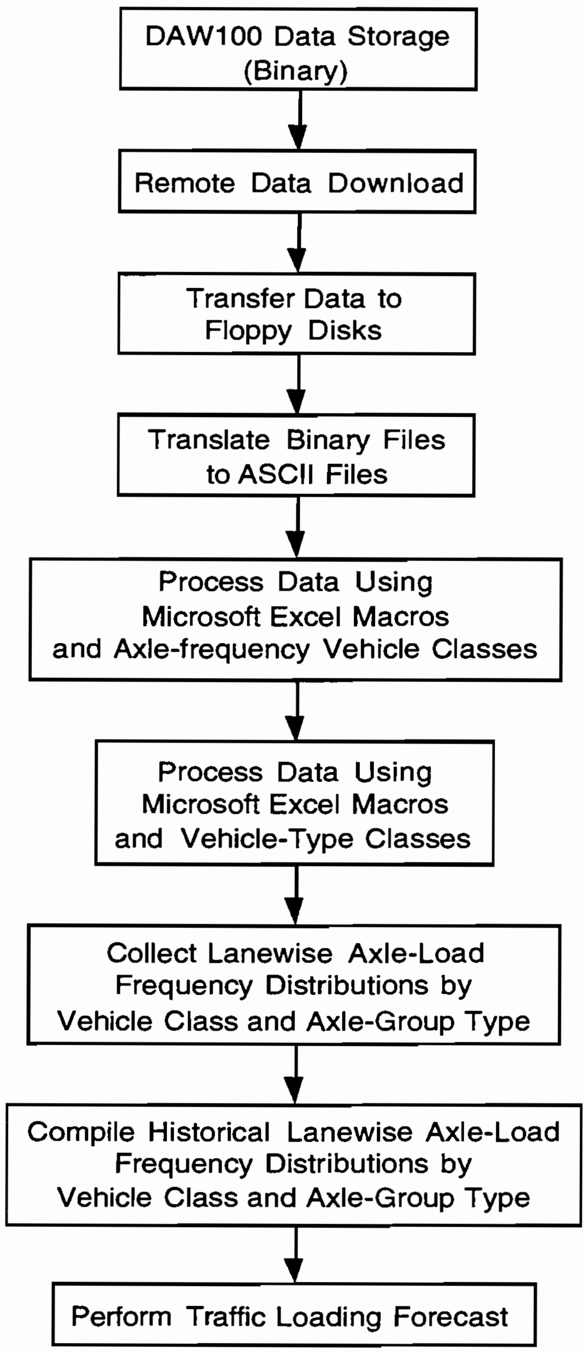

Figure 2.5 Schematic of the data acquisition and analysis procedure

Since replacing the Practical Peripherals roadside modems at both sites with US Robotics modems, communication problems between the sites and the remote computer at Corrigan have been negligible. The hard drive in the IBM-compatible microcomputer originally placed for remote downloading malfunctioned after two years of continuous service; another IBM-compatible microcomputer was substituted and has performed well. Although these malfunctions have resulted in some lost data, the duration of these problems was typically short and sporadic, and otherwise allowed the recording of the majority of data files.

ERRONEOUS DATA

As with any experimental measurements, there is a tendency for erroneous data to arise. The presence of erroneous data is not a sign of unacceptable data, but must nevertheless be edited from the remaining data sets in order to provide worthwhile data. The error can be deemed acceptably small (such as not to affect the outcome of future analyses), with the data seen as introducing a smaller error than would be experienced if the erroneous data were discarded. At the inception of this project, we intended to record every vehicle that traversed the WIM systems for three consecutive years. Unfortunately, owing to events both controllable and uncontrollable, data have either been lost or corrupted. During this project, several types of errors were identified that have been either accepted or corrected. The following four sections deal with some of these problems encountered during this project.

Missing Data

Ideally, the database for this project would contain a valid record of every vehicle that has traversed the WIM systems during the previous three years. Causes of missing data have ranged from the preventable (e.g., not resetting the automated downloading call log in time) to the unforeseen (e.g., equipment malfunctions).

In those cases where data had been lost, several separate techniques were utilized to fill the gaps in the data set so as to provide a complete traffic picture. The simplest case was where data from both the previous week and the subsequent week were available. In this case, simple interpolation was used for each vehicle class (except when holiday traffic was involved, which required the use of an extrapolation technique described below). In those cases where interpolation wasn't practical, the traffic and loading data collected from the adjacent WIM system were used. As shown later in this report, the two WIM sites had very similar data patterns; accordingly, it was determined that data for each site could be extrapolated from data from the other WIM site. In those rare cases where data were not available from either site, a growth factor determined from data available from the previous year was used to project the current missing data. For example, if data from May 6 through May 15 of the current year were unavailable from either WIM site, other May dates from the current year and the previous year were compared in order to determine a growth rate between the two years. This factor is then applied to the previous year's May 6 through May 15 data to generate an estimate of the missing data from the current year.

Ghost Records

The term *ghost records* originated during conversations between PAT personnel and project personnel to describe the occurrence of non-vehicle-generated records. Conveniently, each ghost record consisted of the same default axle spacing with identical wheel loads and a speed value equal to the previous vehicle. Indicating that these records were being generated in response to a very short loop impulse, PAT supplied new EPROMs for the DAW100 systems, which effectively filtered these particular records from being recorded into storage. Recently, different ghost records have been generated that consist of a wheel load value different from the previous ghost records. However, the front axle load measurement for these ghost records is below the specified minimum axle load and, accordingly, the records are not stored. Although the new EPROMs have removed the ghost records from the data sets, the records are still generated and have the effect of increasing the vehicle record number, as occurred with the old EPROMs; therefore, the actual number of vehicles during a day is much lower than the vehicle record number of the last vehicle.

Since the front axle load on lightweight vehicles is typically heavier than the rear axle, the minimum axle load criteria may prevent the recording of some rear axle loads. To alleviate this, the DAW100 is programmed to copy the front axle loads to the rear axle, and to substitute a default axle spacing for the record. It was observed that these records contain the same speed as the previous vehicle in the respective lane. This procedure was determined to be preferable to discarding the entire vehicle record.

Off-Scale

On occasion, a vehicle will miss a weighpad while traversing the WIM sites. Whether owing to one side of the vehicle riding on the shoulder, or to the vehicle executing a lane-change maneuver, the DAW100 does not take any corrective measure; the vehicle record will consist of zero weight for those wheel loads that miss a weighpad.

One solution to this problem would have the DAW100 automatically place a factored wheel load in place of the missing data. The simplest replacement would be to copy the opposite wheel load on the same axle, thereby creating an axle with an even loading distribution between right and left wheel loads. Another approach would be to generate a statistical sample of the distribution of wheel loads on individual axles, and then use this distribution factor and the available wheel load to generate the missing wheel load. Although both corrective measures represent estimates, they would be preferable to either accepting a zero wheel load or discarding the entire record.

Log Trucks

Given the area's numerous tree-logging operations, US 59 carries a substantial number of semi-trailer trucks equipped with telescoping pole trailers. These trailers are typically retracted and carried over the drive tandem of the tractor when not in use. While carrying a load of lumber, the trailer tandem is extended in order to support the length of the load, resulting in a small mass of metal (the telescoping pole) situated between tandems of the semi-trailer truck.

To properly detect the presence of this small mass of steel, the sensitivity of the inductance loop detectors was set fairly high. However, once the sensitivity reached a certain point, the loops tended to generate false impulses owing to extraneous noise activation; accordingly, we set the sensitivity just below this threshold. While the resulting sensitivity detected the presence of these trailers on a frequent basis, it did occasionally cause a vehicle record to be terminated prematurely.

SUMMARY

Despite the various setbacks, the traffic data set collected during this project is one of the most complete and detailed ever compiled. The initial data collection effort was undertaken in late 1992 and has continued through early 1996. When functioning properly, the WIM systems measure and record data on every vehicle traveling through this southbound stretch of US 59; these data include wheel loads, axle spacing(s), lateral lane position, speed, time and date, whether an axle has single or dual tires, and the lane of travel. Meanwhile, other researchers with the Center for Transportation Research have compiled detailed records on pavement conditions and performance throughout the study period. Unlike the AASHO Road Test conducted by the American Association of State Highway Officials (AASHO, now AASHTO), the traffic loading is random in nature, as there are no constraints regarding which vehicles are measured and recorded (i.e., all southbound vehicles traverse the WIM systems). The long duration of the project has also allowed the testing and evaluation of various equipment arrangements, including infrared light-beam sensors. The long-term success of this project has demonstrated the reliability of WIM systems in the collection of traffic loading data.

CHAPTER 3. DATA COLLECTION AND ANALYSIS METHODOLOGY

PROCEDURES

The data analysis for this project was performed by Mr. Joseph Garner. Using Microsoft Excel macros and spreadsheets, an initial sorting of the data by the number of axles within a vehicle record was performed. Records with obvious errors (such as speeds in excess of 100 mph and axle spacings greater than 18 m) were removed. Once sorted, the data were processed for lateral position, for the presence of dual tires on each axle, and for the number of 80 kN (18-kip) equivalent single axle loads (ESALs) for each individual vehicle record.

For this report, additional macros were developed to further sort the records into classes that more closely resemble the vehicle classes used in manual classification surveys conducted annually by TxDOT. Instead of focusing on ESALs for each individual record, a greater emphasis is placed on the axle-group load-frequency distributions for each vehicle class. Using these distributions, traffic volumes by vehicle class were forecast and cumulative ESALs contributed by class were calculated.

INFRARED DATA

The OPCON 1173A-100 thru-beam infrared sensors integrated into the design and construction of both WIM systems have performed well. By aiming the infrared light beam at a 30-degree angle across each lane, we were able to determine both the lateral position of each vehicle and the number of tires on each axle.

Lateral Position

By utilizing the simple geometry of similar triangles, the time differential between weighpad stimulation and infrared blockage provided the independent variable necessary to determine the lateral position of the right or left front tire with respect to the appropriate edge of the lane (Fig 3.1). The right tire is used for vehicles traveling in the right-hand lane, while the left tire is used for vehicles traveling in the left-hand lane. The location of each point on the larger triangle is known (the corner of a weighpad for the infrared source, a location offset from the shoulder for the receiver along the 30-degree infrared light-beam projection, and the convergent point offset from the shoulder); the time differential required to travel between the weighpad and the light-beam multiplied by the speed of the vehicle provides the longitudinal distance (d_x) for the smaller triangle. Knowing this value, a simple proportion is utilized to calculate the distance from the source (the corner of the weighpad) to the tire path (d_y). Subtracting this value from the known distance from the edge of the appropriate lane to the source (the corner of the weighpad) then provides the lateral position of the vehicle from the edge of the lane.

It is important to note that the lateral position measurement contains a sizable degree of error in its calculations, approximately ±150 mm. However, this error was deemed acceptable for the level of accuracy required in this project. The error is due in part to the fact that the time differential between weighpad activation and infrared beam interruption depends on when the weighpad activation precisely occurs. The weighpads are preset with a threshold impulse value that must be reached before the system will begin operation. The amount of time required to reach this threshold may vary according to vehicle type, speed, and environmental conditions. Therefore, the equations used to calculate the final lateral placement value were derived from a linear regression on experimentally measured data. The time differential between weighpad activation and infrared beam interruption, as well as the visually measured lateral placement of sample passenger cars, was analyzed. Using a curve-fitting procedure, equations were derived to equate the measured time differential with an appropriate lateral placement. Creating another minor source of error was the fact that the infrared beam is blocked by the inside of the steering axle tire, not the center of the tire. The fact that dual tires tend to track slightly further outside of the steering axle tires will also impact the lateral placement measurement. The calibration of the lateral placement procedure is detailed in Garner's report.

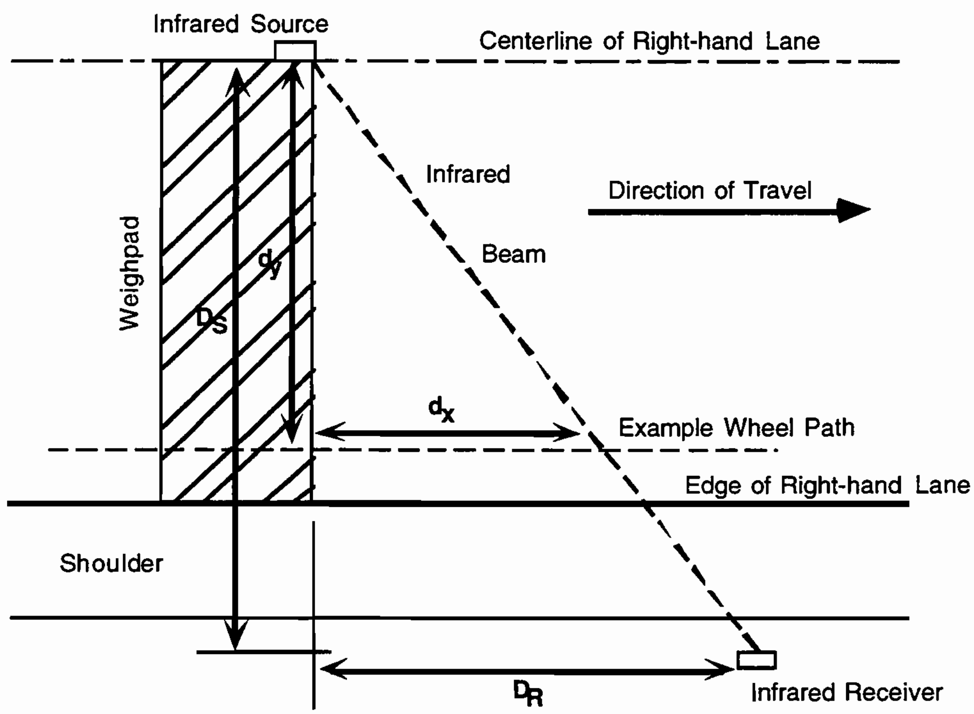

Figure 3.1 Schematic of lateral position measurement

Tire Frequency

A disadvantage associated with the use of WIM systems (and with other automatic vehicle classification systems) is that they are not able to distinguish between certain vehicle classes.

Historically, classification counts were conducted manually, with the operator relying on visual identification of vehicles. With automated systems, this visual aspect has not been available, resulting in traffic being classified according to axle spacings only, with no information on the number of tires on any axles.

By measuring the duration of time the infrared light beam was interrupted by the tire(s) on each axle-half, the number of tires on each axle could be determined. The methodology assumed that the first axle (the steering axle) always has a single tire and used the interrupt duration for this tire (t_s-t_0) as the criterion against which the other axles were measured (Fig 3.2). With the infrared light beam placed at a 30-degree angle, a set of dual tires will cause an interrupt time noticeably longer than that caused by a single tire (Fig 3.3). As the dual tire set first crosses the infrared light beam, the leading edge of the inner tire blocks the beam first (t_0) and maintains the blockage for a duration equivalent to the tire length divided by the speed at which it crosses the beam. However, once the inner tire clears the infrared light beam (t_a), the beam is still blocked by the outer tire (t_b), as illustrated in Figure 3.3. Garner reports that an interrupt time 20 percent greater than that caused by the steering axle tire is an adequate indication of the presence of dual tires on that axle.

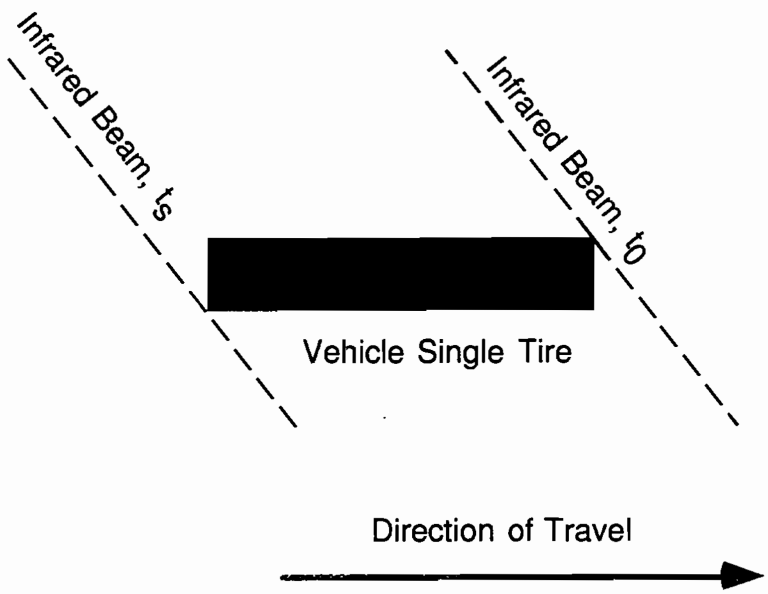

Figure 3.2 Schematic of infrared single tire interrupt duration measurement

Therefore, if:

$$\left(t_b - t_0\right) \geq 1.2\left(\Delta t_s\right)$$

then the axle in question would have dual tires. If the statement is determined to be false, then that would indicate single tires on that axle.

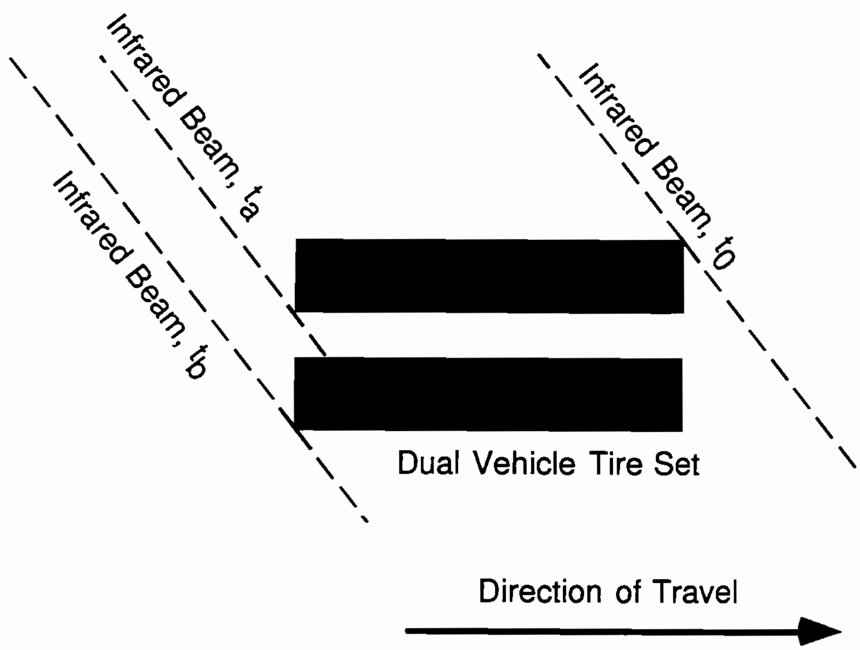

Figure 3.3 Schematic of infrared dual tire interrupt duration measurement

CALIBRATION

The general premise of WIM is that the dynamic tire forces generated by a moving vehicle will be measured and used to estimate the corresponding load that would be indicated on a static scale. In order to accomplish this, the unique characteristics of a site, including the grade, cross-slope, longitudinal and transverse surface profiles, local roughness and other factors, all must be taken into account, along with the complex interaction of the tire-force transducers, the vehicle suspension system, the vehicle tires, and the environment.

The adjustment of the dynamic tire force (measured under a particular set of on-site conditions) to produce the best estimate of the value that would be indicated by a static scale is called calibration. It should be noted that the estimate will rarely agree exactly with the static value, as the aforementioned variables affect the dynamic measurement to various degrees. Owing to the lack of static tire load measurements for the calibration truck, the DAW100 systems were calibrated to provide the best estimate of gross vehicle weight for the existing site conditions.

As a supplementary function to estimating weight, the impulses generated by the staggered weighpads are used in the calculation of axle spacings and vehicle speed. Over time, the response of the weighpads and the pattern of dynamic tire forces may change and cause the weight threshold for a wheel load to be either earlier or later than that previously calibrated. Software in the

DAW100 allows input of an effective distance between weighpads; therefore, axle spacing and speed measurements can be calibrated to coincide with actual values. This user-input adjustment is not to the physical distance between weighpad frames, but to the distance between the points where the tire is located on each respective weighpad when the tire-force signal exceeds a selected threshold value. These are the time points used in the axle spacing and speed calculations.

Calibration frequency is a function of both equipment stability and site-specific roadway conditions. As roadway and equipment variables change, so will the dynamic tire forces and the indicated estimates of applied loads. Calibration of the WIM sites in the Lufkin District has been scheduled on at least an annual basis in order to assure good WIM data quality.

Procedure

Calibration procedures were conducted in September of 1996 with the assistance of Mr. Luis Sanchez-Ruiz, who had previous experience with calibration of the DAW100 WIM systems located in El Paso and Laredo, Texas.

The reference calibration vehicle was a 5-axle semi-trailer (FHWA Class 9 or 3S2), with an air-bag suspension and a flat-bed trailer loaded with a steel-tracked Caterpillar dozer. This vehicle type was utilized for calibration owing to the high percentage (approximately 72 percent) of the overall traffic loading carried by these vehicles. Prior to calibration, the truck was weighed statically by successively moving axle groups onto a certified full-length vehicle scale to measure the axle loads and the gross-vehicle weight. Axle spacings were measured with a 15-m steel tape and were measured to the nearest inch at the site.

The software currently implemented on the DAW100 system has three programmable speed-point correction factors. These factors are applied at selected speed points to provide better results over a speed range than would be generated by a single speed corrective factor. These compensation points are distributed over the expected speed range of traffic at the site in order to improve the tire-force estimates (Ref 15). Static calibration of the weighpads was performed at the factory and was not calibrated in the field. Only the gross-vehicle weight of the calibration truck was used to calibrate the system, although all the measured tire loads were recorded for future reference.

The calibration truck was available for only one work day, and four sets of weighpads needed calibration during this time. Therefore, a minimum sample of only three calibration runs per speed point, per lane, was used. The small sample size was deemed adequate, since the system had been previously calibrated in August of 1993 and had indicated only a small change during this two-year time span. Site characteristics appeared to be basically the same as for the previous calibration, except for some minor pavement shoving upstream of the north site, left-hand lane weighpads where patching had been performed on a number of occasions.

Calibration was achieved by evaluating the existing speed-point corrective factors and adjusting them for the difference between the actual gross-vehicle weight of the calibration truck and the average WIM-estimated gross-vehicle weight of the calibration truck. The percent difference between each estimate and the actual gross-vehicle weight was calculated. If one of the three measurements differed from the other two by more than about 1 or 2 percent, it was discarded and another run was performed. Once three consistent measurements were recorded, the existing speed-point correction factor was adjusted by the appropriate difference in the gross-vehicle weight estimates. If the average percent difference was less than 1 percent, no adjustment was made and the existing speed-point correction factor was kept.

Owing to time constraints, the number of post-calibration runs by the calibration truck was limited. A minimum of one run per speed point was made. If the difference in gross-vehicle weight was greater than 1 percent, additional runs were conducted. If values from additional runs were consistently different, the speed-point correction factors were readjusted. Once the difference in gross-vehicle weight estimates was reduced to less than 1 percent, the calibration was considered complete.

Results

Calibration procedures began with the WIM system in the right-hand lane at the north site. As can be seen from the data in Table 3.1, the pre-calibration runs for the left-hand lane indicated that the estimated weight for all three existing speed-point settings was low and in the range of -2.9 percent to -5.1 percent. By adjusting the 50 mph speed point 3.5 percent higher, a post-calibration run indicated gross-vehicle weight within 1.0 percent of the static value (Table 3.2). Adjusting the 55 mph speed point 2.9 percent higher yielded an estimate within 0.2 percent. However, after adjusting the 60 mph speed point 5.1 percent higher, the estimate was within only 2.0 percent, so the speed point correction factor was increased an additional 2.0 percent. The following run indicated a difference of +1.9 percent, so the previous two speed point correction factors were interpolated to arrive at a new correction factor. The next run estimated the gross-vehicle weight 6.9 percent higher than the static reference value, so two more runs were made. These resulted in a difference of -0.5 percent and +0.2 percent, respectively. Since the last two runs were consistent with the calibration trend, they were accepted, and the third-to-last run was considered an outlier and was discarded. Table 3.2 summarizes data from calibration truck runs for this lane after speed-adjustment factors had been set.

Table 3.1 Left-hand lane pre-calibration data

Pass No.	Speed Point (mph)	Measured GVW (kips)*	% Difference	Avg. % Diff.
1	50	51.1	-5.3	-3.5
2	50	52.3	-3.1	
3	50	52.8	-2.1	
4	55	52.7	-2.3	-2.9
5	55	52.2	-3.3	
6	55	52.3	-3.1	
7	60	51.2	-5.1	-5.1
8	60	50.8	-5.8	
9	60	51.6	-4.4	

1 kip=0.454 Mg

Table 3.2 Left-hand lane post-calibration data

Pass No.	Speed Point (mph)	Measured GVW (kips)***	% Difference	Avg. % Diff.
1	50	53.4	+1.0	
2	55	54.1	+0.2	
3	60	52.9	-2.0*	-0.2
4	60	55.0	+1.9*	
5	60	57.7	+6.9**	
6	60	53.7	-0.5	
7	60	54.1	+0.2	

* Speed point was readjusted following these runs.
** This data point was not used.
***1 kip=0.454 Mg

Calibration data for the remaining lanes are provided in Appendix A.

The distributions of gross-vehicle weights (GVW) for 3S2 semi-trailer trucks have been proposed as a quality check of weigh-in-motion data (Ref 7). By comparing GVWs of these trucks to an established pattern derived from historical data, a shift in the pattern for the data in question can signal that there may be a problem with that data.

At the south-site WIM system, three of the six speed-point correction factors were increased by approximately 4 percent. The impact of this change is evident (Fig 3.4) in the shift of the second distribution peak for the October 1995 GVW sample. This peak shifts from 71 kips to 74 kips, which is approximately a 4-percent change. The ease of constructing and interpreting this plot lends itself well to regular use. Figure 3.5 plots weekly 3S2 GVW distributions: notice the shift starting with the 9/17-9/23 week, which is the first week following WIM system calibration.

The variability in site conditions must always be taken into consideration when operating a WIM system. As can be seen throughout the calibration data, surface irregularities can have a significant effect on individual measurements. Any change in the road profile or surface condition near the weighpads has a significant effect on the validity and accuracy of any data collected. A suggested means for providing an adequate, durable foundation pavement for WIM sensors is to construct a continuously reinforced concrete pavement (CRCP). The CRCP slab should be constructed approximately 61m (200 feet) in length, with the weighpads located approximately 46m (150 feet) downstream of the leading edge of the slab. This length is sufficient to allow two full semi-trailer trucks to stop in advance of the weighpads. This allows the damping of any vertical vehicle mass movements that will affect the dynamic tire forces measured by the WIM system. The slabs themselves should be ground smooth with a milling machine in order to provide the flattest surface possible.

Axle Spacing and Speed: By using the axle spacing of the calibration truck as a reference value, the axle spacing and speed measurements were calibrated. Using the first ten runs in each lane as data points, the average spacing between Axles 1 and 2 was calculated, along with the average overall wheelbase and the differences between these average values and the statically-measured spacings. The average of these two differences was then applied to the existing "distance between weighpads factor."

Even after calibration, the WIM systems displayed some variability in the axle spacing data, which indicates how the dynamic effects of a moving vehicle impact the calculation of both axle spacing and vehicle speed. This variability can be considered small, as the largest difference for an axle group was 3.4 percent, or 0.12 m (0.4 feet) over 3.5 m (11.7 feet) for the spacing between Axles 1 and 2. A range of only 0.03 m (0.1 feet) was experienced for both tandem axle sets. This variability, generally, will not affect the ability to distinguish between vehicle classes.

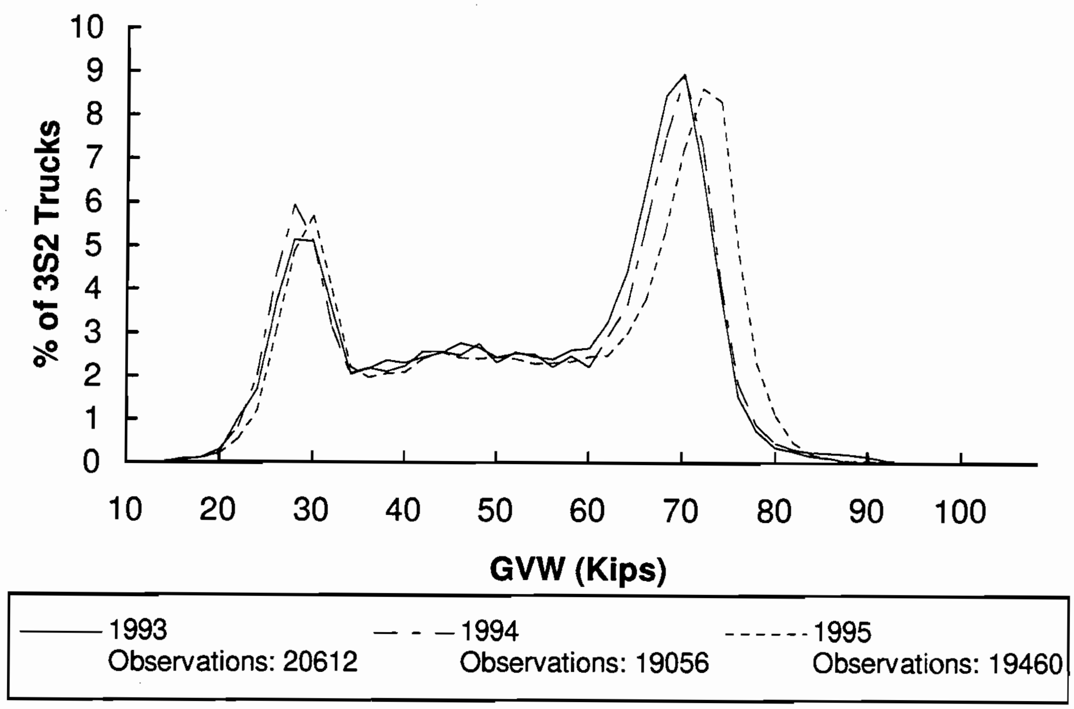

Figure 3.4 October 1993–1995, 3S2 gross vehicle weight distribution

WEIGHT ENFORCEMENT

Using the axle loading information, WIM systems can also be used for the real-time screening of the traffic flow for suspected over-loaded vehicles, which after being identified can be weighed using official static weigh scales to document the infraction. Given its inherent variability, modern WIM technology is not acceptable in a court of law as an enforcement tool, and until the accuracy and reliability of WIM systems can be certified, the data will not be regarded as sufficiently accurate in a court of law. The use of WIM technology allows the continuous operation of vehicle weighing. Studies have shown that only 1 percent of the trucks weighed at an open static weighing station are overloaded, while as many as 30 percent of the population of trucks not being statistically weighed are overloaded (Ref 16).

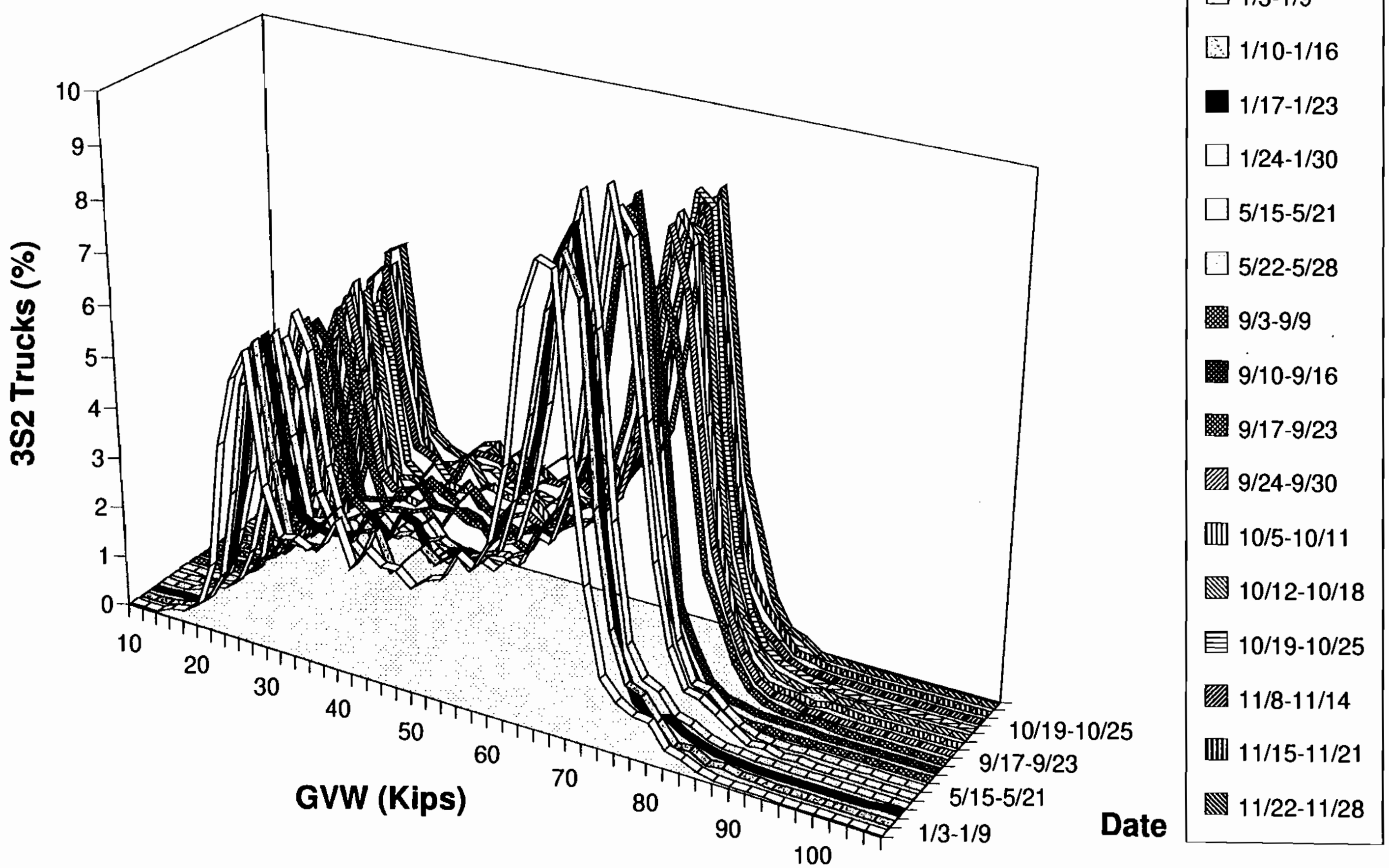

Figure 3.5 Weekly 3S2 Gross Vehicle Weight Distributions — 1995

An important aspect of WIM technology is the ease of identifying potential axle loading violations. Although a vehicle may be under the maximum allowable gross vehicle weight of 36 metric tons, it may contain one or more axle groups that exceed the allowable loads of 18 kips on a single axle, 34 kips on a tandem axle set, or by applying the bridge formula 48 kips for a tridem axle set. Since it is the axle groups that damage the pavement, not the overall gross vehicle weight, greater attention should be applied towards the axle group loadings.

TRAFFIC TRENDS

Since the main goal of this project was to produce a long-term forecast of traffic loading on US 59, historical traffic data were collected from the TxDOT Traffic Planning and Programming Division in Austin, Texas. The accumulated data consisted of traffic classification counts, annual average daily traffic maps for the Lufkin District, and past WIM data collected at Station 505, which was located on US 59 in Nacogdoches, Texas. With the full implementation of the North American Free Trade Act (NAFTA) looming in the near future, any traffic forecast based on historical data will be suspect until the full impacts of NAFTA traffic can be analyzed. However, given the recent (December 1995) postponement of the NAFTA provision to allow Mexican trucks access to the border states of California, Arizona, New Mexico, and Texas, travel patterns may remain the same for an extended period of time. This report will forecast traffic to be an extension of past traffic trends.

Annual Average Daily Traffic

Annual average daily traffic (AADT) maps for the Lufkin District were obtained from the TxDOT Traffic Planning and Programming Division for 1984 through 1994. From these data, traffic volumes were analyzed for the US 59 corridor and were used to estimate traffic growth rates. The AADTs on US 59 include both northbound and southbound traffic; therefore, the growth rates will be applied to both directions.

Using simple linear regression over the 11-year period, a best fit was performed to determine a linear growth rate. Three of the seven sites provided a goodness of fit greater than 0.90, while the remaining two sites with significant traffic provided goodness of fits equal to 0.76 and 0.86. These five sites had annual growth rates between 3.0 percent and 2.5 percent. The Meldrum and Nacogdoches north sites provided goodness of fit measurements less than 0.6, owing to the low AADT in this region. Both sites experience traffic volumes below 8,000 vehicles per day and annual growth rates slightly less than 1 percent. Therefore, these two sites were assumed to have 1 percent annual growth rates in order to provide a conservative forecast and account for their sporadic growth. Figure 3.6 indicates the AADTs for these seven sites.

According to the previous analyses, growth rates were assigned to sections of US 59 between available AADT data sites. Generally, the annual growth rate for US 59 from the Beaumont District border to Nacogdoches (where US 259 and US 59 intersect) is between 2.5 percent and 3.0 percent. The portion of US 59 from Nacogdoches to the Atlanta District border is less than 1 percent; however, 1 percent was used, as explained above. Table 3.3 identifies the individual growth rates calculated for US 59.

Traffic Classification Counts

An accurate traffic forecast must analyze not only the overall traffic growth or decline, but also the growth in individual vehicle classes. For example, from 1970 to 1983, 5-axle vehicles in the traffic stream increased from 9 to 17 percent. With 5-axle vehicles accounting for a large majority of ESALs on a pavement, a forecast that did not account for this growth would have resulted in a significant under-design of a facility.

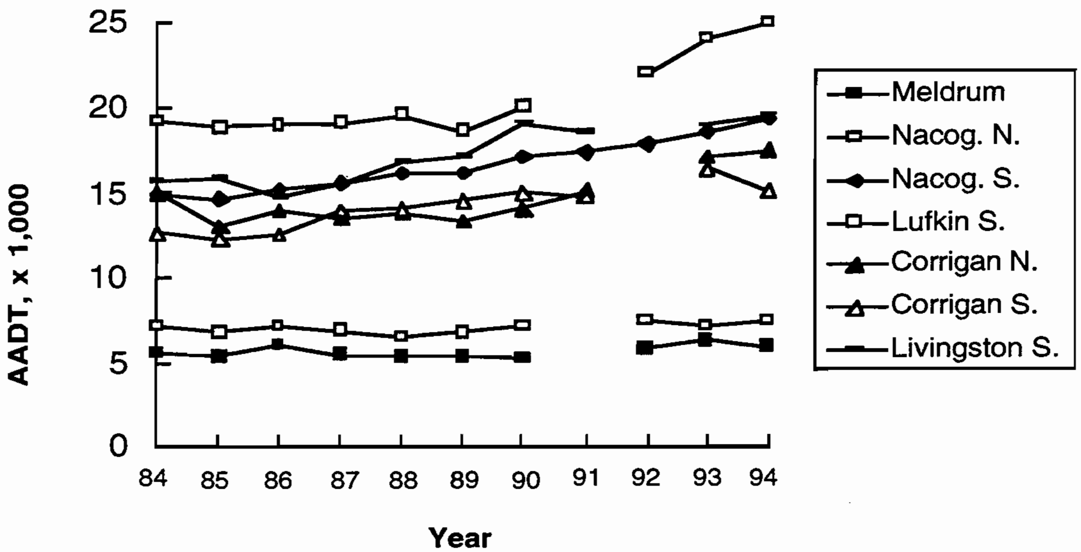

Figure 3.6 Historical US 59 AADT volumes

Table 3.3 Annual growth rates experienced on US 59 (1984–1994)

From	To	Annual Growth Rate (%)
Atlanta District Border	Nacogdoches	1
Nacogdoches	Lufkin	2.9
Lufkin	Diboll	3.0
Diboll	Corrigan	2.5
Corrigan	Livingston	2.7
Livingston	Beaumont District Border	2.8

Analysis of the classification trends for US 59 over the past 10 years indicates there has not been any significant growth in any of TxDOT's vehicular classification classes. There are differences from year to year (Figs 3.7 and 3.8), but no discernible trend is evident for the entire period. Therefore, the traffic loading forecast will assume a constant vehicle distribution among the traffic classes.

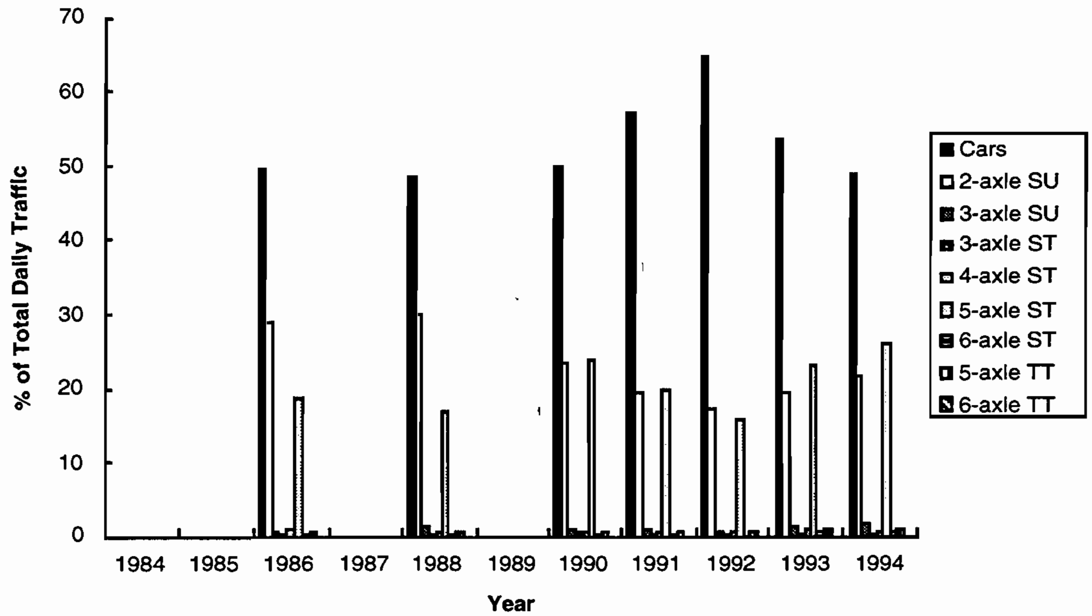

Figure 3.7 Historical traffic classification, northbound US 59 Polk, Texas

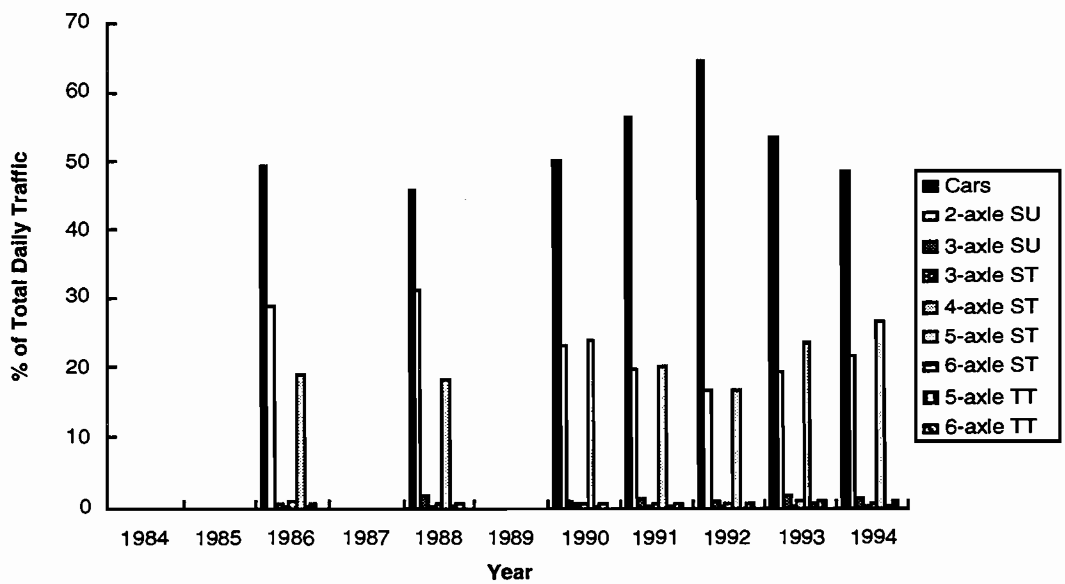

Figure 3.8 Historical traffic classification, southbound US 59 Polk, Texas

SUMMARY

In order to provide the best estimate of the gross-vehicle weight, axle-spacing, and speed for each vehicle, the WIM systems were calibrated in September 1995. The resulting adjustments were generally less than 2 percent for the load measurements and less than 3 percent for the axle spacing/speed measurements, reflecting the changing site conditions. A 4-percent shift in the calibration at the south site was documented by analyzing the gross vehicle weight distribution patterns for 3S2 tractor semi-trailer trucks. The 3-percent difference in axle spacing measurements was determined to not have a major impact on the classification of vehicles. Utilizing infrared light-beam sensors, both the lateral position and the frequency of tires on an axle were measured for every vehicle traversing the WIM systems. Analysis of historic TxDOT traffic data indicated an annual traffic volume growth rate of approximately 3 percent for the majority of the US 59 corridor, with a small 1-percent growth rate between the Atlanta District border and the intersection of US 59 and US 259. Traffic volume classification trends on US 59 at Polk, Texas, were also analyzed from the TxDOT data; no discernible annual trends were determined for the individual vehicle classes. Accordingly, an average classification distribution was used to represent US 59 throughout the study period.

CHAPTER 4. PROJECT RESULTS

TEMPERATURE

In addition to the traffic data collected, two thermocouples registered both the ambient air temperature and the pavement temperature at hourly intervals. These data have significance in evaluating how the environment impacts the pavement condition, and also in predicting the cumulative expected temperature-induced effects on future pavement designs. Temperature change causes volume change in asphalt concrete and in portland cement concrete, and also sometimes induces freezing and thawing of the roadbed soil. According to criteria stated in the AASHTO Design Guide, US 59 is located in temperature Region A, where "Low temperatures are not a problem but stability at high temperature should be considered." US 59 also borders both moisture Region I and Region II — Region I having a "High potential for moisture presence in the entire pavement structure throughout the year," and Region II having "Seasonal variability of moisture in the pavement structure."

It has been shown for flexible pavements that, as the surface temperature increases from 25°C (77°F) to 49°C (120°F), the relative fatigue damage caused solely by pavement strain decreases (Ref 18). However, the absolute levels of rut depth increase, such that a single 8.2 Mg (18-kip) single axle load application on 49°C (120°F) pavement represents up to 17 times more damage than the same axle load application on a 25°C (77°F) pavement (Ref 18). A temperature gradient within a rigid slab will increase fatigue damage. Rigid pavement slabs with a surface temperature higher than that at the base can curl downward at the edges, creating additional tensile stresses within the interior bottom surface of the slab when axle loads are superimposed, thereby increasing fatigue damage. Similarly, the opposite temperature gradient can cause additional tensile stress in the top surface of the slab when the edges and corners curl upward and wheeloads are applied at the edge.

The average minimum hourly pavement temperature was much warmer than the average minimum air temperature for every month of 1994 (Fig 4.1), a finding that underscores the short cold spells that are characteristic of this area. The average maximum pavement temperature was also greater than the average maximum air temperature, as the intensity of solar radiation often heats the asphalt concrete pavement for many hours during the months between March and September. During the month of July the asphalt concrete pavement reached an average maximum temperature of 47°C (117°F), while the average maximum air temperature was 43°C (109°F). In all, the average monthly maximum pavement temperature was in excess of 38°C (100°F) for five consecutive months, May through September. In contrast, during January the minimum air temperature was 3°C (37°F), while the minimum pavement temperature reached only 8°C (47°F). Pavement and air temperatures varied by approximately 21°C (70°F) over the course of the year.

These temperature data are important to the pavement design process, as temperature fluctuations generally have a negative impact on the performance of pavements and must be taken into account. The winter data are of particular importance to this project, as they indicate that there are few, if any, freeze-thaw cycles to weaken the subgrade at these sites.

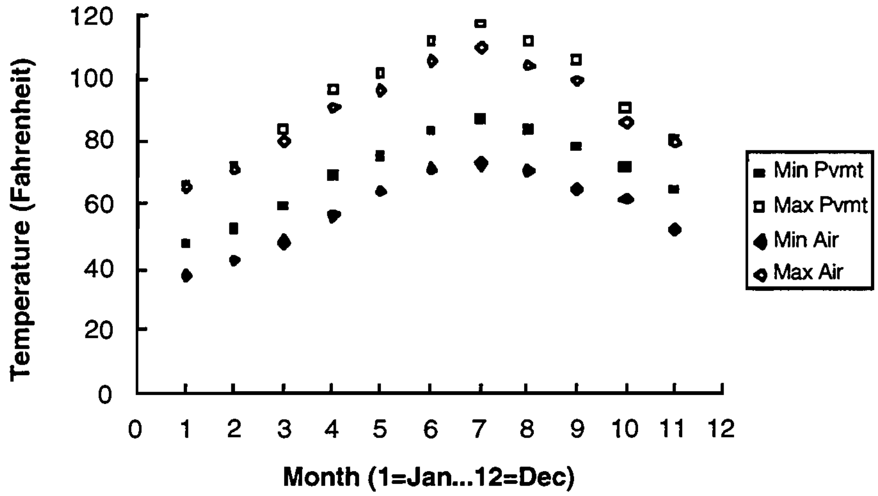

Figure 4.1 Average monthly temperatures, 1994 (°C=[°F-32]/1.8)

The 1995 temperature data (Fig 4.2) are included for comparison. As can be seen, the same temperature patterns prevailed, indicating the consistency of these data. Temperature data for 1993 are reported elsewhere (Ref 8).

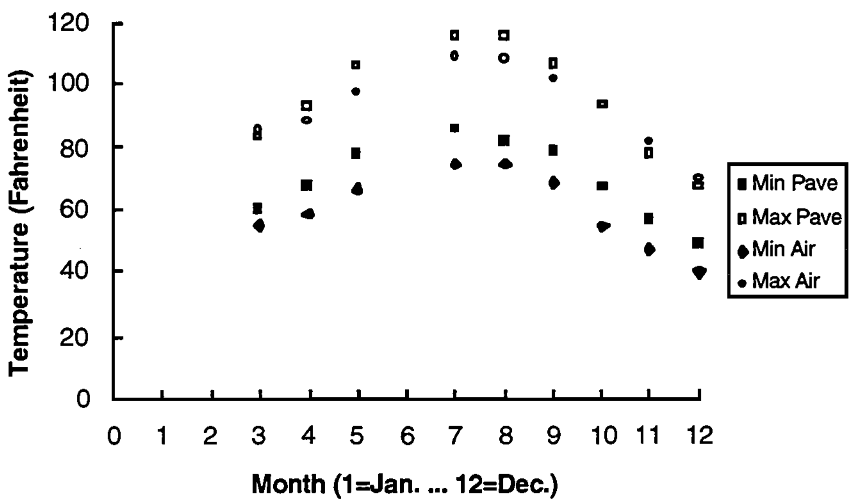

Figure 4.2 Average monthly temperatures, 1995 (°C=[°F-32]/1.8)

VEHICLE CLASSIFICATION

During the initial stages of this project, we expected that a simple vehicle classification scheme based on the number of axles per vehicle — referred to as the axle frequency vehicle classification scheme — might be appropriate. Therefore, much of the data was analyzed in terms of five vehicle classes: 2-, 3-, 4-, 5-, and 6-or-more axle vehicles; in addition, a sixth class for 2-axle trucks was added in 1994. A shortcoming of this scheme is the ambiguity within the 2 and 3-axle classes, as both passenger cars and pickups with trailers are combined with large 2- and 3-axle trucks or buses. Although data analysis is simplified and the number of unclassifiable vehicles is reduced using this axle frequency vehicle classification scheme, applying existing classification counts is difficult. However, in order to maintain continuity in this project, most of the traffic volume trends will be summarized using this axle frequency vehicle classification scheme.

A question that has been posed throughout this project is whether the WIM data being collected were reliable. A number of reliability checks have been exercised throughout the project to identify errors that might be occurring. One reliability check has been a simple visual examination of samples of real-time data records. Items such as axle spacings, speed, front-axle loads, and other items were examined for reasonableness. Although this check is both quick and simple and can catch obvious errors as they occur, the procedure examines only a small amount of data and is incapable of catching sporadic or intermittent problems.

Other types of reliability checks have been performed after data processing and, thus, may have allowed a significant period of time to pass before identifying a problem. Vehicle classification patterns, which are expected to remain fairly consistent over time, were examined periodically. For example, 17 continuous weeks of data were plotted in two separate formats. One format is an overlay plot of the daily traffic volume by day of the week (Fig 4.3). The traffic volume pattern for this site is clearly evident in this plot, and outliers are easily identified — for example, Labor Day (Sept. 5) and Independence Day (July 4). If outliers did not have a reasonable explanation, the data point(s) in question were discarded in order to eliminate bias in the data set.

Another format is a line plot of the daily volumes of each vehicle class. As can be seen in Figure 4.4, the patterns in this plot are also very consistent, with outliers easily identified.

Although the axle frequency vehicle classification scheme was used to check the reliability of the data and used to determine traffic volume trends, a new classification scheme that resembles the vehicle classes used in TxDOT's 24-hour average classification counts was developed. Using vehicle class definitions outlined in the Traffic Monitoring Guide and the vehicle classes used in TxDOT classification counts for guidance, we developed a series of 15 vehicle classes to summarize the US 59 traffic stream. Of the 15 classes, two are for not-identified 5-axle vehicles and not-identified 6-axle vehicles. Owing to the large variance in axle configurations for vehicles having more than 6-axles, all vehicles with between 7 and 11 axles are placed in one class; these vehicles comprise less than 0.5 percent of the daily traffic. Owing to the software limitations of the DAW100, vehicle records are limited to a maximum of 11 axles; data for additional axles are discarded when using this version of the software.

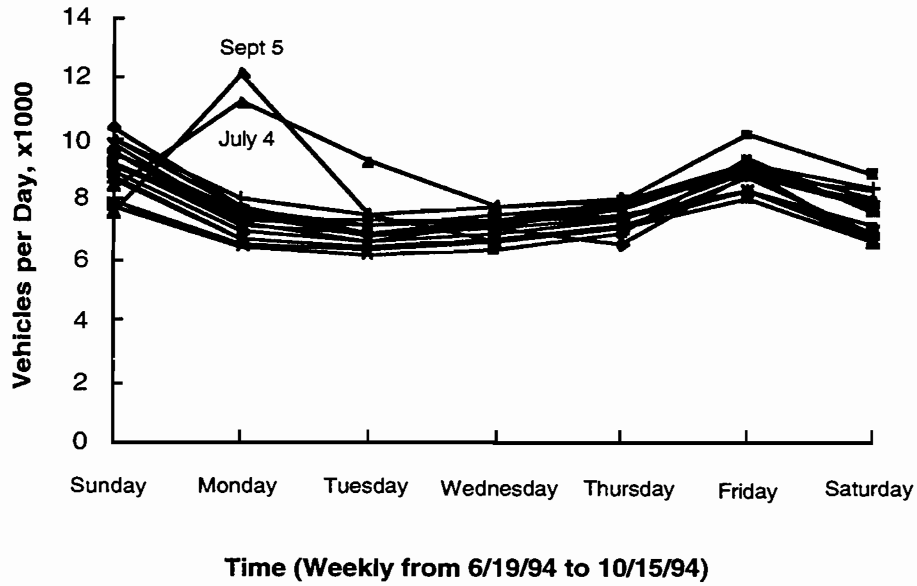

Figure 4.3 Daily traffic volumes — north site

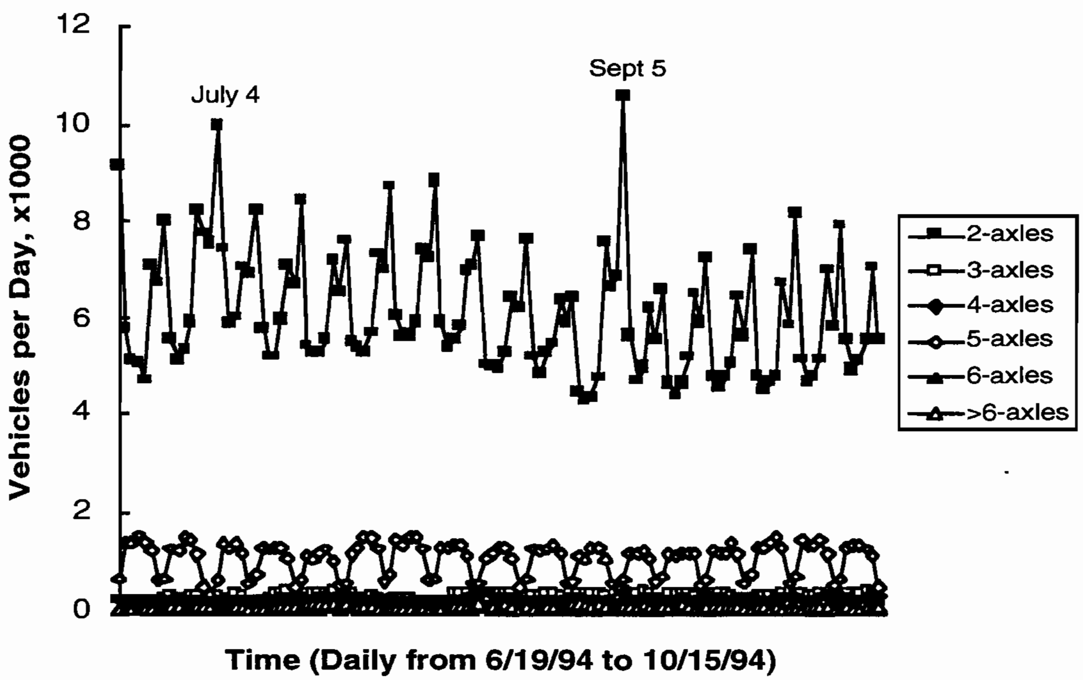

Figure 4.4 Daily traffic volumes by number of axles — north site

A description of the 15 classes is provided below, along with a listing of the range of axle spacing between axles (Table 4.1). The final five digits, 20131 in this case, identify the date and site. The first digit represents the site number (2) while the second and third represent the month (01 or January) and the last two represent the day (31) of the respective month.

2sx20131: o-o	2-axle passenger cars, motorcycles, or light-duty pickups with axle spacings less than 3.3 m (11 feet), with or without a recreational or light-duty trailer.
2st20131: o---o	2-axle trucks (or buses) with an axle spacing in excess of 3.3 m (11 feet), with or without a recreational or light-duty trailer.
3su20131: o--oo	3-axle single-unit vehicles.
3st20131: o--o----o	3-axle truck (2S1) consisting of a 2-axle tractor and single-axle semi-trailer.
4s120131: o--o----oo	4-axle truck (2S2) consisting of a 2-axle tractor and tandem-axle semi-trailer.
4s220131: o--oo----o	4-axle truck (3S1) consisting of a 3-axle tractor and a single-axle semi-trailer.
4tt20131: o--o----o----o	4-axle truck (2S1-1)consisting of a 2-axle tractor and two single-axle semi-trailers.
51t20131: o--oo----oo	5-axle truck (3S2) consisting of a 3-axle tractor and a tandem-axle semi-trailer.
52t20131: o--o---o-o---o	5-axle truck (2S1-2) consisting of a 2-axle tractor, a single-axle semi-trailer, and a trailer with two single axles.
53t20131: o--oo---o-o	5-axle truck (3S2 Spread) consisting of a 3-axle tractor and a semi-trailer with two spread single axles.
50t20131:	5-axle truck not classifiable above.
6st20131: o--oo----ooo	6-axle truck (3S3) consisting of a 3-axle truck and a tridem-axle semi-trailer.
6tt20131: o--oo---o-o---o	6-axle truck (3S1-2) consisting of a 3-axle tractor, a single-axle semi-trailer, and a trailer with two single axles.

60t20131: 6-axle truck not classifiable above.

shv20131: A truck with 7 to 11 axles.

A significant number of 5-axle semi-trailer trucks with spread single axles on the semi-trailer were observed at the WIM sites. A separate vehicle class (53t) was established for analyzing the axle loading patterns, as the spread axles are considered single axles rather than tandems in ESAL calculations. An axle spacing pattern resembling a 2-axle tractor, a single-axle semi-trailer, and a second single-axle semi-trailer was observed daily. Although it is not apparent what vehicle type is responsible for this axle spacing pattern, it was coded as "4tt" and is described above. This vehicle class was responsible for only about 0.25 percent of the daily traffic, but it did contribute approximately 0.75 percent of the overall ESALs generated.

Table 4.1 Range of spacing between axles (adapted from Ref 3)

Vehicle Class	Range of Spacing Between Axles (ft)*					
	1 - 2	**2 - 3**	**3 - 4**	**4 - 5**	**5 - 6**	**etc.**
2sx	6 - 11	*	*			
2st	11 - 40	*	*			
3su	8 - 26	2 - 6				
3st	8 - 20	11 - 45				
4s1	8 - 20	2 - 6	11 - 45			
4s2	8 - 20	11 - 45	2 - 6			
4tt	8 - 23	11 - 36	6 - 35			
51t	8 - 25	2 - 6	11 - 55	2 - 6.5		
52t	8 - 20	11 - 36	6 - 20	6 - 35		
53t	8 - 25	2 - 6	11 - 55	6.5 - 20		
50t	**** Other 5-axle Vehicles ****					
6st	8 - 20	2 - 6	11 - 42, (2 - 6)	2 - 6, (11 - 42)	2 - 6	
6tt	8 - 20	2 - 6	11 - 30	7 - 15	11 - 25	
60t	**** Other 6-axle Vehicles ****					
shv	**** Other 7-11 axle Vehicles ****					

*1 foot=0.304 m

Slightly more than 98 percent of the 1995 daily traffic at the south site was accounted for by the 12 identifiable classes. As stated previously, less than 1 percent of the initially recorded records were deemed to contain erroneous data and were separated into a separate error file. On average for 1995, only 0.21 percent of the traffic comprised 5-axle vehicles that could not be classified into one of the typical 5-axle vehicle classes, while less than 0.06 percent of the traffic comprised 6-axle vehicles that were not classified into one of the two typical 6-axle vehicle classes. Vehicles consisting of between 7 and 11 axles accounted for only 0.4 percent of the traffic volume on average. The relative contribution of each vehicle class towards the average annual daily traffic for 1995 is included graphically below (Fig 4.5), with contributions in excess of 1 percent indicated on the chart.

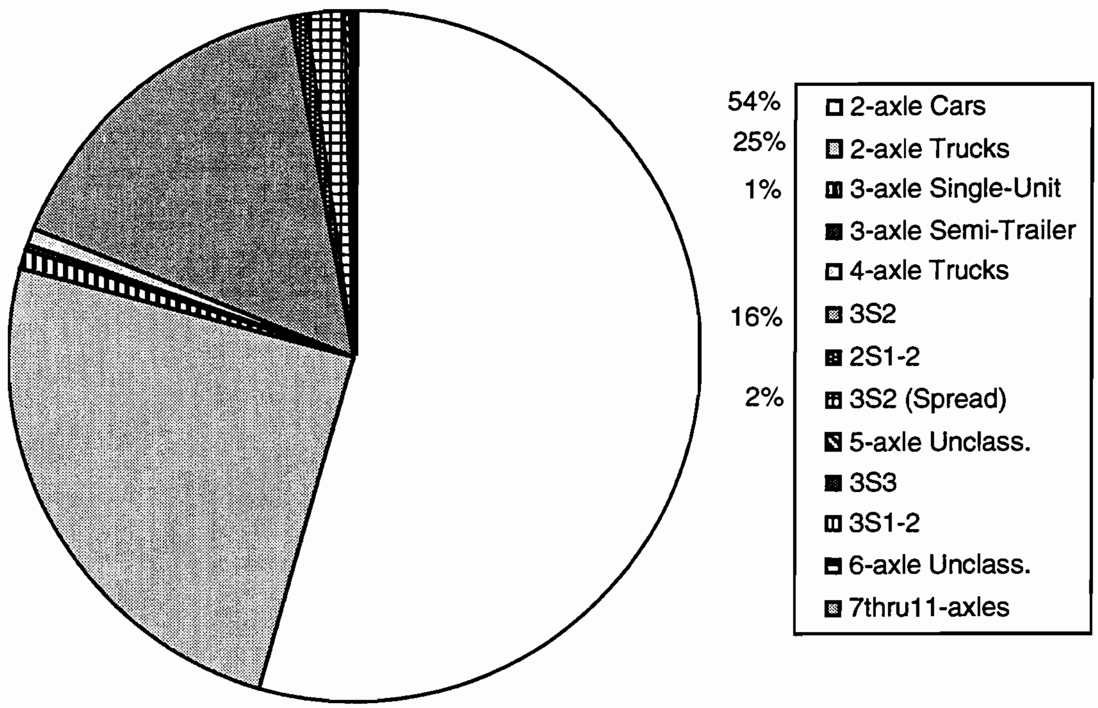

Figure 4.5 Average daily vehicle classification — south site, 1995

Since TxDOT's classification counts are performed visually, there were cases in which the current WIM data could not be separated into the same categories. Generally, the difference occurs when the TxDOT vehicle classification scheme distinguishes between the number of tires on a vehicle, such as the 2-axle 6-tire class. Although the special WIM systems used during this project were able to distinguish the number of tires on an axle via the infrared light-beam system, such data would not normally be available from WIM or automated classification systems; therefore, it was not incorporated into the vehicle classification analyses. Classes that distinguished between the number of tires on a vehicle were combined with classes that had the same axle spacings and groupings. For example, the TxDOT 2-axle 6-tire class was included in the 2-axle truck class.

The optional Federal Highway Administration (FHWA) Class 1 is reserved for motorcycles. It is left to the state's discretion whether or not to report this class to FHWA. This class of vehicle, which has minimal impact on pavement performance and typically occurs at a low frequency, was not recorded by the WIM system as a separate vehicle class. If the front axle load on a Class 1 vehicle was in excess of the 0.230-Mg (500-lb) threshold set in the WIM systems, the vehicle was included in the 2-axle passenger car class.

In the vehicle classification scheme presented herein, the axle spacing criteria were taken from Table 4 of ASTM E 1318 - 94 (Ref 3), which quantifies an axle spacing criteria for using the general FHWA vehicle classes. The ASTM data were used, since "FHWA does not endorse 'Scheme F' or any other classification algorithm" (Ref 19).

Front-axle loads were utilized to further refine the classification and to provide a validity check for each record. The front-axle load helped distinguish 2-axle single unit vehicles pulling recreational or other light trailers from 4-axle tractor semi-trailer trucks. Generally a 2.3 Mg (5-kip) front-axle load criterion was used as a load threshold. The Traffic Monitoring Guide (Ref 19) recommends classifying a passenger car, or "other 2-axle 4-tire single unit vehicle," pulling a recreational or light-duty trailer, as either a passenger car, or "other 2-axle 4-tire single unit vehicle," respectively. Classifying the vehicles simply by the number of axles had the adverse effect of placing these light-duty vehicles with light-utility trailers in the same class as 3- or 4-axle tractor semi-trailer trucks.

TRAFFIC TRENDS

As mentioned previously, an overall objective of this project is to establish a methodology for estimating axle loads from traffic survey data, which do not necessarily include observed axle loads. Therefore, most of the traffic trends will be presented according to vehicle types that are related to the number of axles per vehicle.

Overall traffic trends are presented in various formats, including summarized daily, weekly, and monthly observations. Seasonal trends are evident after examining the daily traffic volumes from 1993 through 1995 (Fig 4.6). Most noticeable are the two peaks for the Thanksgiving and Christmas holiday seasons, where the daily volumes exceed 60,000 vehicles. The low volume before January 1995 was associated with the October 1994 flooding, which closed portions of US 59. A slight increase in traffic volumes can be seen during this three-year period. Lower traffic volumes at the south site are also evident throughout the period and can probably be attributed to the proximity of the site to Corrigan, Texas, and also to the presence of the US 287 intersection between the two sites. The 1993 and 1994 traffic volumes presented here include traffic volume data that have been estimated by the methods previously described in Chapter 3, while 1995 traffic volumes are as-collected (missing data have not been replaced with estimated values).

For further analysis, the observed weekly traffic volumes are shown in Figures 4.7 and 4.8. These plots, which indicate annual traffic volume patterns at each site, can aid in making traffic volume forecasts for longer periods of time.

A chart of the monthly traffic volumes (Fig 4.9) is also included to indicate the month-to-month variability in traffic volumes. From data shown in this chart, seasonal adjustment factors can be determined for estimating traffic volumes from short-period traffic surveys.

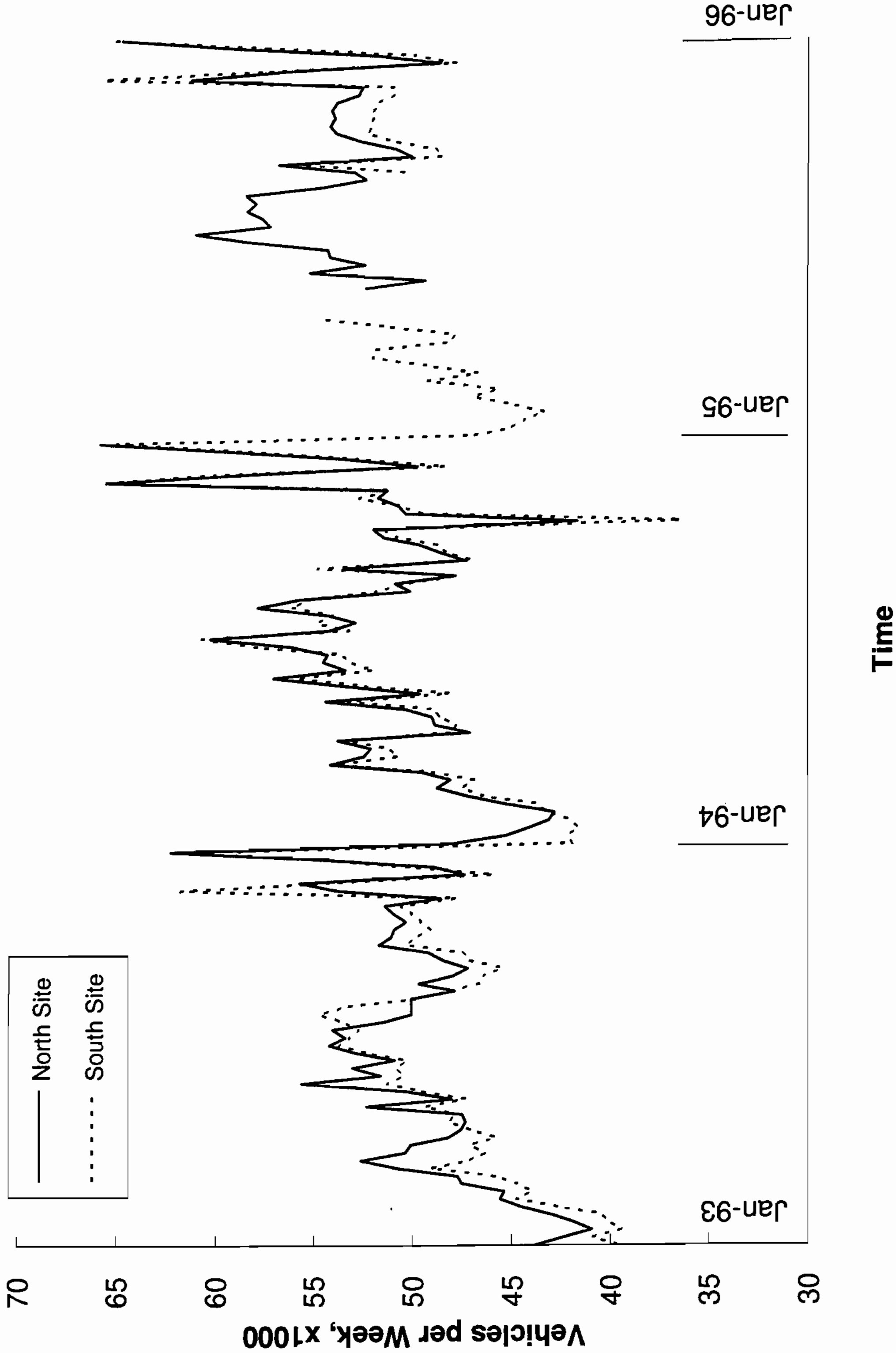

Figure 4.6 Daily traffic volumes, 1993 through 1995

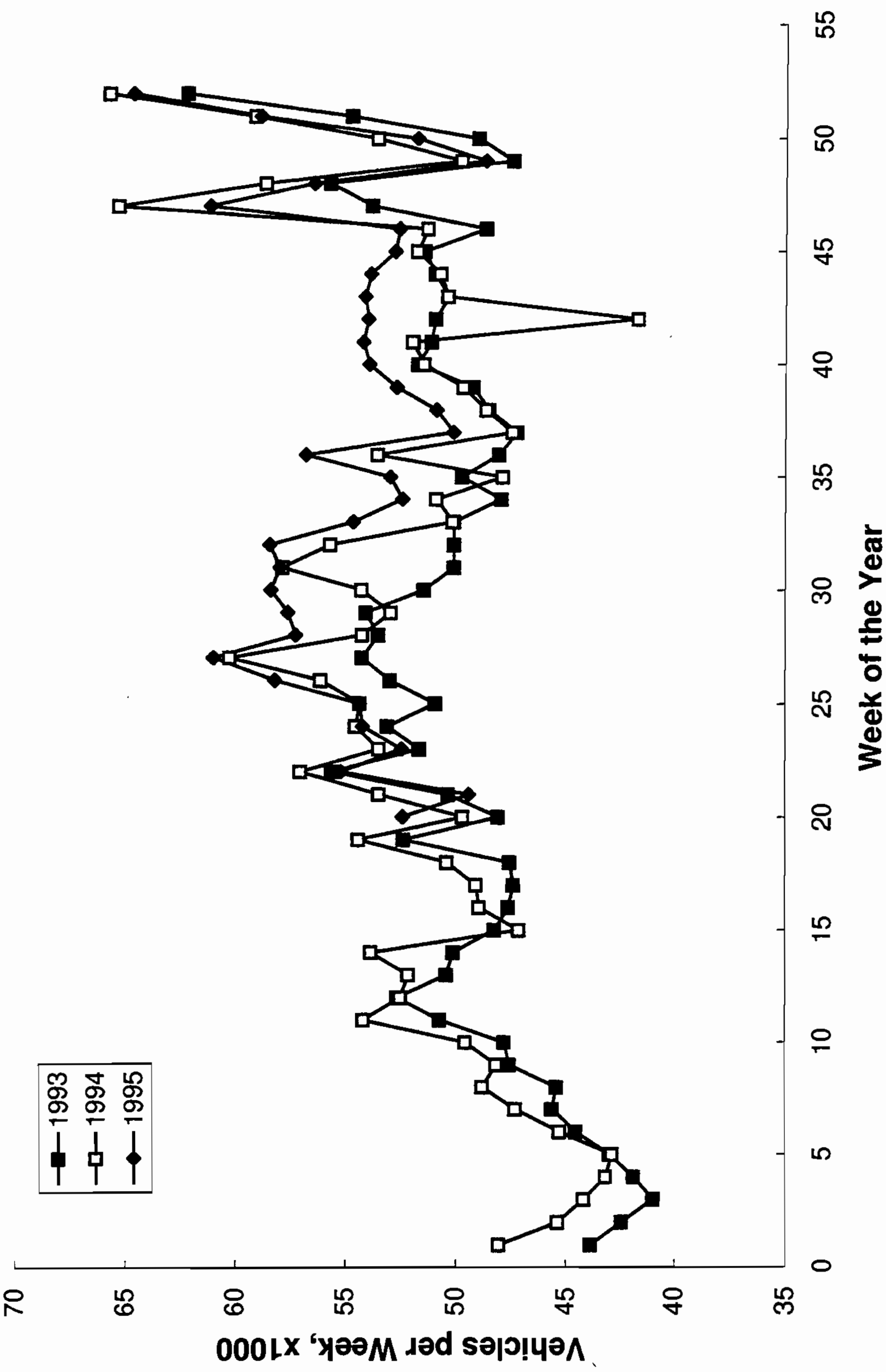

Figure 4.7 Weekly traffic volumes — north site, 1993 through 1995

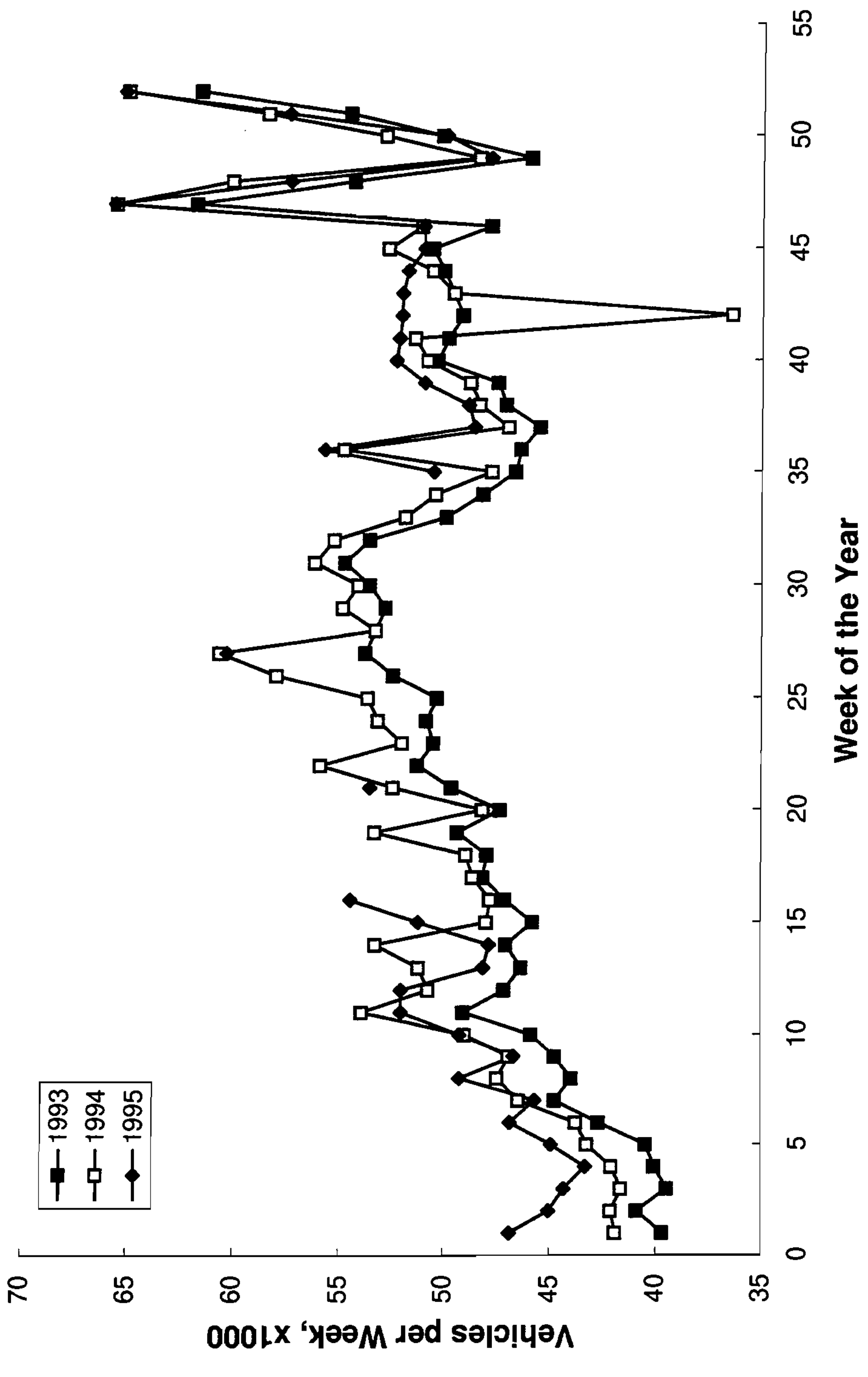

Figure 4.8 Weekly traffic volumes — south site, 1993 through 1995

Traffic volume variances between lanes are documented elsewhere (Ref 8). It was reported that 75 percent of the total traffic volume was carried in the right-hand lane, of which 76 percent represented 2-axle vehicles, while the traffic in the left-hand lane consisted of 89 percent 2-axle vehicles.

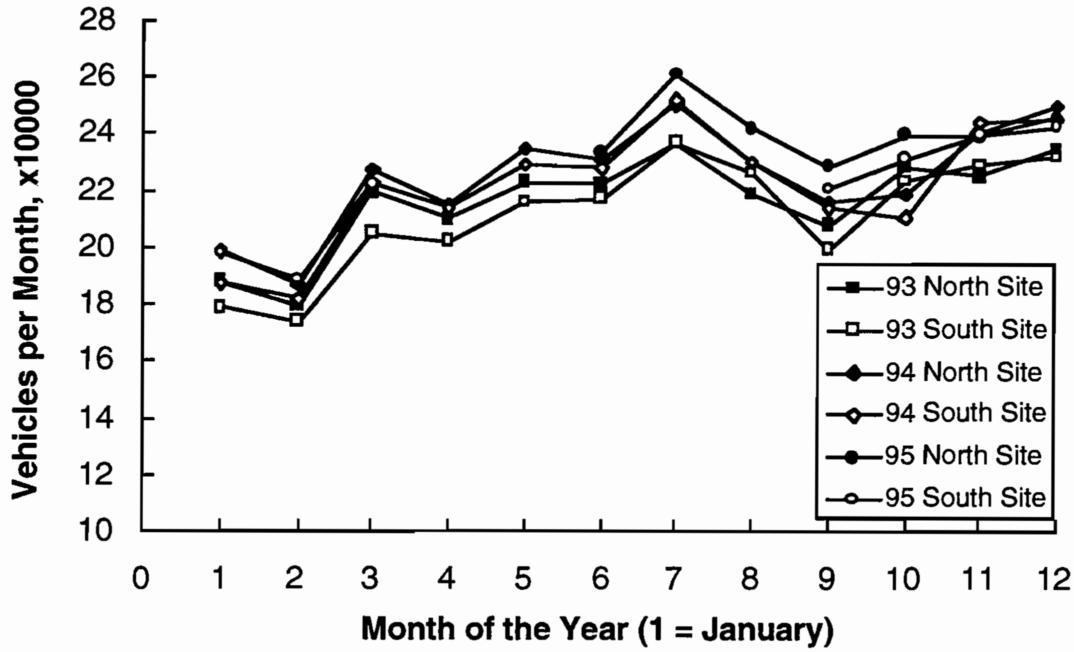

Figure 4.9 Monthly traffic volumes, 1993 through 1995

LATERAL POSITION

The lateral positioning of traffic wheel loads on pavements has been investigated for years, and the results of an extensive study are presented by Lee et al. (Ref 10). The use of lateral placement data in pavement design includes not only the average placement of the loads from the lane edge, but also the variance of the placement. In terms of pavement performance, a greater variance is preferable, as the repetition of load is distributed over a greater area of pavement. Such variance is desired on flexible pavements, where rutting caused by consistent, channelized wheel loads can result. On rigid pavements, wheel loads applied near the lane edge cause higher stresses than those applied further toward the center, or interior, of the lane. Lee et al. show an example in which taking the lateral distribution of the wheel loads into effect allowed a reduction in rigid pavement design thickness of almost 25 mm (1 inch), as compared with designing the same rigid pavement using normal edge-loading design assumptions for all traffic. The edge-loading design resulted in a 15 percent greater pavement thickness, which indicates considerable savings in rigid pavement thickness can be realized by incorporating the lateral distribution of the traffic loading into structural design procedures.

An analysis of the data collected during March 1995 at the south site (Figs 4.10 and 4.11) indicates the normally distributed nature of the lateral wheel position data. From these distributions, the differences in lateral position between vehicle type and between lanes can be easily identified. The 2-axle vehicles traveling in the right-hand lane had a median lateral position of about 0.9 m for the outside wheels, while trucks traveled slightly closer to the edge at a median position of about 0.76 m. The variability in the lateral position of the outside wheels on 2-axle vehicles was also slightly greater, with an 85th-percentile lateral position of 1.4 m from the edge of the right-hand lane, while trucks had an 85th-percentile lateral wheel position of about 1 m. Approximately 73,500 2-axle vehicles and 22,500 trucks were used in the right-hand lane analysis.

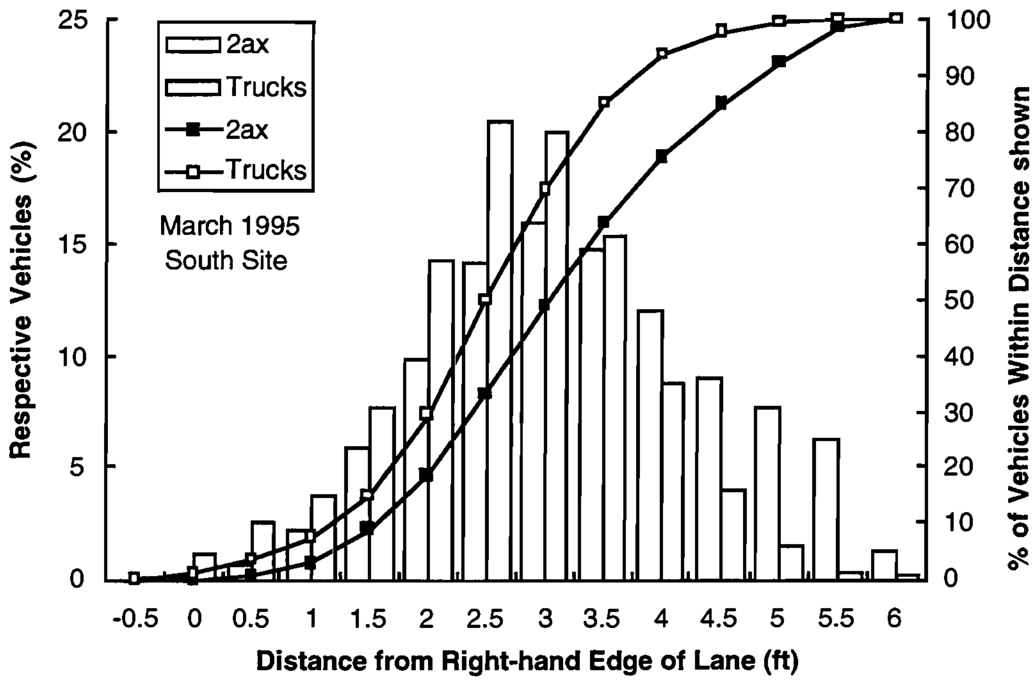

Figure 4.10 Lateral position of outside wheels on 2-axle vehicles and trucks in right-hand lane

The lateral position distributions for the outside wheels on 2-axle vehicles and trucks in the left-hand lane were slightly more concentrated than those in the right-hand lane. The 2-axle vehicles had a median lateral position between 0.76 and 0.9 m from the left-hand edge of the lane. However, trucks traveled closer to the outer edge, with a median lateral position of about 0.45 m from the left-hand edge of the lane. This indicates that, although the left-hand lane may carry a much smaller percentage of the total loading on a highway, the loading in that lane is placed closer to the edge of the lane, which tends to induce higher pavement stresses. The 85th-percentile lateral position of the truck wheels was 0.76 m from the left-hand edge of lane, while the 2-axle vehicles was 1.2 m.

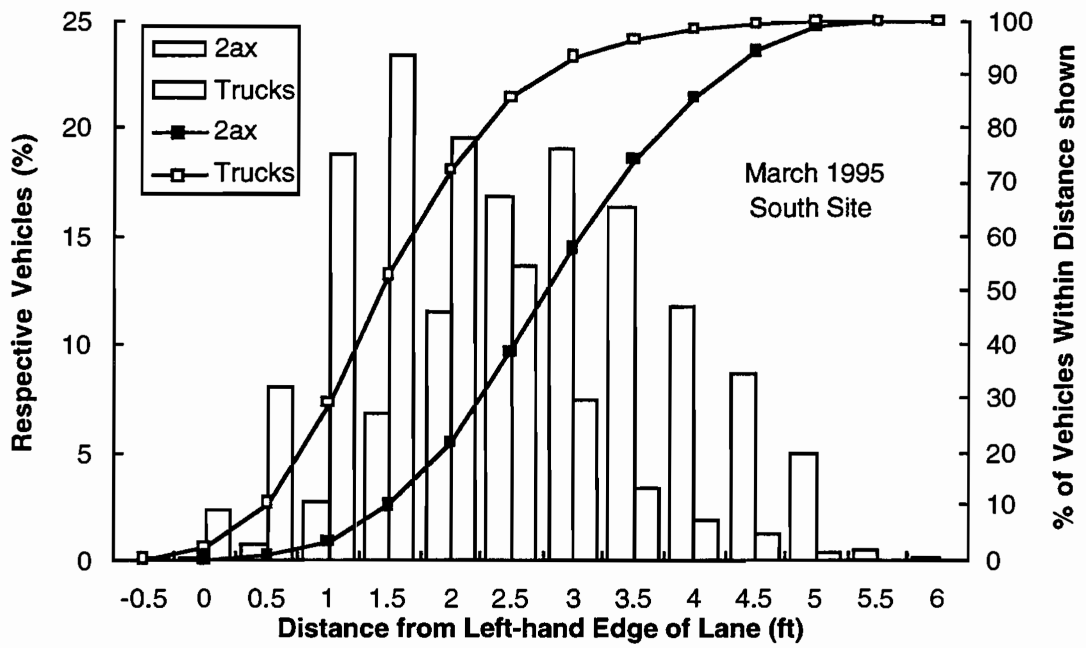

Figure 4.11 Lateral position of outside wheels on 2-axle vehicles and trucks in the left-hand lane

The tendency for vehicles in the left-hand lane to travel closer to the edge of the lane is probably due to the left-hand lane being used primarily for passing maneuvers, with vehicles in this lane allowing more clearance from a vehicle in the right-hand lane.

TIRE FREQUENCY

As mentioned previously in this report, the infrared light-beam sensors were used to identify axles with dual tires. This type of data is not currently used in routine vehicle classification, but it offers another level of detail for categorizing traffic data for analysis.

Although the light-beam data indicate whether single or dual tires are on each observed axle, the analysis software written for this project indicates only whether or not a vehicle has at least one axle with dual tires. In a study of a sample of 5-axle semi-trailer trucks, approximately 98 percent of the trucks were correctly identified as having one or more axles with dual tires. The 2 percent not identified correctly included trucks that had missing infrared data or infrared readings that were obviously false. These problems were probably due to vehicles changing lanes or to the inadequacy of the processing speed of the DAW100 for handling all the input data during the time-gaps between successive vehicles when traffic was concentrated. A symptom of the processing-speed problem was that, occasionally, the infrared readings were not recorded for the last axle of a multi-axle vehicle, or the system recorded a false measurement. A possible means of simplifying dual-tire data analysis would be to process the infrared data only for those vehicle classes that require it. For example, vehicles classified as 3S2 trucks via number and axle spacing could be assumed to have four dual-tire axles and a single-tire steering axle, without analyzing the infrared data. The 2-axle trucks, on the other hand, do not always have dual-tires on the rear axle, and the infrared data could be used to distinguish among the various 2-axle truck and bus classes.

SPEED DATA

Vehicle speeds appear to have a minimal impact on pavement performance, with higher speeds being slightly more damaging to rigid pavements and slightly less damaging to flexible pavements. Rigid pavements have no well-established damage mechanisms that are speed sensitive (Ref 18). Flexible pavements have exhibited a relationship between rutting and vehicle speed that favors higher speeds. The reduction in load application time with higher speeds has shown that rutting decreased 16 percent when speeds were increased from 55 mph to 65 mph (Ref 18). Even though vehicle speeds have not been incorporated into pavement design procedures, the speed data that have been collected as part of this project are reported here.

Vehicle speeds are recorded routinely by all WIM systems. These data should provide an accurate assessment of free flow speeds along a stretch of highway, as the sensors and instrumentation are quite inconspicuous.

Effect of 55 mph Speed Limit Change

On December 8, 1995, the federal government repealed the mandated 55 mph speed limit and returned the power to set speed limits to the individual states. Soon thereafter, Texas began to institute new speed limits of up to 70 mph on selected highways throughout the state. US 59 in the vicinity of Corrigan had the posted speed limit increased to 70 mph on January 5, 1996. A brief assessment of the speed changes that have been observed at the two WIM sites near Corrigan is presented here.

Data collected from November 29 to December 12, 1995, were used for the "before" case and consisted of about 70,000 vehicles at each site. Data collected from January 10–23, 1996, were used for the "after" case and consisted of approximately 66,000 vehicles at each site. It should be noted that the data used for the after analyses are only approximately two weeks older than the posted speed limit change. Evaluation of the long-term effects will require a later study.

Analysis of data from the north site (Fig 4.12) indicates that for the periods in question, there was an increase of approximately 2 mph in the median speed at the north site. During the December 1995 period, the median speed was approximately 59 mph, while during the January 1996 period, the median speed was approximately 61 mph. The greatest shift in the distribution curve occurred in the 50 mph to 70 mph range. Approximately 20 percent fewer vehicles were traveling slower than 60 mph in January, while the percent of vehicles traveling slower than 55 mph decreased from an already low 20 percent to only 10 percent. However, the percentage of vehicles traveling in excess of 70 mph remained fairly constant — approximately 5 percent of all vehicles, representing an increase of less than 2 percent. The 85th-percentile speed at this site increased to 67 mph.

Data from the south site (Fig 4.13) show speed patterns similar to those observed at the north site, though the speed is slightly slower overall. The lower speeds could be due to the location of the WIM system just downstream from an extended, gradual up-grade, and also to the fact that traffic is required to reduce speed through the town of Corrigan, which is about 3.2 km upstream from the WIM site. Analysis indicates a shift of approximately 2 mph in the median speed of all vehicles, from approximately 57 mph to 60 mph. The percentage of vehicles traveling below 55 mph was almost 30 percent and declined to approximately 18 percent in January. Again, there was only about a 2-percent increase in the number of vehicles traveling in excess of 70 mph (approximately 3 percent of all vehicles), while the 85th-percentile speed increased to 66 mph.

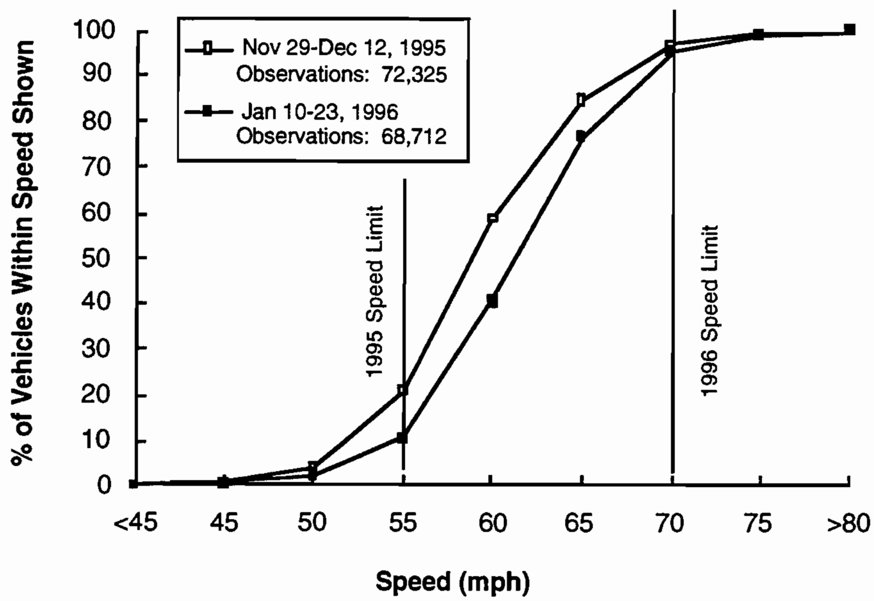

Figure 4.12 Speed distribution for all vehicles during periods indicated — north site

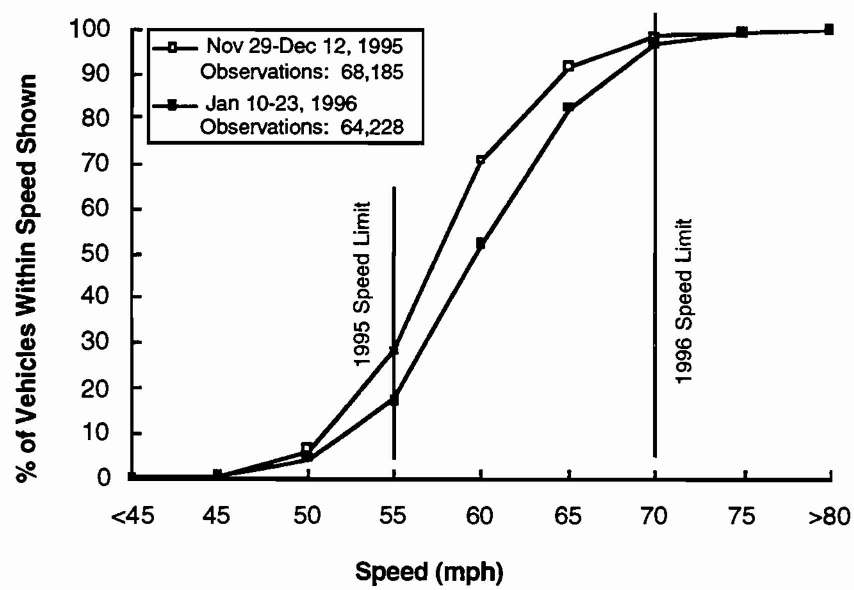

Figure 4.13 Speed distribution for all vehicles during periods indicated — south site

Preliminary analysis of the aforementioned speed data indicates a small, general increase in the speed of the overall traffic stream. Median speeds have increased approximately 2 to 3 mph, while the percentage of the traffic stream traveling below 55 mph has decreased to approximately 15 percent of all vehicles. However, the most encouraging aspect of the analyses is the very minor increase (approximately an additional 2 percent) of vehicles traveling in excess of 70 mph. Table 4.2 summarizes the important speed statistics for both sites.

Table 4.2 Summary of speed data statistics

	North Site	**North Site**	**South Site**	**South Site**
	Dec. 1995	**Jan. 1996**	**Dec. 1995**	**Jan. 1996**
Median Speed (mph)	59	61	57	60
85th Percentile Speed (mph)	65	67	63	66
% below 55 mph	21	10	29	17
% above 70 mph	3	5	1	3

Speed Characteristics

In addition to examining the effects of the increase in the posted speed limit on the composite traffic stream, we studied the speeds of 2-axle vehicles and trucks during both day (6 AM to 6 PM) and night (6 PM to 6 AM) periods. Data from the north site during November 29 through December 12, 1995, were used for this analysis. Approximately 52,500 2-axle vehicles and 19,500 trucks were contained in this sample.

It is evident that trucks, with a median speed between 59 and 60 mph, were traveling slightly faster than 2-axle vehicles, with a median speed of about 58 mph, during this period (Fig 4.14). However, both classes had the same 85th-percentile speed of approximately 65 mph.

The difference between daytime and nighttime speeds is basically negligible for 2-axle vehicles and very minor for trucks. The truck speed distribution curves indicate nighttime speeds that are slightly higher than those during the daytime, although there is basically no difference in the 85th-percentile speeds.

As can be expected, speeds in the right-hand lane are lower than those in the left-hand lane (Figs 4.15 and 4.16). Traffic speeds have been relatively consistent throughout the study period, with the apparent shifts during September 1995 caused by calibration of the WIM systems. The higher median speeds at the north site and also the larger speed differential between lanes at the north site are both noticeable. Again, this is probably due to the free-flow traffic conditions existing at the north site, where traffic has had ample time to assume a desired speed before going through Corrigan.

AXLE LOADS AND ESALS

Axle-group load frequency distributions were developed for each vehicle class previously defined within this chapter. In order to apportion the relative contribution to cumulative pavement damage of the mixed axle-group load distributions, equivalency equations were applied to convert the mixed loads into 8.2 Mg (18-kip) equivalent single axle loads (ESALs).

Background

The AASHO Road Test conducted during the late 1950s and early 1960s represents one of the most comprehensive pavement analysis projects ever completed. AASHO (now AASHTO, the American Association of State Highway and Transportation Officials) attempted to correlate the relative effects of various axle loads and configurations on the performance of test pavements and bridges constructed near Ottawa, Illinois. Various pavements, which were constructed in special loops to allow the continual travel of the test vehicles, consisted of both flexible and rigid pavement sections. The test vehicles traveled over 27,000,000 km and applied over 1 million axle loads.

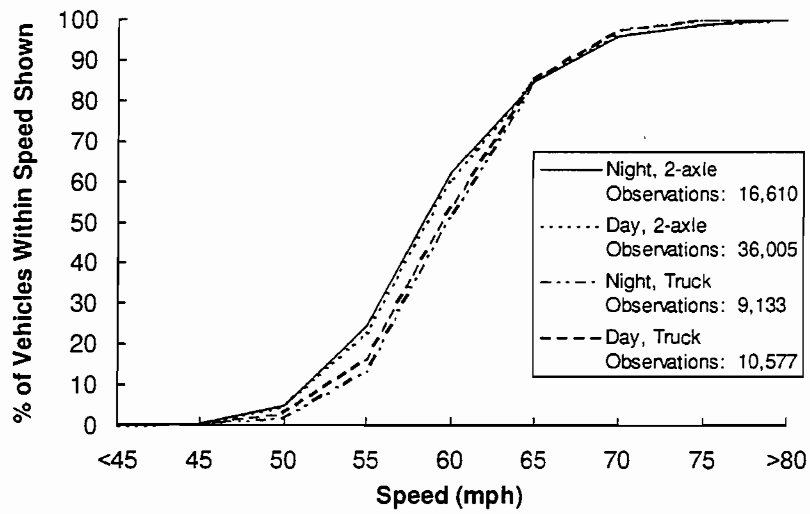

Figure 4.14 Vehicle speed distribution (Nov. 29 – Dec. 12, 1995) — north site

Using statistical procedures, the cumulative effects of the axle loads applied by the selected types of test trucks during the Road Test were distributed among the axle loads and axle groups. Separate equations were fitted to express the damage caused by single and tandem-axle groups, for various axle loads, on both flexible and rigid pavements, as an equivalent number of standard 18-kip single axle load applications. The 8.2-Mg (18-kip) single axle load acts as a common denominator so that potential pavement damage caused by the various axle loads from a mixed vehicle type traffic stream can be assessed. An axle load of 8.2-Mg (18-kip) on a single axle has an ESAL factor of 1.00, while an axle load greater than 8.2-Mg (18-kip) on the same single axle would result in an ESAL factor greater than 1.00. However, the difference in ESAL factors is not linearly proportional to the axle load difference; it varies by a fourth-power exponent, approximately. Thus, the doubling of an axle load will increase pavement damage by a factor of about 16.

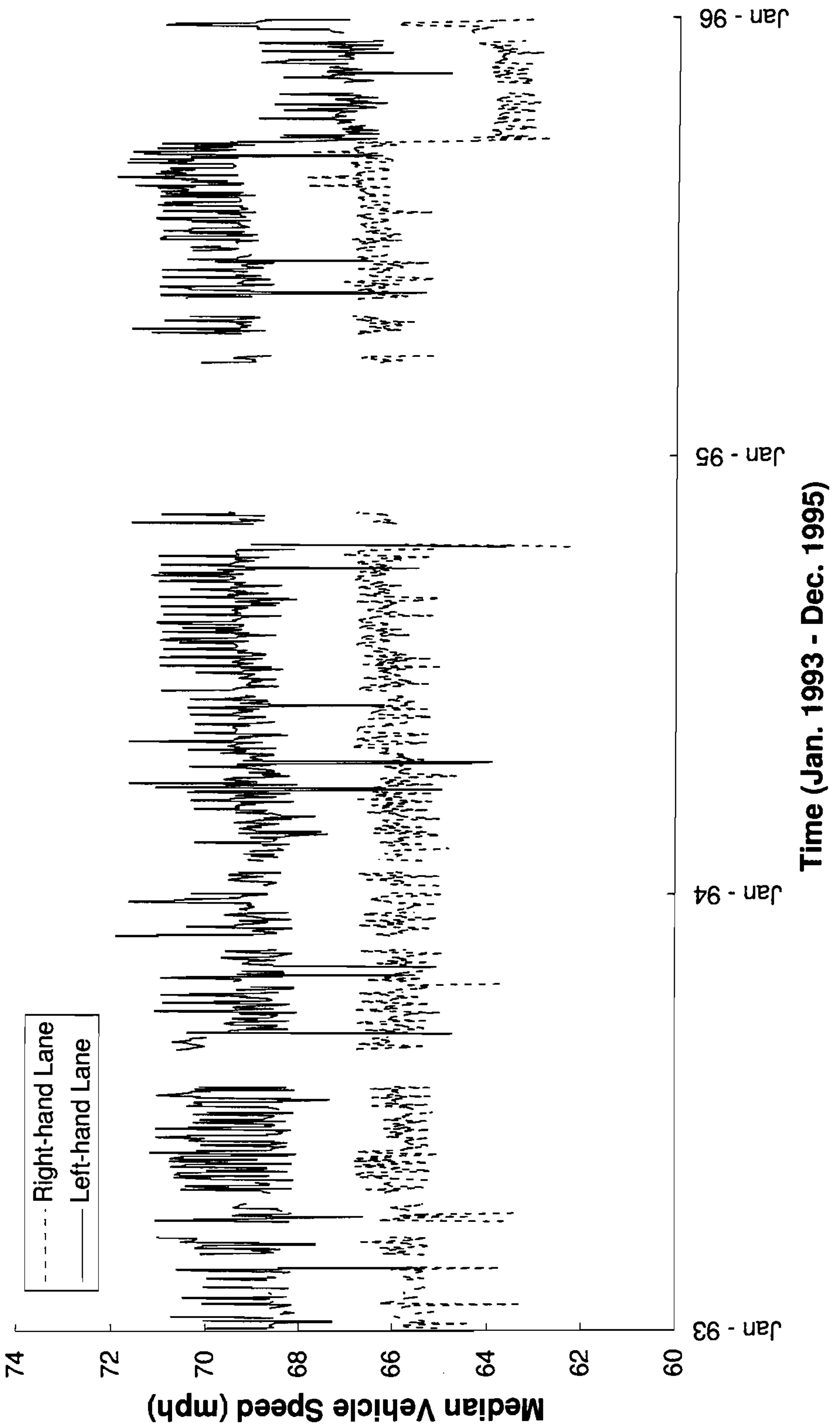

Figure 4.15 North site median traffic speeds — all vehicles, 1993 through 1995

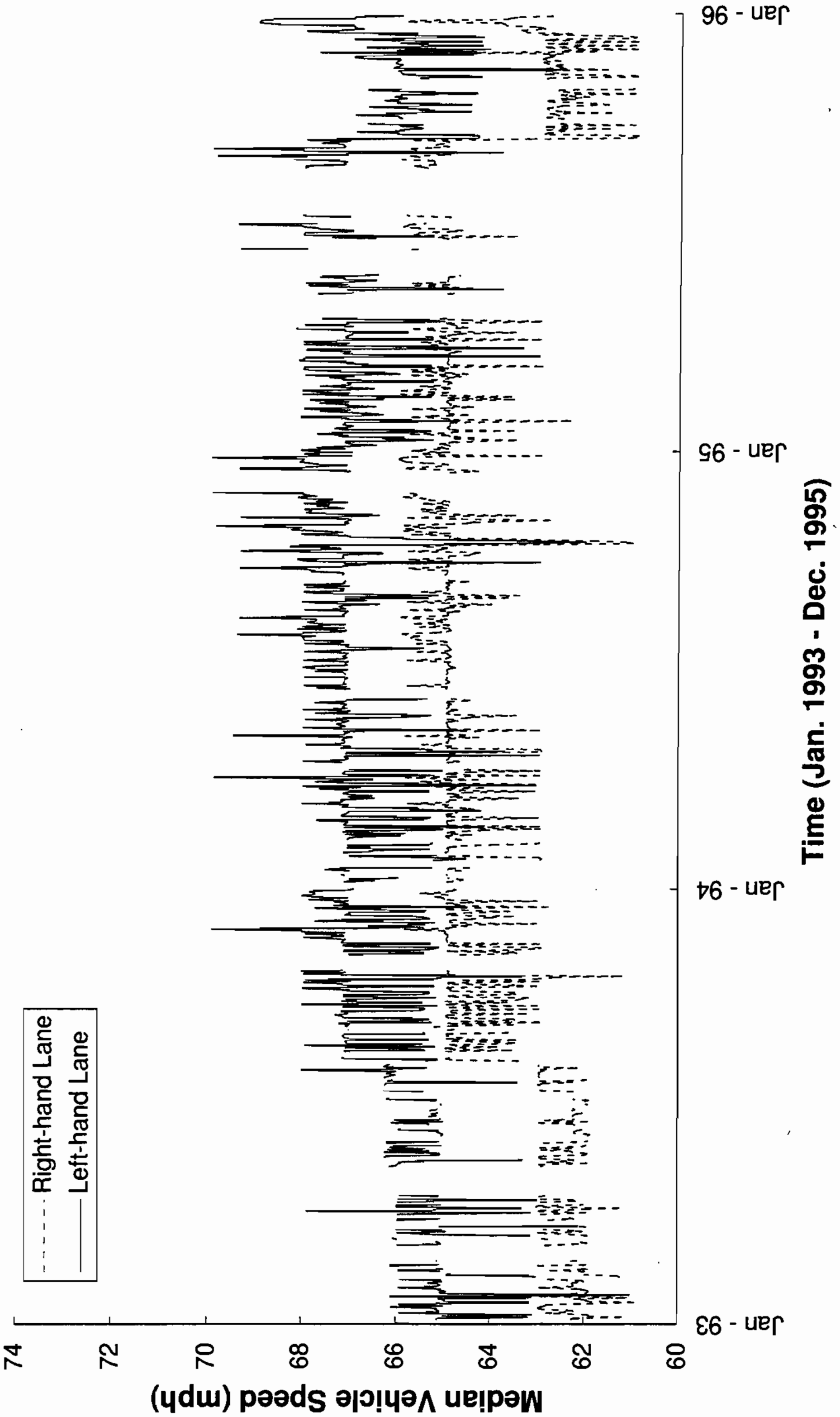

Figure 4.16 South site median vehicle speeds — all vehicles, 1993 through 1995

Axle loads for the AASHO test trucks ranged from 0.9–5.4 Mg (2–12 kip) on the steering axles, 0.9–13.6 Mg (2–30 kip) on single axles, and 10.9–21.8 Mg (24–48 kip) on tandem axles. The damaging effects of the steering axles were not analyzed separately but were incorporated into the single-axle and tandem-axle load factors. Research conducted by Izadmehr (Ref 9) utilizes previous work by Carmichael et al. that attempted to separately identify the damage caused by steering axles and to also develop ESAL factors for tridem-axle sets, which were not included in the Road Test. As stated above, steering axle loadings up to 5.4 Mg (12 kip) were used during the Road Test and their equivalent damage factor was incorporated into the damage factor of other axle groups. However, the damage caused by steering axle loads in excess of 5.4 Mg (12 kip) should be accounted for. Steering axle loads of 5.4 Mg (12 kip) are as damaging to a flexible pavement as an 9.1-Mg (20-kip) single axle with dual tires (Ref 18). Izadmehr presents a table of steering axle equivalencies developed by Carmichael et al. by axle load and terminal serviceability for flexible pavements; this is included here as Table 4.3. For this report, any steering axle load in excess of 5.4 Mg (12 kip) on a flexible pavement was accounted for by applying the appropriate equivalency factor from Table 4.3. Steering axle loads less than 5.4 Mg (12 kip) are accounted for by using the normal AASHTO axle-load equivalencies, which already incorporate the effect of the steering axle load. An attempt by Carmichael et al. to develop steering axle equivalencies for rigid pavements was unsuccessful; therefore, no additional steering axle load factor was incorporated for this report.

Table 4.3 Steering axle equivalencies by axle load and terminal PSI for flexible pavement (Ref 9)

Axle Load	P_t			
Kip*	**1.5**	**2.0**	**2.5**	**3.0**
2	0.0005	0.0009	0.002	0.004
4	0.008	0.01	0.02	0.03
6	0.04	0.05	0.06	0.09
8	0.13	0.14	0.18	0.23
10	0.28	0.31	0.36	0.41
12	0.52	0.54	0.62	0.66
14	0.92	0.86	0.93	0.94
16	1.42	1.31	1.33	1.28
18	2.12	1.94	1.90	1.74
20	2.95	2.52	2.44	2.16
22	4.02	3.35	3.15	2.70
24	5.29	4.40	3.95	3.28
26	6.73	5.49	4.82	3.89
28	8.31	6.67	5.83	4.59
30	10.19	8.05	6.80	5.23

1 kip=0.4536 Mg

The damage equivalency equations derived by AASHO after the conclusion of the Road Test are the same equations still in use today and are presented in the following sub-paragraphs.

Flexible Pavement Equivalence Factor Equations: The flexible pavement equivalency equations as developed from the AASHO Road Test, and described in ASTM E1318-94 (Ref 3), are as follows:

$$\log W_t = 5.93 + 9.36 \log(\overline{SN} + 1) - 4.79 \log(L_1 + L_2) + 4.331 \log L_2 + G_t / \beta$$

$$\beta = 0.40 + \frac{0.081(L_1 + L_2)^{3.23}}{(\overline{SN} + 1)^{5.19} L_2^{3.23}}$$

Where:

W_t = number of axle load applications at the end of time *t* for axle sets with dual tires,

$\overline{SN}$ = structural number, an index derived from an analysis of traffic, roadbed soil conditions, and regional factor which may be converted to a thickness of flexible pavement layers through the use of suitable layer coefficients that are related to the type of material being used in each layer of the pavement structure,

L_1 = load on one single axle, or on one tandem-axle set for dual tires, or on one tridem-axle set for dual tires, kip,

L_2 = axle code (one for single axle, two for tandem axle sets, three for tridem axle sets)

p_t = serviceability at end of time *t* (Serviceability is the ability of a pavement at the time of observation to serve high-speed, high-volume automobile and truck traffic.),

G_t = a function (the logarithm) of the ratio of loss in serviceability at time *t* to the potential loss taken to a point where p_t = 1.5, or

$$G_t = \log\left[(4.2 - P_t) / (4.2 - 1.5)\right], \text{ and}$$

β = a function of design and load variables that influences the shape of the P-verses-W serviceability curve.

The following equation expresses any axle load (W_t) in terms of the number of applications of the standard 8.2-Mg (18-kip) single-axle load (W_{18}) that would cause the same flexible pavement damage as the (W_t) axle load,

$$E_i = \frac{W_{t_{18}}}{W_{t_i}} \left[\frac{(L_i + L_2)^{4.79}}{(18+1)^{4.79}} \right] \left[\frac{10^{G_t/\beta_{18}}}{\left(10^{G_t/\beta_i}\right) L_2^{4.331}} \right]$$

Rigid Pavement Equivalence Factor Equations: The rigid pavement equivalency equations as developed from the AASHO Road Test, and described in ASTM E1318-94 (Ref 3), are as follows:

$$\log W_t = 5.85 + 7.35(\log D + 1) - 4.62 \log(L_1 + L_2) + 3.28 \log L_2 + {G_t}/{\beta}$$

$$\beta = 1.0 + \frac{3.63(L_1 + L_2)^{5.20}}{(D+1)^{8.46} L_2^{3.52}}$$

Where:

$D =$ thickness of rigid pavement slab, in., and

$$G_t = \log\left[{(4.5 - P_t)}/{(4.5 - 1.5)} \right]$$

All other terms as defined above.

The following equation expresses any axle load (W_t) in terms of the number of applications of the standard 8.2-Mg (18-kip) single-axle load (W_{18}) that would cause the same rigid pavement damage as one single pass of the (W_t) axle load,

$$E_i = \frac{W_{t_{18}}}{W_{t_i}} \left[\frac{(L_i + L_2)^{4.62}}{(18+1)^{4.62}} \right] \left[\frac{10^{G_t/\beta_{18}}}{\left(10^{G_t/\beta_i}\right) L_2^{3.28}} \right]$$

This expression is defined as the equivalency factor for rigid pavements.

ESALs

Given the large variability of existing US 59 pavement structures throughout the project limits, it is preferable to utilize the raw axle load data for forecasting future pavement needs. However, some preliminary work was performed assuming a flexible pavement (SN = 6, p_t = 2.5) in the early stages of the project (Ref 8). In addition, preliminary calculations were performed on the 1995 south site data using Excel macros originally developed by Garner to assess the relative ESAL impact of the traffic stream.

Using the 15-vehicle classification scheme introduced in this report, the following distribution of daily ESAL contribution was derived (Fig 4.17).

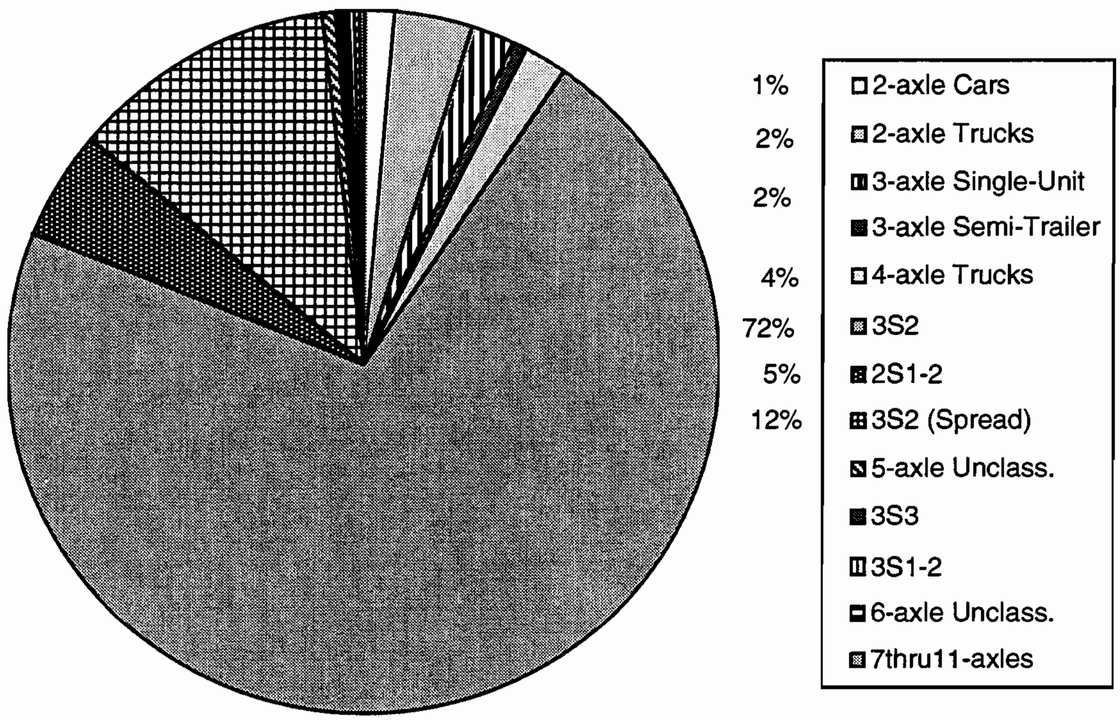

Figure 4.17 Daily ESAL contribution by vehicle class — south site, 1995

Comparison with Figure 4.5 indicates that, although the 3S2 trucks comprise only 16 percent of the traffic, they contribute 71 percent of the ESALs. Meanwhile, 2-axle vehicles comprise 78 percent of the traffic yet contribute only 5 percent of the ESALs.

Axle Loads

It was determined that a procedure utilizing the pavement structure as a variable and incorporating the actual axle loading patterns of the mixed traffic stream would be the preferable method of providing numerous traffic load forecasts. In order to provide an increased level of accuracy in the traffic load forecasts, axle loading patterns were developed for each axle group type within each vehicle class.

The 1995 south site traffic sample was utilized for developing the axle load distributions and patterns. Since both sites experience a majority of the same vehicles, data from only one site were sought in order to reduce any redundancy that would be incorporated by including the axle loadings from the same vehicles twice. Examination of the 1995 south site data set indicated that there were roughly 50 percent fewer errors recorded daily and that more data were available than at the north site; therefore, the south site data set was used to compile the axle loading summary.

Incorporating the previous work by Izadmehr (Ref 9), axle load distributions were compiled for steering axles in each vehicle class to be used in traffic load forecasts for flexible pavements. Axle group loads for vehicle classes with a consistent volume in both lanes were compiled separately in order to analyze any differences between the two distributions. If no discernible difference existed, then the axle load distributions were combined and utilized for forecasts for either lane. Axle load differences between lanes were kept separate and used for the respective lane forecast. Six of the twelve vehicle classes had volumes that allowed lane-by-lane comparisons. These six classes were the 2-axle passenger cars, 2-axle trucks, 3-axle single-unit trucks, 5-axle 3S2 trucks, 5-axle 2S1-2 double trailer trucks, and the 5-axle 3S2 trucks with spread single axles on the trailer.

Because the axle loads for the 2-axle passenger car class were all less than 1.8 Mg (4 kip), with approximately 97 percent less than 1.3 Mg (3 kip) in each lane, they did not warrant separate distributions by lane. Owing to the low distribution of axle spacings within the 2-axle car class, the average ESAL factor calculated for each vehicle at the south site in 1995 was used to estimate the ESALs produced by this class. Axle loadings for the 2-axle trucks class (Fig 4.18) were also very similar for each lane. Since 85 percent of these vehicles in both lanes have axle loadings less than 1.8 Mg (4 kip), this class will be represented by the combined axle loading distributions of both lanes. The next class analyzed was the 3-axle single-unit trucks. Although the steering axle loads in the left-hand lane are noticeably less than those in the right-hand lane (Fig 4.19), the difference is negligible once the ESAL calculation threshold of 5.4 Mg (12 kip) is reached. In addition, the axle loading distributions for the tandem axle group (Fig 4.20) are very similar. Therefore, this class will also be represented by a combined axle loading distribution from all 3-axle single unit trucks. The remaining three classes analyzed for lanewise loading differences were the 5-axle truck classes. Of these, only the 5-axle 3S2 semi-trailer trucks exhibited a significant difference in axle group loadings (Fig 4.21), with the exception of the steering axles. However, since the steering axle distributions do not have significant differences beyond the 5.4-Mg (12-kip) threshold, they will not be considered separately. Axle load distributions for each axle group per vehicle class are included as Appendix B.

The importance of analyzing axle group load distributions separately by class is indicated in the following charts (Figs 4.22 and 4.23). Single-axle group and tandem-axle group load distributions were plotted for all applicable vehicle classes. The variability in axle load patterns between classes is evident in these charts. It should be noted that the 5-axle, 3S2 semi-trailer trucks with spread single axles on the trailer have the heaviest loads in both charts.

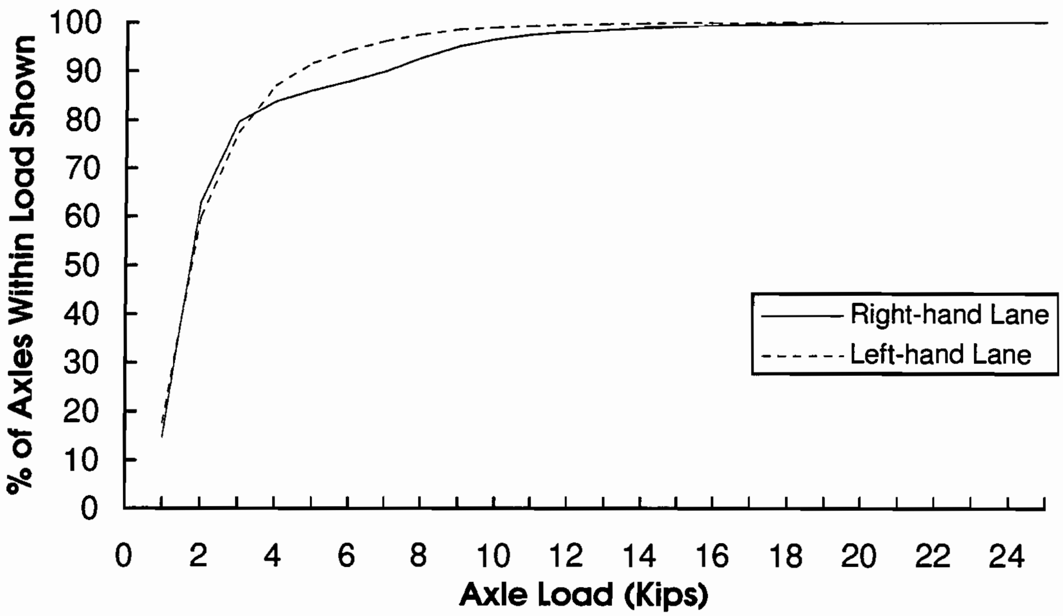

Figure 4.18 Axle load distribution: 2-axle trucks

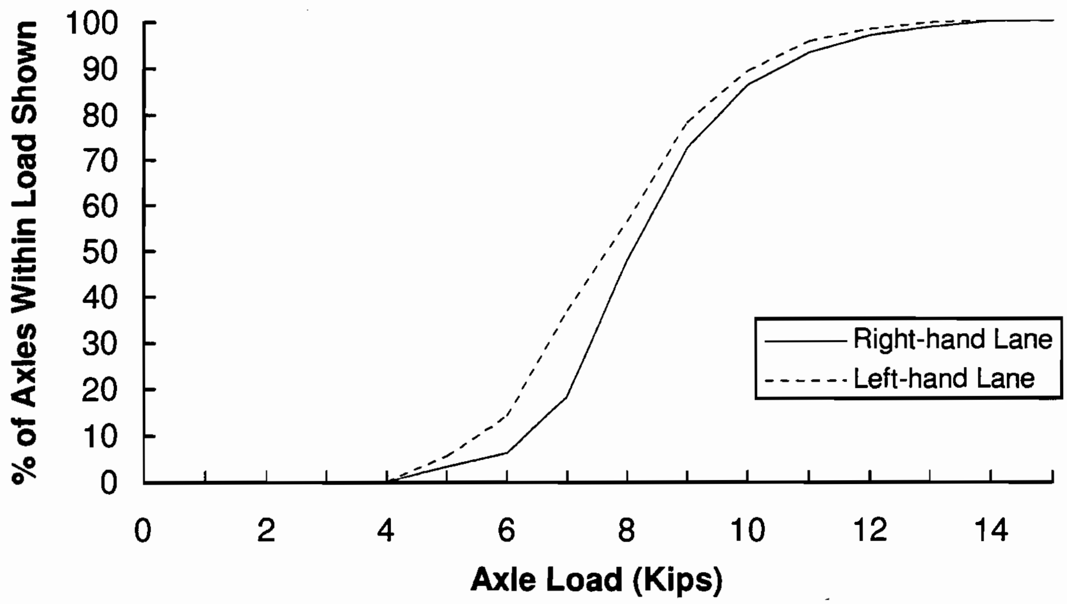

Figure 4.19 Steering axle load distribution: 3-axle single-unit trucks

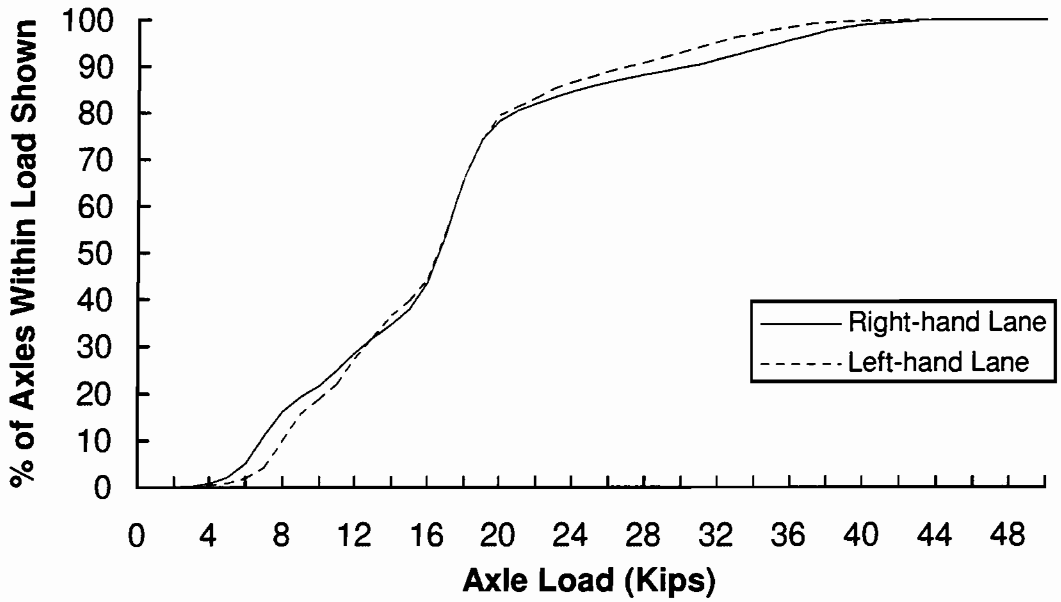

Figure 4.20 Tandem axle load distribution: 3-axle single unit trucks (1 kip=0.454 Mg)

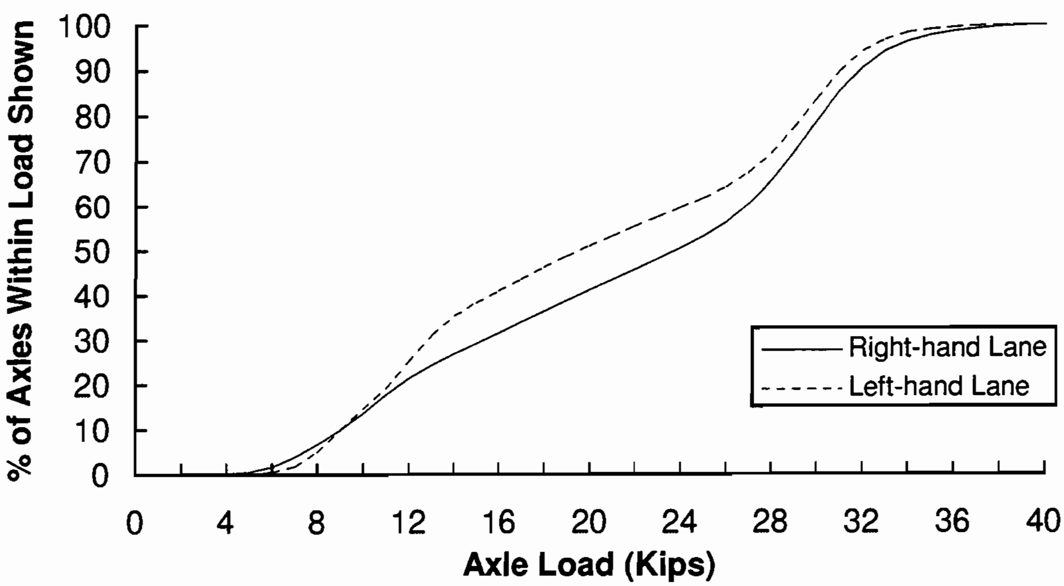

Figure 4.21 Tandem axle load distribution: 5-axle 3S2 trucks (1 kip=0.454 Mg)

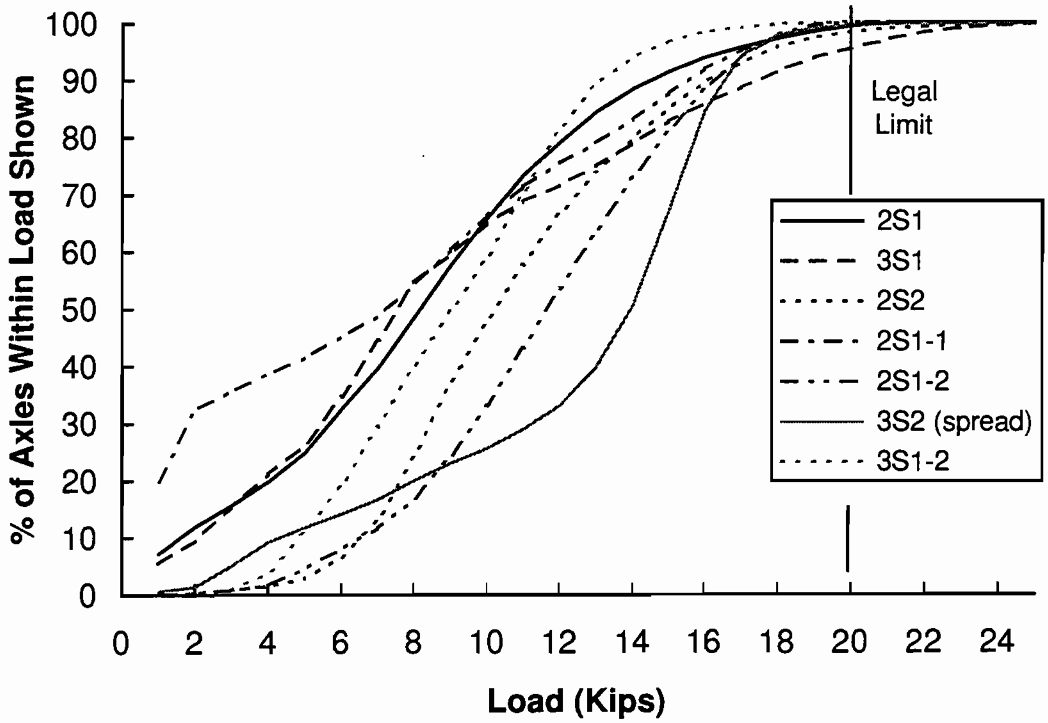

Figure 4.22 Single-axle load distributions: Trucks (1 kip=0.454 Mg)

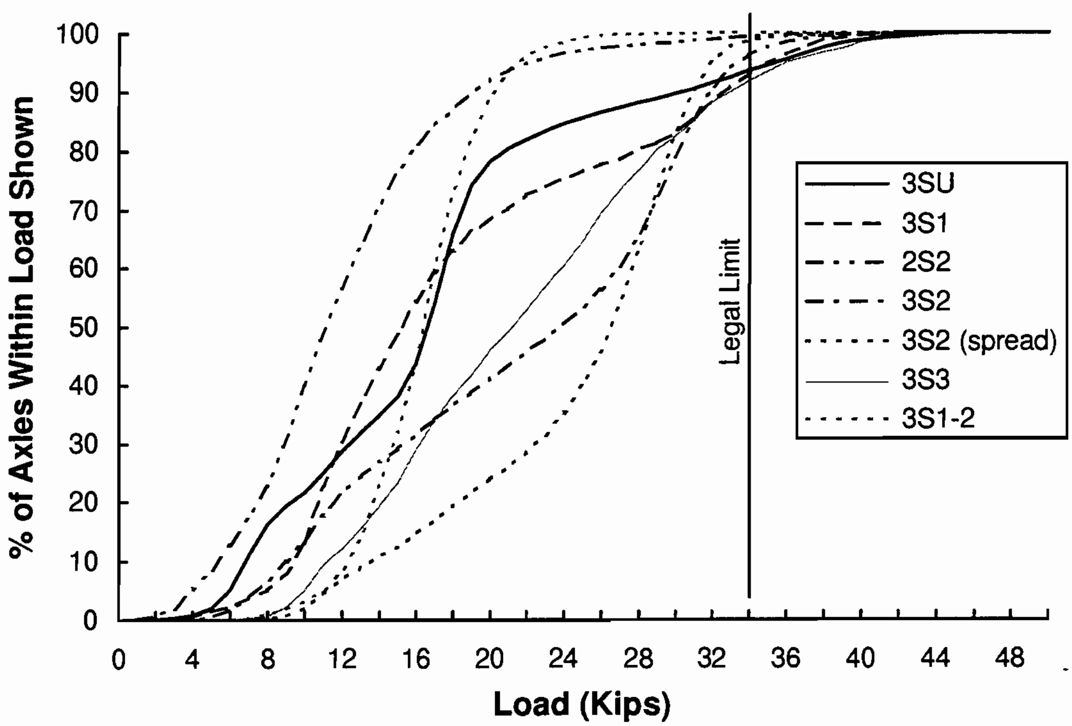

Figure 4.23 Tandem-axle load distributions: Trucks (1 kip=0.454 Mg)

NORTH–SOUTH SITE COMPARISON

The 11-km proximity of the two WIM sites allowed comparison of individual vehicle records from each site. By identifying a time offset to allow travel between the two sites, the vehicle records for an individual vehicle were captured from each site. This procedure was performed only on a limited basis and utilized 5-axle vehicles in order to provide a sufficient amount of data to accurately identify an individual vehicle.

Analysis of 55 5-axle vehicle records just prior to re-calibration in September of 1995 indicated a difference in the sums of gross vehicle weights of 3.8 percent between sites. Since the 55 vehicles should have nearly identical gross vehicle weights when measured at both sites, the difference indicated a change in system performance during the time after initial calibration. As outlined in Chapter 3, calibration of each WIM site is an extremely important factor in assuring quality load estimates. Calibration is designed to account for site-specific operating conditions, such as pavement profile, grade, pavement surface condition, the speed of traffic at the site, and the overall behavior of the tire-force transducers as they interact with the pavement structure.

After re-calibration of both WIM sites, the difference was only 1.0 percent in the sum of measured gross vehicle weights for 57 5-axle trucks identified at both WIM sites. The average difference between measured gross vehicle weights of individual trucks was 5.0 percent. Individual axle load estimates for the two sites ranged from an average 2 percent difference for the steering axle, to an average 3 percent, 4 percent, 4 percent, and 7 percent difference for the 2nd through 5th axles, respectively.

SUMMARY

As mentioned previously, the traffic data set from this project is one of the most complete and detailed ever compiled. The two unique WIM systems installed on US 59 near Corrigan, Texas, have allowed the determination of traffic volume, traffic loading, lateral wheel position, and both air and pavement temperature trends from late 1992 through early 1996. Daily, weekly, monthly, and annual comparisons have validated the reliability of this data set. A site-specific vehicle classification scheme was developed that estimates the vehicle classes used during TxDOT manual classification counts. This classification scheme successfully classified over 98 percent of the observed vehicles at these sites. Lanewise axle-group load distributions were developed from 1995 data collected at the south site for each vehicle class described herein by lane. A small increase in the speed of the traffic stream has also been documented at both sites after a change in the posted speed limit from 55 mph to 70 mph. By establishing these site-specific trends, we have developed a reliable methodology for forecasting future traffic loadings on this highway. This methodology is presented in Chapter 5.

CHAPTER 5. TRAFFIC-LOAD FORECASTING PROCEDURE

OBJECTIVE

As mentioned in Chapter 3, initial efforts in the project to produce a traffic-load forecasting methodology were focused on calculating the equivalent single axle load (ESAL) for every observed vehicle individually. While this is perhaps the most accurate method, it is also the most demanding in terms of data acquisition and processing requirements. It also requires that a terminal serviceability index and an index for pavement strength — slab depth for rigid pavements or structural number for flexible pavements — be defined before making the ESAL calculations.

Given that quantification of the estimated performance of various pavement structures under a specific, forecasted pattern of cumulative traffic loads is the objective of pavement analysis, design, and management, we developed the traffic-load forecasting methodology described in this chapter to characterize patterns of axle loads in terms of the expected frequency of occurrence of loads of specific magnitude, which we termed *load frequency distributions*. Such load frequency distributions can be derived from observed WIM data and can be arranged according to vehicle class, axle configuration (steering, single, tandem, tridem), and to other criteria. The arrangements found appropriate for use in the traffic-load forecasting methodology presented in this chapter are described and illustrated in Chapter 4.

The concept of equivalent single axle load (ESAL), which had its origin in the AASHO Road Test, is perhaps the most popular procedure for apportioning the relative contribution to cumulative pavement damage by a given axle load. The AASHO Road Test ESAL calculation procedure (see Chapter 4), which operates on data sets arranged in an axle-load frequency distribution format, has, therefore, been incorporated into the traffic-load forecasting computer program that is described in this chapter. Other calculation procedures can, of course, be applied to the axle-load frequency distribution data sets described above.

To be useful in the forecasting of traffic loads at sites where WIM data are not available, it is desirable to relate the observed load frequency distributions at a WIM station to other associated traffic parameters, such as AADT, directional distribution, lanewise distribution, vehicle class, and axle-group type. Then, by assuming that the pattern of vehicle loading at the site of interest is similar to that at the WIM station, an estimate of the expected traffic loads can be made — based on the forecasted values of conventionally-observed traffic parameters — without actually weighing vehicles at the site.

PROCEDURE

A schematic of the traffic-load forecasting methodology is provided on the following page (Fig 5.1). There are two types of variables within this methodology: pavement variables and traffic variables. Pavement variables consist of pavement type (flexible or rigid), structural number (SN) for flexible pavement analysis or pavement slab thickness (D) for rigid pavement analysis, and terminal serviceability (p_t). Traffic variables consist of the annual average daily traffic (AADT), the directional distribution of the traffic volume, the traffic volume distribution between lanes, traffic volume classification distribution by lane, analysis period (n) in years, and the average annual compound traffic volume growth rate (g) in percent. Utilizing these input parameters, the methodology will produce an estimated equivalent number of 8.2-Mg (18-kip) single axle loads (ESALs) that will be experienced over the design period in each lane and in the design direction.

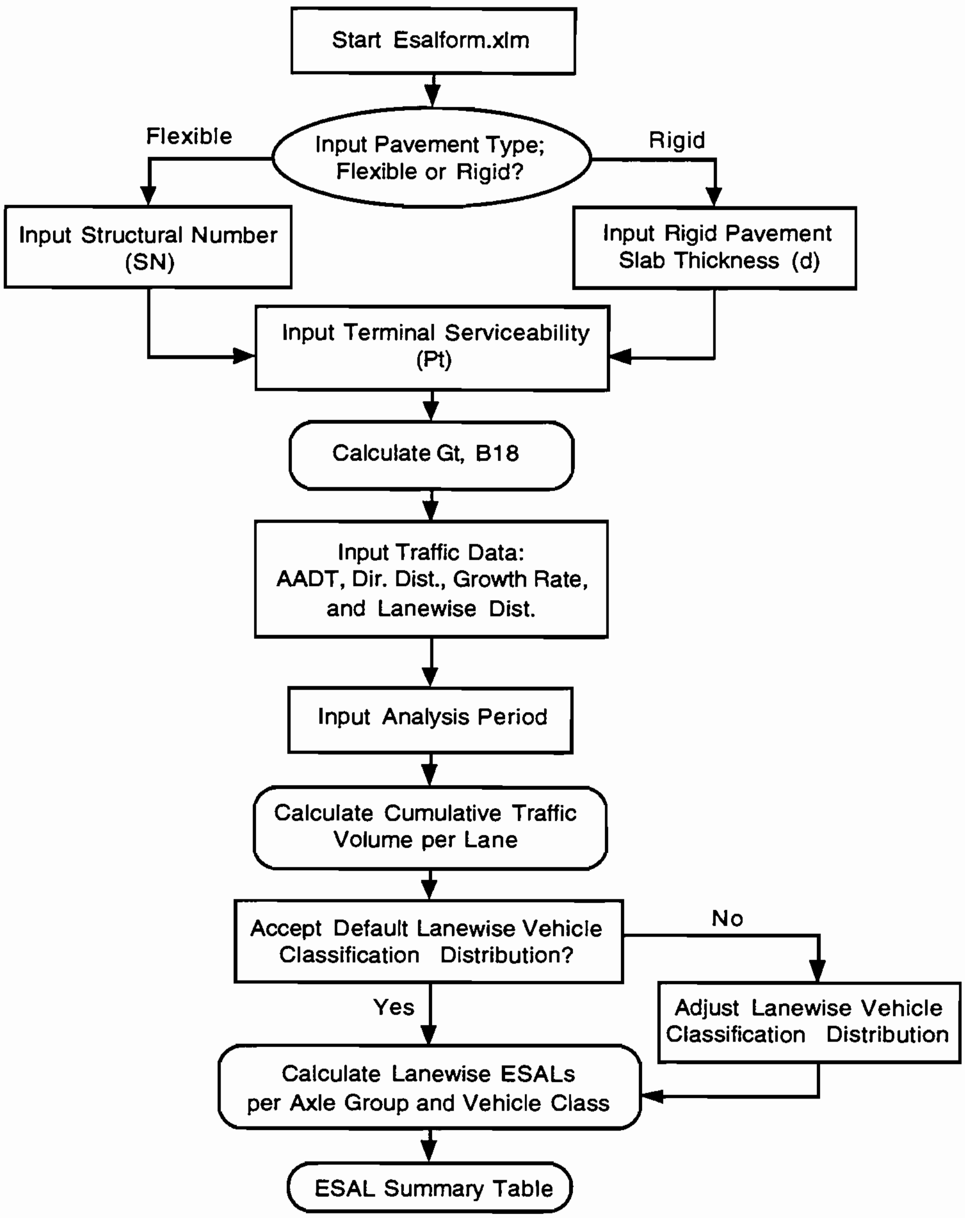

Figure 5.1 Schematic of ESALFORM.xlm procedure

Continuing early efforts in this project, we created the computer program developed to facilitate this methodology within Microsoft Excel as an internal macro program — one designed to be used after completing the data acquisition and analysis procedure outlined in Figure 2.4. In order to demonstrate the use of this methodology and the related macro, traffic data recorded throughout the duration of this project will be used to develop a sample traffic loading forecast for US 59.

This methodology incorporates many traffic volume and lanewise statistics that have been compiled throughout the project, including lanewise vehicle classification distributions, lanewise axle loads by axle-group type and vehicle class, and annual average daily traffic volumes by lane. In order to avoid redundancy within the data set, only the south site traffic data were used in developing the axle-load frequency distributions.

For the vehicle classes that have a significant volume in both the right-hand lane and the left-hand lane, the axle load distributions were compiled separately for each lane. If a significant volume of axle loads could not be gathered for a vehicle class in the left-hand lane, the load distributions for that vehicle class from both lanes were combined and used for forecasting the loadings in both lanes. While only data from 1995 were used in compiling the axle load distributions, the patterns of traffic volumes and loadings have been quite consistent (see Chapter 4) over the three years of the project. The data used for the load distributions are listed below:

January 1st – 31st, Total number of observations 197,920
March 1st – 31st, Total number of observations 221,723
May 15th – 28th, Total number of observations 103,670
September 1st, and 3rd – 30th, Total number of observations 213,625
October 1st, 3rd, and 5th – 30th, Total number of observations 211,691
December 1st – 14th, and 20th – 31st, Total number of observations 201,918

Not all 1,150,547 observations were included in the distributions, as approximately 60 percent of these vehicles were passenger cars. The following table provides a breakdown of the number of vehicles used to compile the axle load distributions for each lane.

Table 5.1 Number of vehicle observations utilized in compiling the axle loading distributions

Vehicle Class	Right-hand Lane	Left-hand Lane	Both Lanes
2-axle Trucks	119,603	64,136	
3-axle Single Unit	9,944	3,102	
3-axle Semi-trailer			2,993
4-axle Semi-trailer (3S1)			825
4-axle Semi-trailer (2S2)			5,127
4-axle Double-trailer (2S1-1)			2,229
5-axle Semi-trailer (3S2)	154,625	26,078	
5-axle Double-trailer (2S1-2)	10,662	1,034	
5-axle Semi-trailer (3S2)*	16,791	3,219	
6-axle Semi-trailer (3S3)			2,052
6-axle Double-trailer (3S1-2)			1,487

Although we wanted to incorporate a growth factor for individual vehicle types, only a simple compounding growth rate was utilized to account for the overall growth in AADT throughout the study area. Incorporating individual vehicle class growth rates, such as a growth rate for passenger cars and a separate growth rate for semi-trailer trucks, will have a significant impact on the long-range loading forecasts. Examination of classification trends on US 59 since 1984 indicates a steady classification distribution; therefore, the simple compound growth rate was deemed adequate. The following equation was utilized to account for the compound growth.

$$Growth\_Factor = \frac{\left[(1+i)^n - 1\right]}{i}$$

where:

i = yearly growth rate, and

n = time period in years.

By multiplying the AADT by the directional distribution, the lanewise distribution, the growth factor, and 365 days, the cumulative traffic volume per lane is derived. Using this traffic volume and the traffic classification distribution described previously, the respective number of vehicles in each class is calculated. Once the number of vehicles in a particular class is known, the number of axle group loads applied by that class is distributed across the axle load distributions. For example, of the approximately 48 million vehicles applied over 20 years in the right-hand lane of this example, about 8.4 million are 5-axle semi-trailer 3S2 trucks, which will apply 8.4 million steering axle loads and 16.8 million tandem axle loads. Once the number of axle group loads is known, the equivalency factors developed by AASHTO are applied to the axle group loads and the aggregate number of ESALs produced by each vehicle class is determined. The axle loads range from 0.9 to 18 Mg (2 to 40 kip) for single axles, 0.9 to 22.7 Mg (2 to 50 kip) for tandem-axle groups, and 0.9 to 27.2 Mg (2 to 60 kip) for tridem-axle groups. Only steering axles on flexible pavements, and in excess of 5.4 Mg (12 kip), are used in calculating the total traffic loading.

Both the 2-axle car and the 7-or-more axle truck classes incorporate an average ESAL value per vehicle, instead of utilizing axle load distributions. This method is used for the 2-axle cars because of the small magnitude of the axle loads within this class. The ESAL value used is an average of the individually calculated ESAL values calculated from the south site data set consisting of every 2-axle car recorded in 1995. A flexible pavement with a structural number of 6 and a terminal serviceability of 2.5 was used for the pavement variables. Owing to the large variance in axle group types and low frequency of occurrence within the 7-or-more axle truck class, another average ESAL value was utilized for this class of vehicles. Again, the ESAL value used is the average ESAL value calculated for every individual vehicle recorded at the south site in 1995.

EXAMPLE TRAFFIC-LOAD FORECAST

The example consists of a typical pavement structure located within the project limits: a flexible pavement with a structural number of 6 and a terminal serviceability of 2.5. The traffic data consist of trends observed at the south site between 1993 and 1995. Although no northbound data have been collected as part of this project, analysis of the historical classification counts performed by TxDOT indicates a directional split of approximately 50 percent. Therefore, the AADT used for this example is the annual average daily southbound traffic multiplied by a factor of 2. This southbound traffic was measured at approximately 7,400 vehicles per day, with the right-hand lane carrying about 70 percent of this traffic. Analysis of the AADT over the past ten years throughout the project area indicated an average annual traffic volume growth rate of 2.5 percent. This was applied over the analysis period of 20 years.

Once all of the above data are input, the program prompts the user to either accept the default traffic classification distribution or edit the distribution. The default distribution is from the 1995 south site data set and has been broken down by lane in order to assess the impact on each lane separately. Therefore, the default classification distribution was used in this example. If the user does not have lanewise classification data, the same classification distribution can be input for each lane. Because existing WIM systems are currently operating only on four-lane highways, this program was designed to incorporate only two lanes of directional traffic data.

After the classification data are entered, the program (1) calculates the estimated traffic loading for each lane separately and (2) sums both lanes to provide a total projected loading for the design direction. For each lane, the appropriate axle load distributions are utilized in estimating the cumulative ESALs produced by each respective vehicle class. The steering axle load distribution is used only in flexible pavement analyses, as recommended in Izadmehr's report (Ref 9). The lanewise axle load distributions are stored in two separate files and can be updated at any time.

The completed traffic loading forecast for this example (Table 5.2) predicts about 10 million ESALs in the southbound direction. Of the 10 million ESALs, the right-hand lane is predicted to carry 8.8 million ESALs, or about 85 percent of the total ESALs forecasted. This large ESAL discrepancy between lanes is due to the higher percentage of overall traffic traveling in the right-hand lane, the larger percentage of passenger cars in the left-hand lane, and to the slightly lower axle loading distributions recorded in the left-hand lane. It is important to note that the lanewise characteristics used in this example may be unique to the south site on US 59 and site-specific traffic characteristics should be collected for the particular area in question. For example, recalculating the above traffic loading forecast using the same inputs except for using the right-hand lane classification distribution for both lanes, results in a 16.6-percent increase in the overall ESAL prediction (Table 5.3). This increase is due mostly to a 134-percent increase in ESALs predicted in the left-hand lane for 5-axle (3S2) semi-trailer trucks, from about 0.9 million ESALs to approximately 2.1 million ESALs.

Table 5.2 Example traffic-load forecast, 1995-2015 (lanewise traffic classification data)

Pavement	SN / D	Pt	Gt	B18		
Flexible	6	2.5	-0.20091	0.44496		
AADT	Dir. Dist.	Lane Dist.	n	g	G.F.	
14800	50	70	20	2.5	25.54466	
					ESALs	ESALs
		R-hand %	L-hand %		R-hand	L-hand
o-o	2-axle cars	55.54	63.31		182405	89110
o-----o	2-axle trucks	21.63	26.57		226700	32644
o---oo	3-axle single-unit	1.07	0.91		126053	32504
o---o------o	3-axle semi-trailer	0.28	0.17		46604	12126
o---oo------o	4-axle semi-trailer (3S-1)	0.08	0.03		22303	3584
o---o------oo	4-axle semi-trailer (2S-2)	0.52	0.16		82669	10901
o---o-----o-----o	4-axle double-trailer (2S-1-1)	0.24	0.03		68324	3660
o---oo------oo	5-axle semi-trailer (3S-2)	17.46	7.45		6301374	920046
o---o-----o-o-----o	5-axle double-trailer (2S-1-2)	0.89	0.27		508008	56603
o---oo-----o-o	5-axle semi-trailer (3S-1-1)	1.90	0.88		1117925	223329
o---oo------ooo	6-axle semi-trailer (3S-3)	0.19	0.1		52968	11948
o---oo-----o-o-----o	6-axle double-trailer (3S-1-2)	0.14	0.07		30237	6479
	7 or more axle trucks	0.06	0.05		39854	14234
				ESALs =	8,805,423	1,417,168
			Total ESALs by Direction = 10,222,592			

Table 5.3 Revised example traffic-load forecast, 1995-2015 (directional classification data)

Pavement	SN / D	Pt	Gt	B18		
Flexible	6	2.5	-0.20091	0.44496		
AADT	Dir. Dist.	Lane Dist.	n	g	G.F.	
14800	50	70	20	2.5	25.54466	
					ESALs	ESALs
		R-hand %	L-hand %		R-hand	L-hand
o-o	2-axle cars	55.54	55.54		182405	78174
o-----o	2-axle trucks	21.63	21.63		226700	26574
o---oo	3-axle single-unit	1.07	1.07		126053	38220
o---o------o	3-axle semi-trailer	0.28	0.28		46604	19973
o---oo------o	4-axle semi-trailer (3S-1)	0.08	0.08		22303	9558
o---o------oo	4-axle semi-trailer (2S-2)	0.52	0.52		82669	35429
o---o-----o-----o	4-axle double-trailer (2S-1-1)	0.24	0.24		68324	29282
o---oo------oo	5-axle semi-trailer (3S-2)	17.46	17.46		6301374	2156241
o---o-----o-o-----o	5-axle double-trailer (2S-1-2)	0.89	0.89		508008	186579
o---oo-----o-o	5-axle semi-trailer (3S-1-1)	1.90	1.90		1117925	482187
o---oo------ooo	6-axle semi-trailer (3S-3)	0.19	0.19		52968	22701
o---oo-----o-o-----o	6-axle double-trailer (3S-1-2)	0.14	0.14		30237	12959
	7 or more axle trucks	0.06	0.06		39854	17080
				ESALs =	8,805,423	3,114,957
			Total ESALs by Direction = 11,920,380			

CHAPTER 6. CONCLUSION AND RECOMMENDATIONS

As part of Project 987, WIM data measured and recorded from late 1992 through early 1996 have been used to develop a traffic-load forecasting methodology, the main objective of this report. Mixed traffic loadings by various vehicle types and axle groups were summarized into axle-load distributions by vehicle type and were converted into 8.2-Mg (18-kip) equivalent single axle loads by the use of the AASHTO equivalency equations first developed after the AASHO Road Test in the 1950s. The two unique WIM systems used in this project, located on the southbound lanes of US 59 in east Texas, have been in service continually and have collected such data as the date, time, speed, lateral lane position, axle spacings, and wheel loads for about 7,500 individual vehicles per day. These data have been used to establish axle-group loading distributions for individual vehicle classes by lane.

CONCLUSIONS

(1) The weigh-in-motion systems are a reliable and efficient means of collecting vehicle loading patterns and complete traffic volume characteristics. In addition to establishing the lanewise axle-group load patterns for several individual vehicle classes, the data collected during this project were used to establish the annual average daily traffic, traffic stream classification characteristics, and various other site-specific trends. The speed data collected were also used to document a minor increase (about 2–3 percent) in traffic volume speed statistics after a change in the posted speed limit from 55 mph to 70 mph.

(2) Infrared sensors provide a feasible means of determining the lateral position of a vehicle within a lane, as well as the number of tires on each axle of a vehicle. Lateral position frequency distributions were established for 2-axle vehicles and trucks by lane. Analyzing a sample of 5-axle semi-trailer trucks, the sensors correctly identified 98 percent as having dual tires on at least one axle. By collecting these data in an inconspicuous manner, topics such as the edge loading of pavements and more advanced vehicle classification techniques can be examined with an unbiased data set.

(3) Remote operation of the WIM systems is practical and efficient. Utilizing the remote communication capabilities of the WIM systems, data were downloaded automatically on a consistent basis and the real-time operations of the systems were monitored from remote locations.

(4) Daily, seasonal, and annual trends are evident in the traffic volumes on US 59. Data collected from both WIM sites followed clear patterns that indicate the possibility of using data sampling.

RECOMMENDATIONS

Through this project, project personnel have gained numerous insights into the operation and maintenance of a continuously operated WIM system, many of which will provide invaluable assistance on future projects. The following recommendations are based on the aforementioned operation of the two unique WIM systems and the data analysis procedure.

(1) Northbound traffic loading data should be collected at this site. Although some historical northbound traffic volume data have been gathered from TxDOT, no current northbound traffic loading patterns on US 59 are available.

(2) In view of the significant amount of time required to download data, the efficiency of the data storage and transmission procedures should be improved. Currently, downloading one daily file of approximately 7,500 records requires about 11 minutes. Downloading these data via long-distance telephone is an expensive process. Therefore, some future efforts should be directed towards expediting this process as numerous agencies, including TxDOT, utilize the remote downloading capabilities of these systems.

(3) Loading data recorded from motorcycles, passenger cars, and pickup trucks should not be stored. The minimal impact on pavement performance caused by these vehicles should preclude their inclusion in data collection efforts. Instead, just the frequency of these vehicles should be recorded in order to provide the means of compiling numerous traffic volume and classification trends.

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13. Machemehl, R., C. E. Lee, and C. M. Walton, "Truck Weight Surveys by In-Motion Weighing," Report No. 181-1F, Center for Highway Research, The University of Texas at Austin, 1975.

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17. Transportation Research Board, "Truck Weight Limits," Special Report 225, National Research Council, Washington, D.C., 1990.

18. Transportation Research Board, "Effects of Heavy-Vehicle Characteristics on Pavement Response and Performance," Report No. 353, National Cooperative Highway Research Program, National Research Council, Washington, D.C., 1993.

19. U.S. Department of Transportation Federal Highway Administration, "Traffic Monitoring Guide," Office of Highway Information Management, Washington, D.C., 1995.

APPENDIX A:

WIM Calibration Data

In September of 1995, the two WIM systems used for the project were calibrated. A 5-axle tractor semi-trailer (3S2) truck was used as a reference vehicle. The truck was supplied by TxDOT and consisted of an air-bag suspension with a flat-bed semi-trailer carrying a steel-track Caterpillar dozer for additional load. This vehicle type was used for calibration due to the high percentage (more than 70%) of the overall traffic loading carried by these vehicles.

The calibration truck was weighed statically by successively moving axle groups onto a certified full-length vehicle scale to measure axle loads and gross-vehicle weight. Axle spacings were measured with a 15-m (50-foot) steel tape and were measured to the nearest inch at the site. The following is a summary of the calibration truck statistics.

Weight Data:

Steering Axle:	10,700 lb.
Drive Tandem:	19,460 lb.
Trailer Tandem:	23,800 lb.
Gross-vehicle Weight:	53,960 lb. (53.96 kip)

(1000 lb = 1 kip = 0.454 Mg)

Axle Spacing:

Axle 1 - 2:	3.55 m (11.7 ft.)
Axle 2 - 3:	1.33 m (4.4 ft.)
Axle 3 - 4:	9.18 m (30.2 ft.)
Axle 4 - 5:	1.25 m (4.1 ft.)
Wheelbase:	15.3 m (50.4 ft.)

Table A.1 North Site - Lane 1 Pre-Calibration

Pass No.	Speed Point (mph)	Measured GVW (kips)	% Difference	Avg. % Diff.
1	50	56.7	+5.0%	
2	50	56.1	+4.0%	+4.5%
3	50	56.3	+4.3%	
4	55	57.7	+7.0%	
5	55	57.8	+7.1%	+7.8%
6	55	59.1	+9.5%	
7	60	56.7	+5.0%	
8	60	57.5	+6.6%	+5.6%
9	60	56.8	+5.3%	

Table A.2 North Site - Lane 2 Pre-Calibration

Pass No.	Speed Point (mph)	Measured GVW (kips)	% Difference	Avg. % Diff.
1	50	51.1	-5.3%	
2	50	52.3	-3.1%	-3.5%
3	50	52.8	-2.1%	
4	55	52.7	-2.3%	
5	55	52.2	-3.3%	-2.9%
6	55	52.3	-3.1%	
7	60	51.2	-5.1%	
8	60	50.8	-5.8%	-5.1%
9	60	51.6	-4.4%	

Table A.3 South Site - Lane 1 Pre-Calibration

Pass No.	Speed Point (mph)	Measured GVW (kips)	% Difference	Avg. % Diff.
1	50	53.6	-0.7%	
2	50	50.8*	-5.8%*	
3	50	53.5	-0.8%	-1.0%
4	50	53.1	-1.6%	
5	55	52.3	-3.1%	
6	55	51.8	-4.0%	-3.8%
7	55	51.6	-4.4%	
8	60	52.3	-3.1%	
9	60	52.1	-3.4%	-4.0%
10	60	51.0	-5.5%	

*This data point was not used.

Table A.4 South Site - Lane 2 Pre-Calibration

Pass No.	Speed Point (mph)	Measured GVW (kips)	% Difference	Avg. % Diff.
1	50	50.3	-6.8%	
2	50	52.7	-2.3%	
3	50	50.8	-5.8%	-4.8%
4	50	51.7	-4.2%	
5	55	54.7	+1.4%	
6	55	54.6	+1.2%	+0.4%**
7	55	53.5	-0.8%	
8	60	52.0	-3.6%	
9	60	49.9*	-7.5%*	-1.0%**
10	60	54.0	+0.1%	
11	60	54.3	+0.6%	

*This data point was not used.

**No adjustment made to existing speed point correction factor.

Table A.5 North Site - Lane 1 Post-Calibration

Pass No.	Speed Point (mph)	Measured GVW (kips)	% Difference	Avg. % Diff.
1	50	56.4	+4.5%	
2	50	54.7	+1.4%	+4.1%*
3	50	56.4	+4.5%	
4	50	57.2	+6.0%	
5	55	55.1	+2.1%	+2.1%*
6	60	53.5	+0.8%	+0.8%

*Speed point was readjusted following these runs.

Table A.6 North Site - Lane 2 Post-Calibration

Pass No.	Speed Point (mph)	Measured GVW (kips)	% Difference	Avg. % Diff.
1	50	53.4	+1.0%	
2	55	54.1	+0.2%	
3	60	52.9	-2.0%*	
4	60	55.0	+1.9%*	
5	60	57.7	+6.9%**	
6	60	53.7	-0.5%	-0.2%
7	60	54.1	+0.2%	

*Speed point was readjusted following these runs.

**This data point was not used.

Table A.7 South Site - Lane 1 Post-Calibration

Pass No.	Speed Point (mph)	Measured GVW (kips)	% Difference	Avg. % Diff.
1	50	55.4	+2.6%	
2	50	54.5	+1.0%	+1.8%
3	55	52.7	-2.3%*	
4	55	52.0	-3.6%*	
5	55	54.9	+1.7%	
6	60	53.6	-0.7%	

*Speed point was readjusted following these runs.

Table A.8 South Site - Lane 2 Post-Calibration*

Pass No.	Speed Point (mph)	Measured GVW (kips)	% Difference	Avg. % Diff.
1	50	55.0	+2.0%	

*Only the 50 mph speed point required adjustment.

APPENDIX B.

Axle Load Frequency Distributions

Data measured and recorded at the south site in 1995 was used to compile axle load frequency distributions for each axle-group per vehicle class. These load distributions were then used to develop a traffic-load forecasting methodology. The charts included in this Appendix are axle load frequency distributions for each vehicle class developed within this report and have been separated by lane if an adequate sample size was obtained. Data collected from the following dates comprises the total sample.

January 1st - 31st	Observations:	197,920
March 1st - 31st	Observations:	221,723
May 15th - 28th	Observations:	103,670
September 1st, and 3rd - 30th	Observations:	213,625
October 1st, 3rd, and 5th - 30th	Observations:	211,691
December 1st - 14th, and 20th - 31st	Observations:	201,918

The 2-axle passenger car and 2-axle truck distributions do not include all of the representative samples as an adequate sample size was quickly obtained for these vehicle classes.

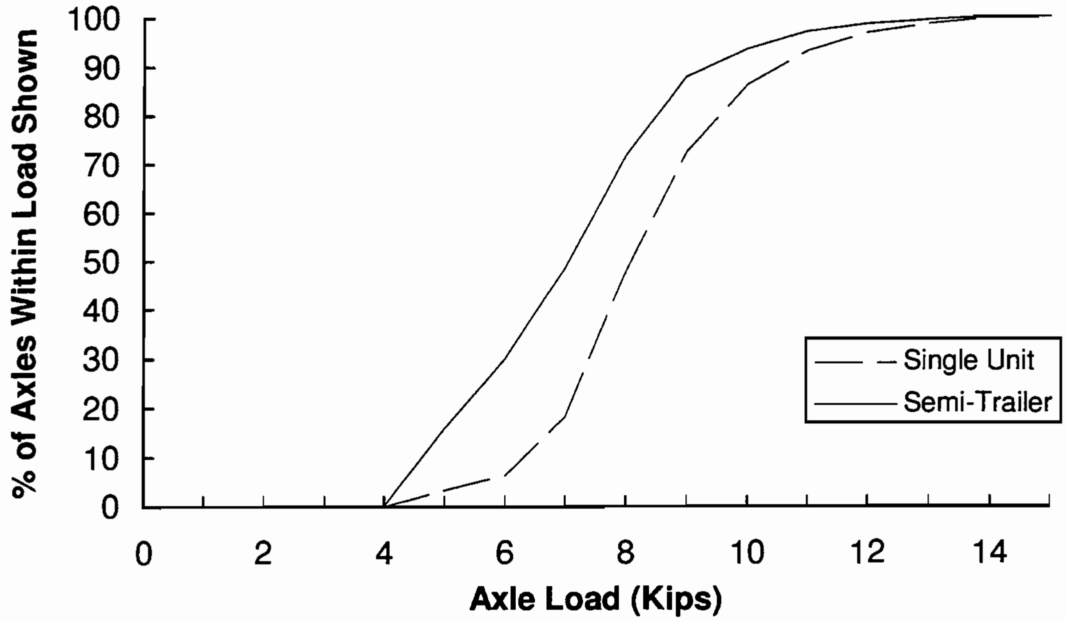

Figure B.1 Steering-axle Load Distribution: 3-axle Trucks

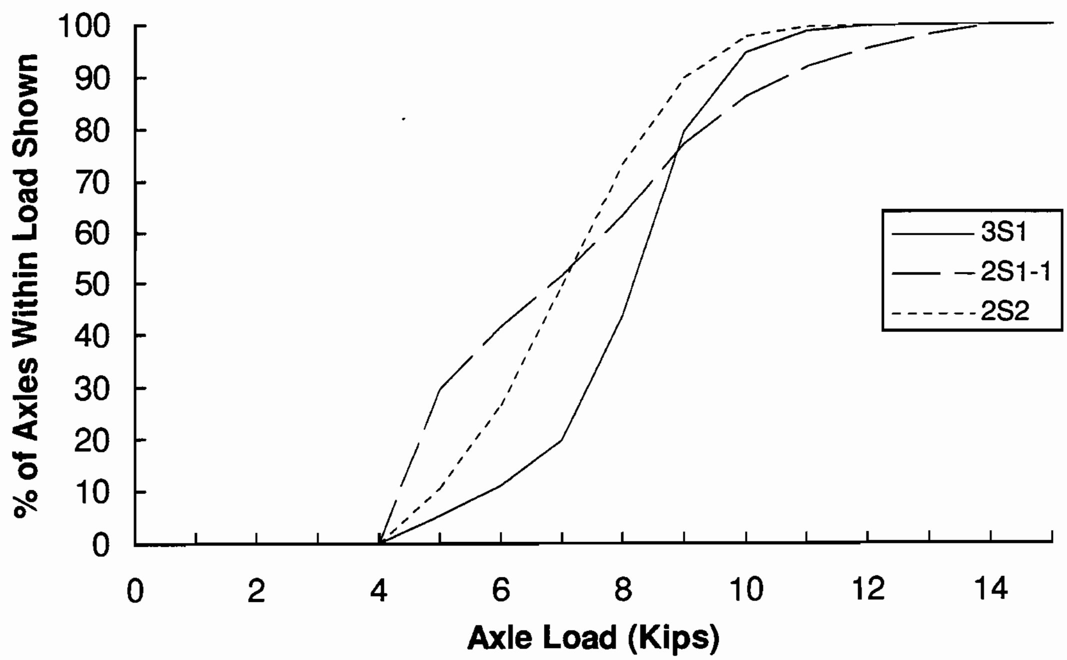

Figure B.2 Steering-axle Load Distribution: 4-axle Trucks

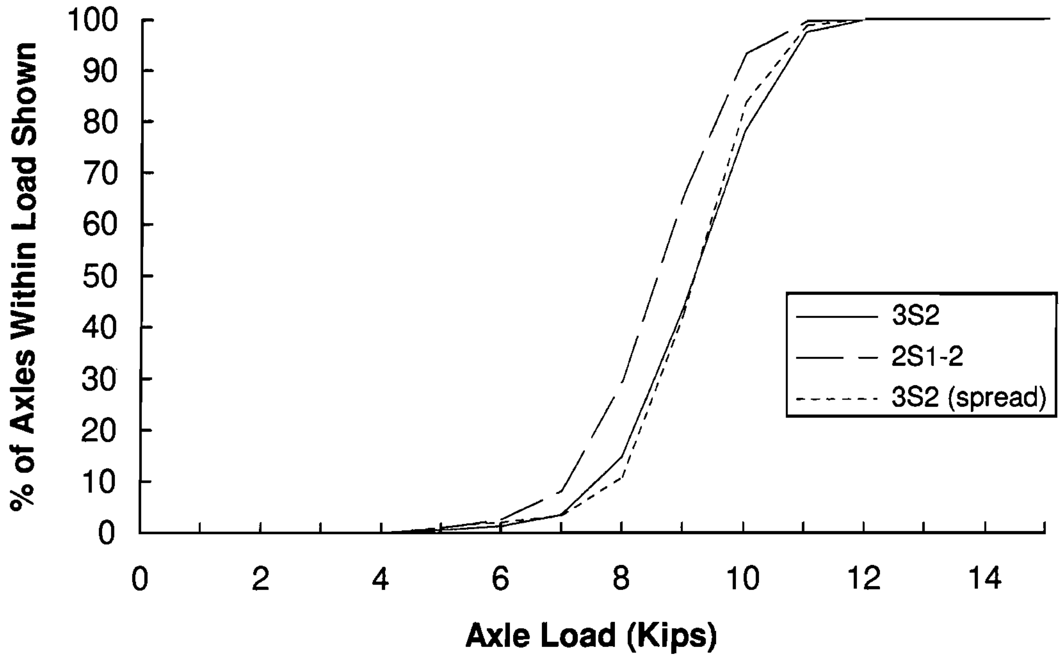

Figure B.3 Steering-axle Load Distribution: 5-axle Trucks

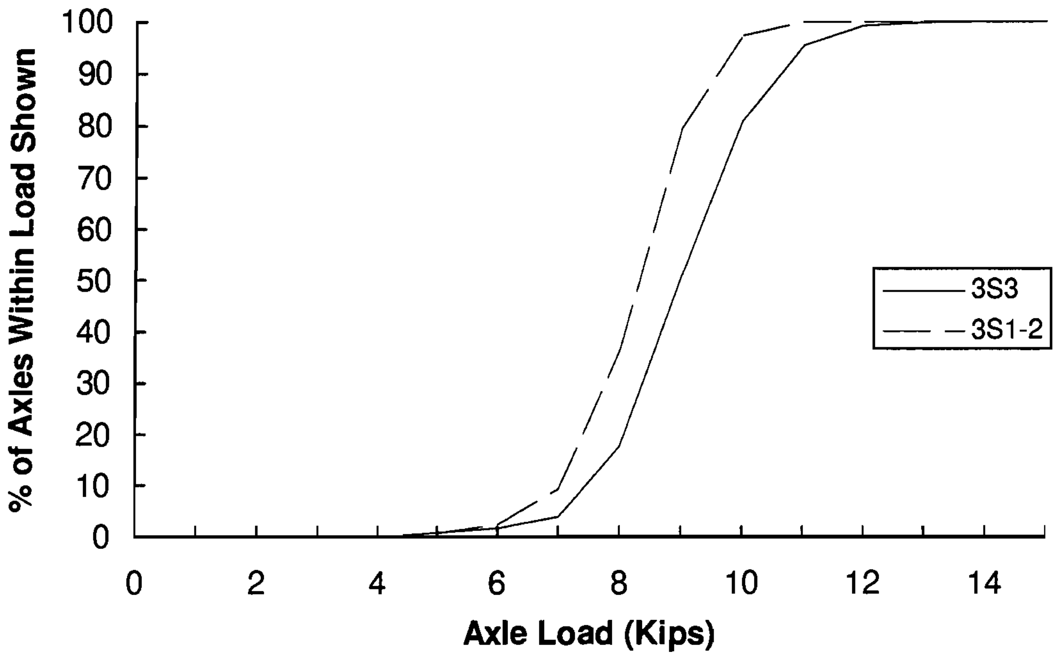

Figure B.4 Steering-axle Load Distribution: 6-axle Trucks

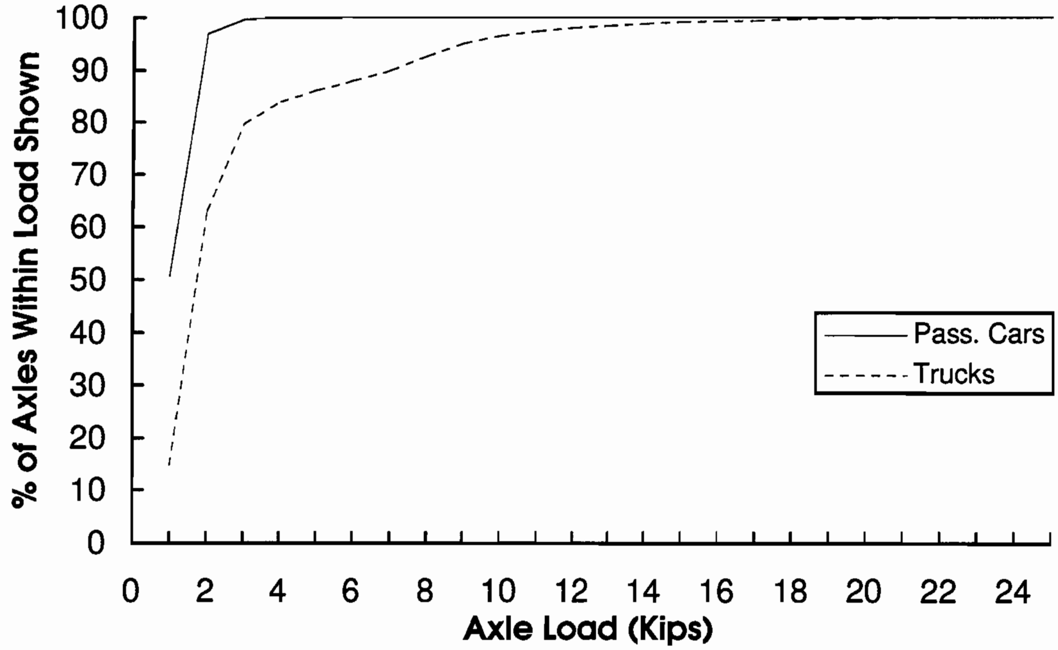

Figure B.5 Axle Load Distribution: 2-axle Vehicles

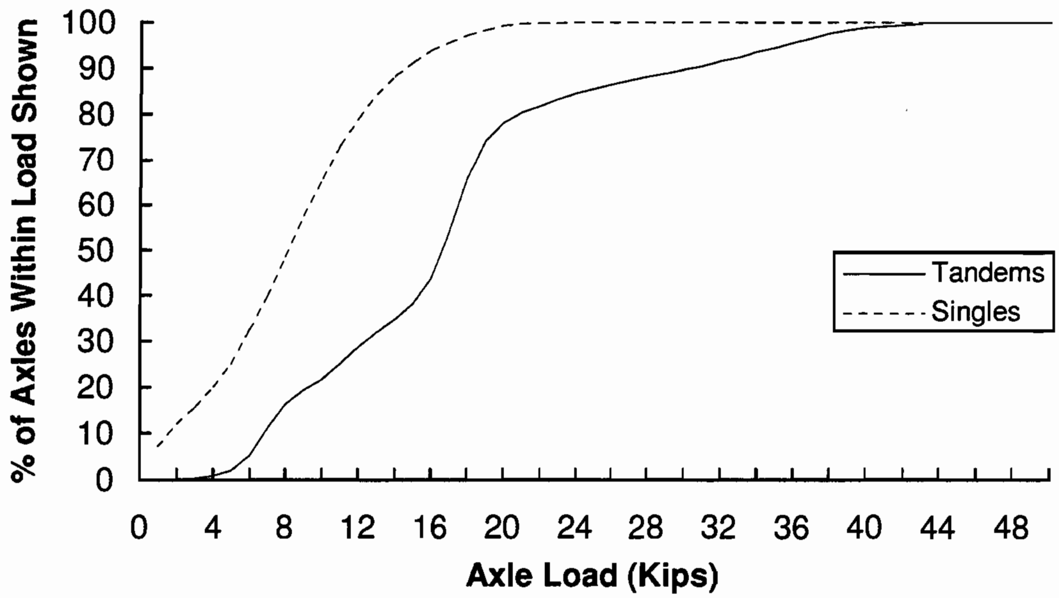

Figure B.6 Axle Load Distribution: 3-axle Trucks

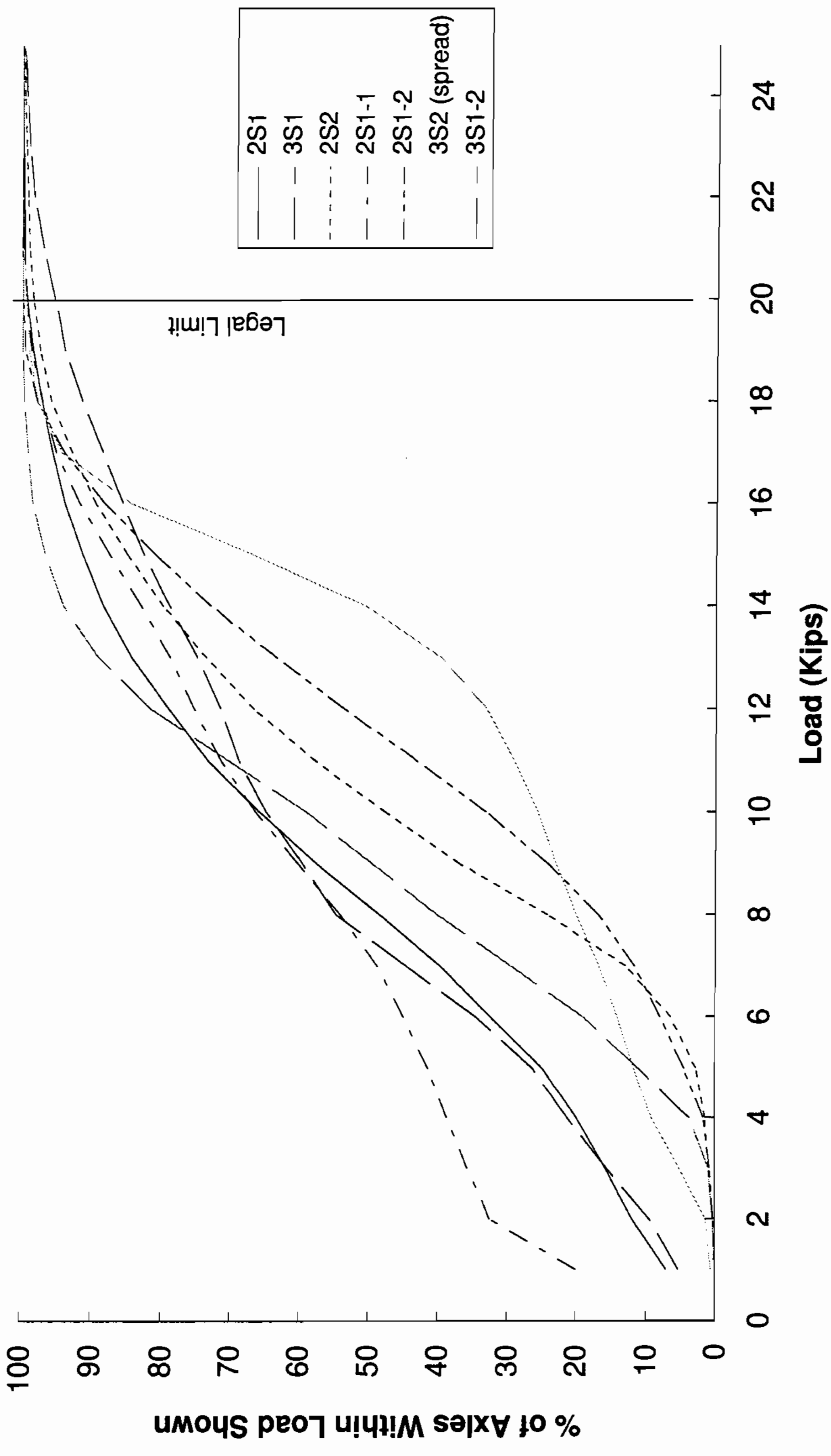

Figure B.7 Single Axle Load Distribution: All Trucks

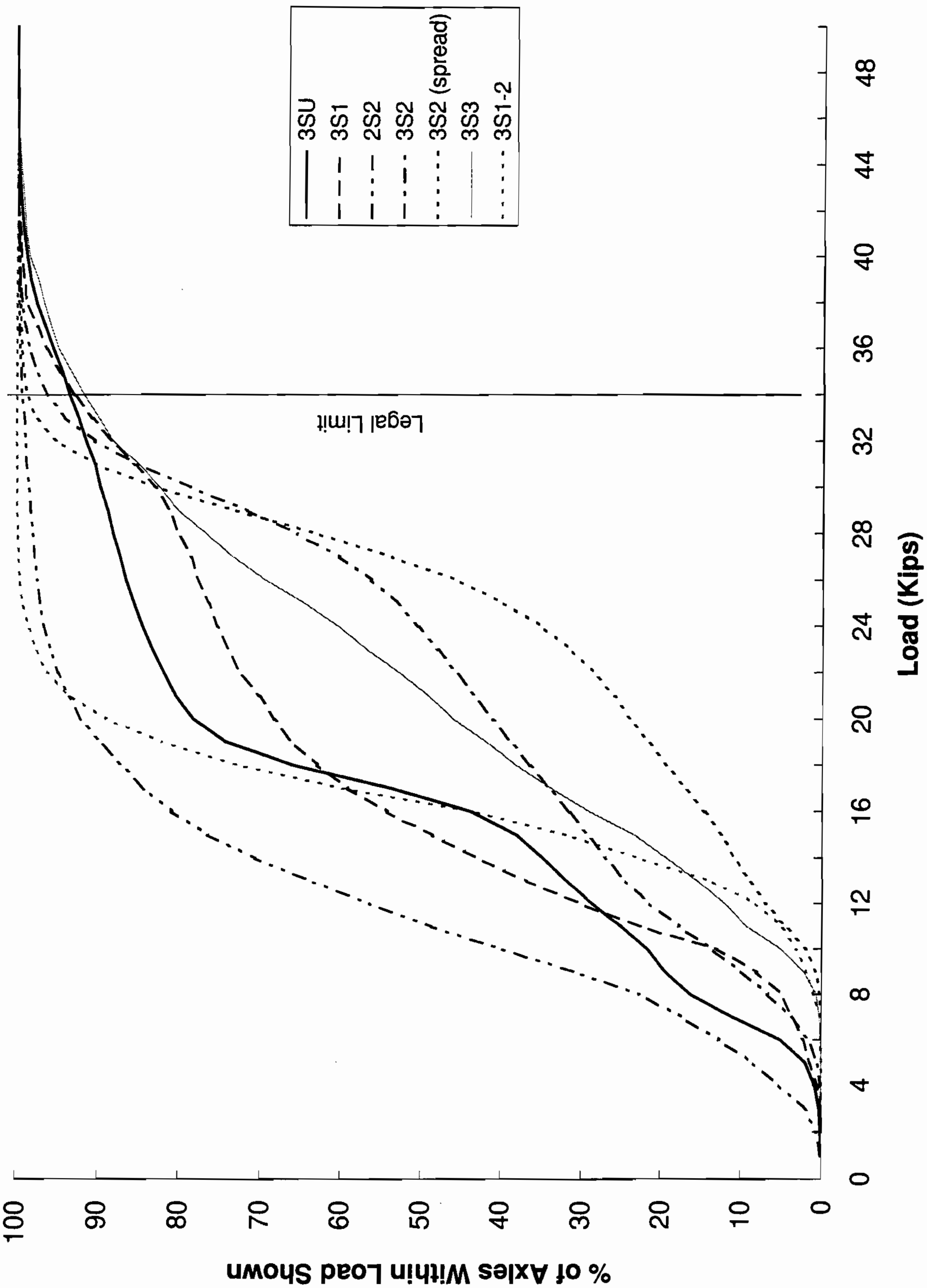

Figure B.8 Tandem Axle Load Distribution: All Trucks

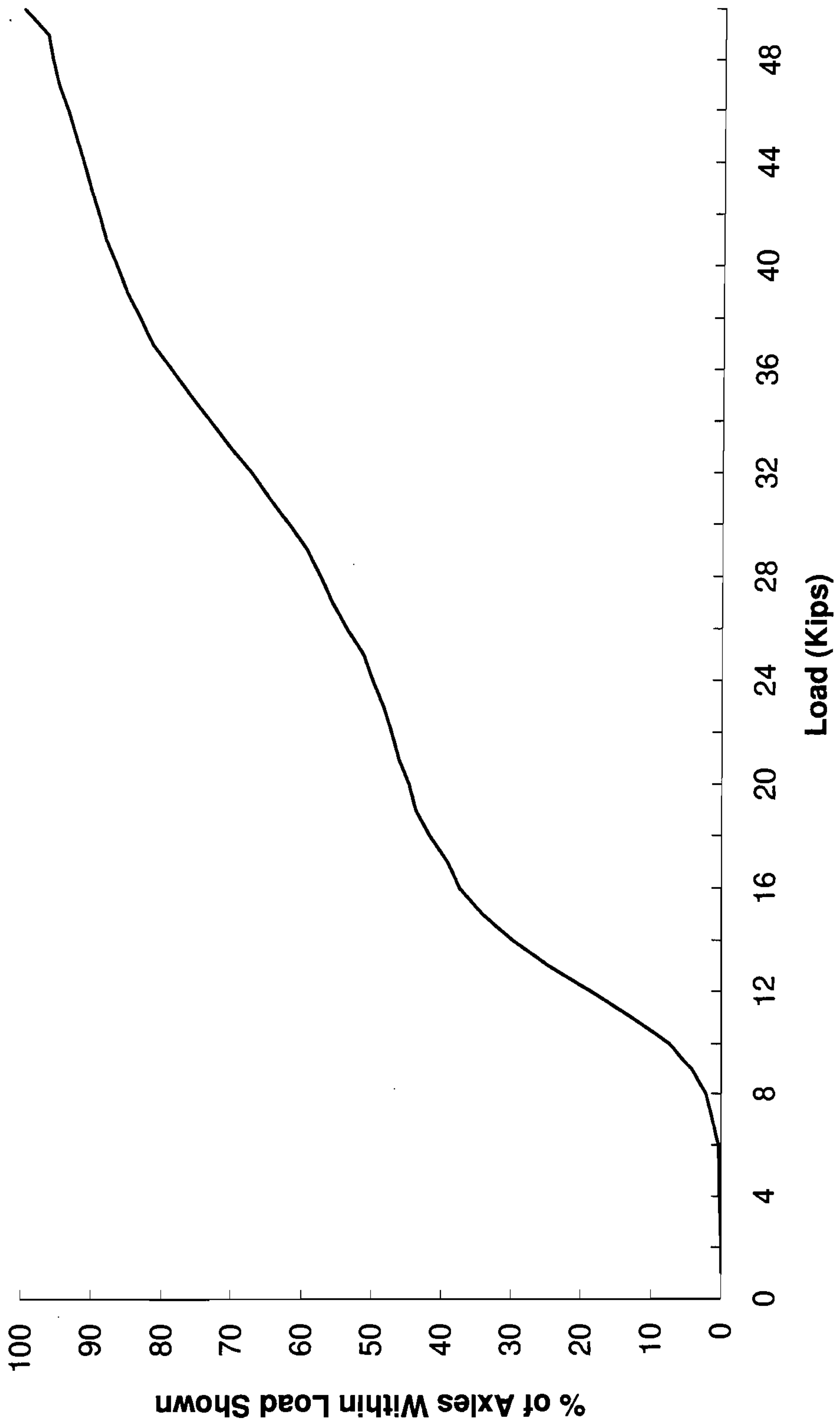

Figure B.9 Tridem Axle Load Distribution: All Trucks

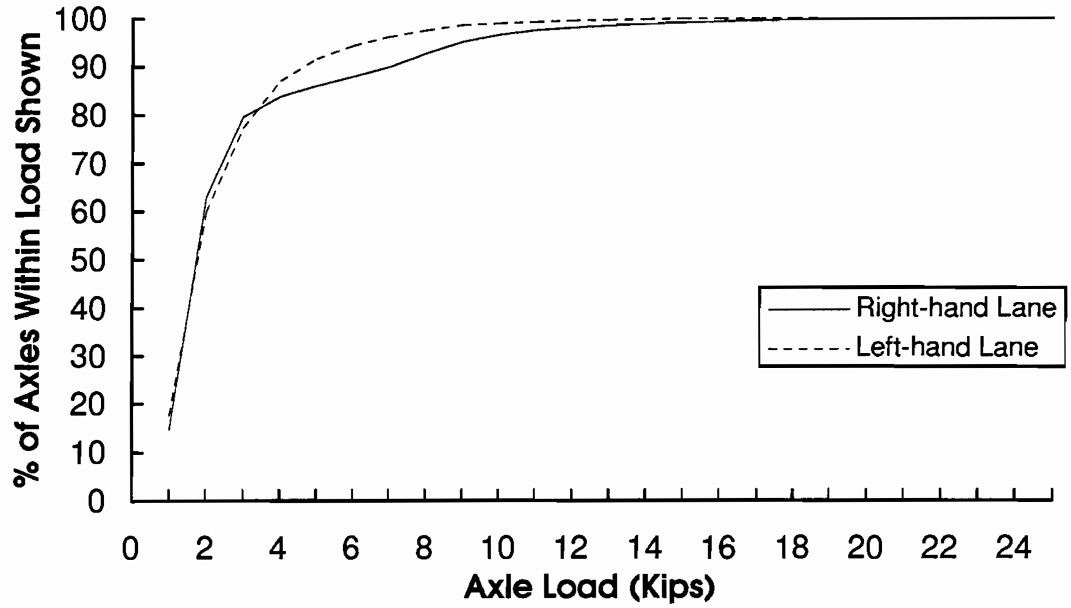

Figure B.10 Axle Load Distribution by Lane: 2-axle Trucks

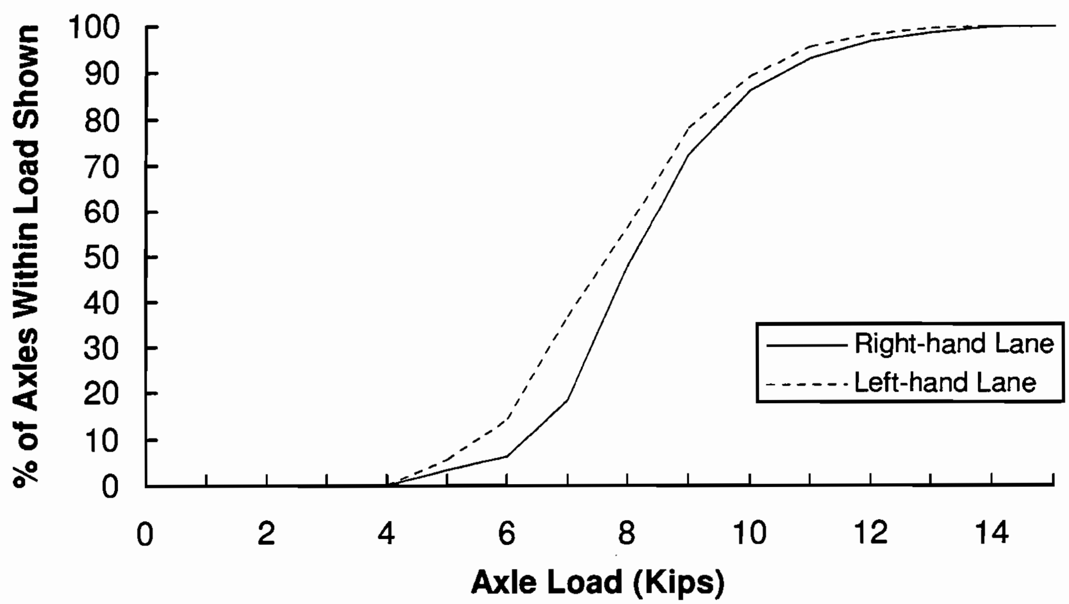

Figure B.11 Steering-Axle Load Distribution by Lane: 3-axle Single-Unit Trucks

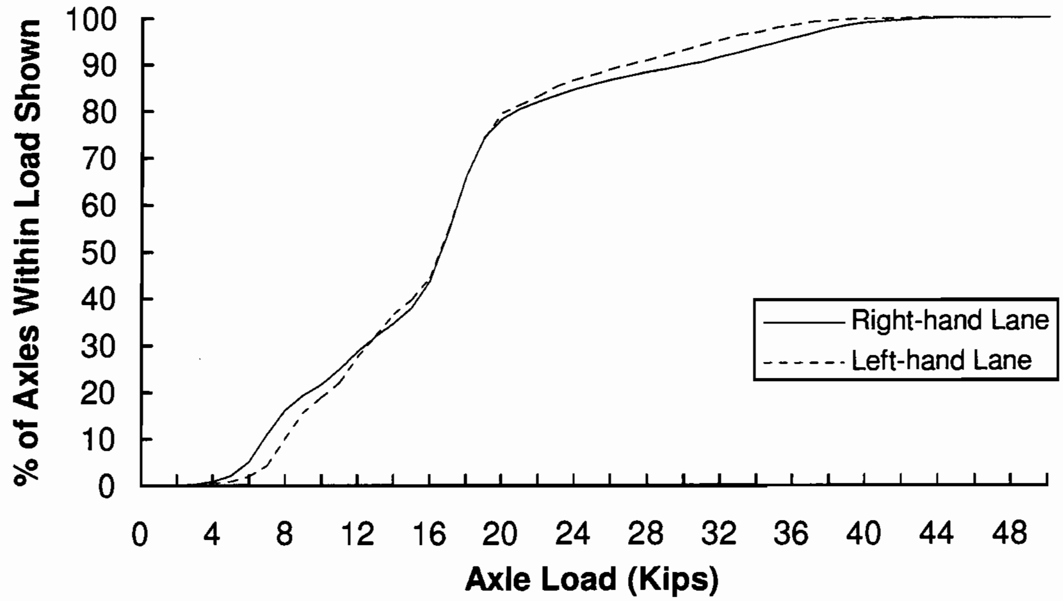

Figure B.12 Tandem Axle Load Distribution by Lane: 3-axle Single-Unit Trucks

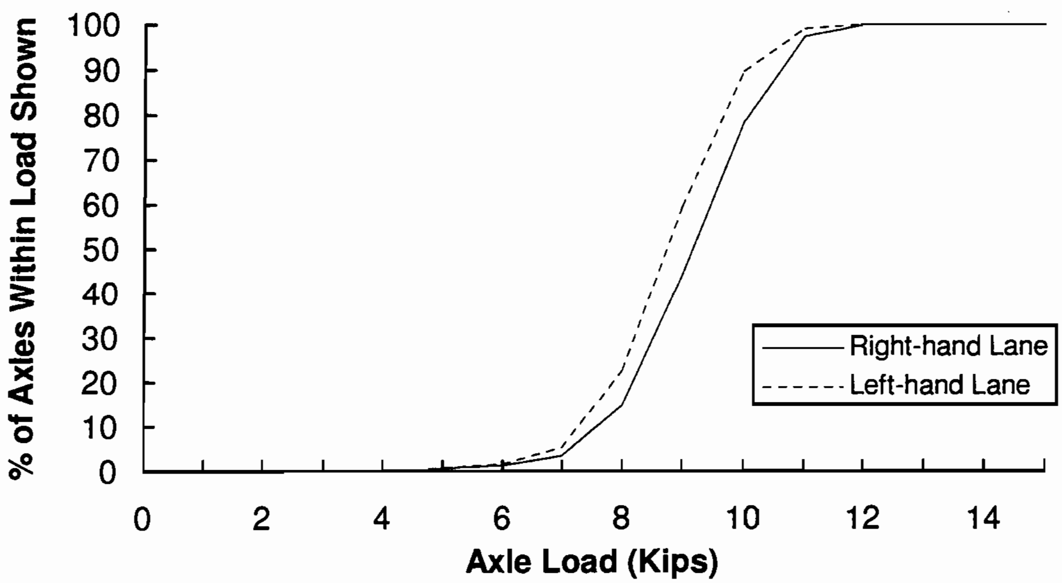

Figure B.13 Steering-Axle Load Distribution by Lane: 5-axle Semi-trailer Trucks (3S2)

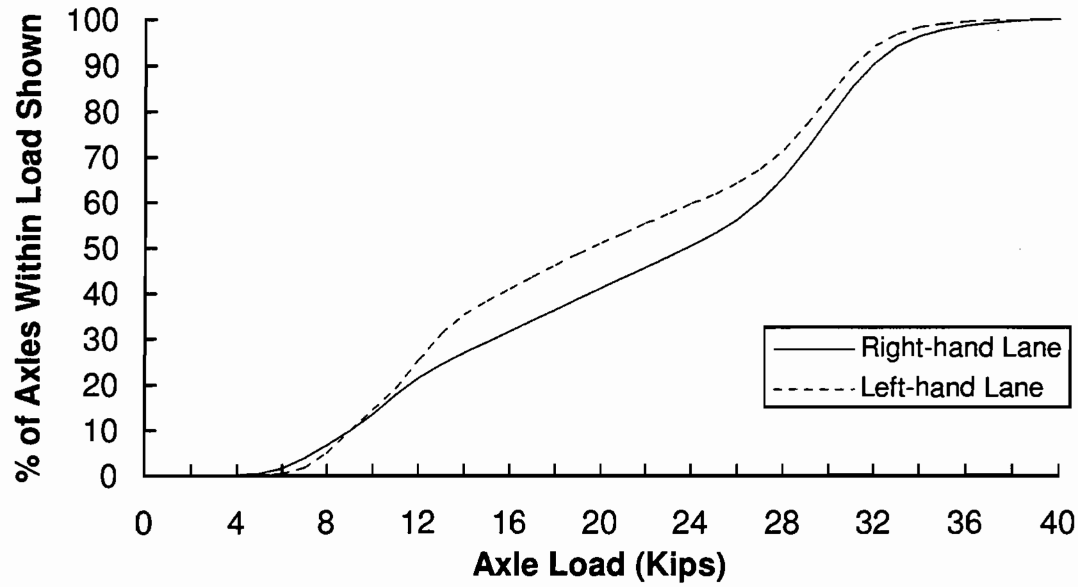

Figure B.14 Tandem Axle Load Distribution by Lane: 5-axle Semi-trailer Trucks (3S2)

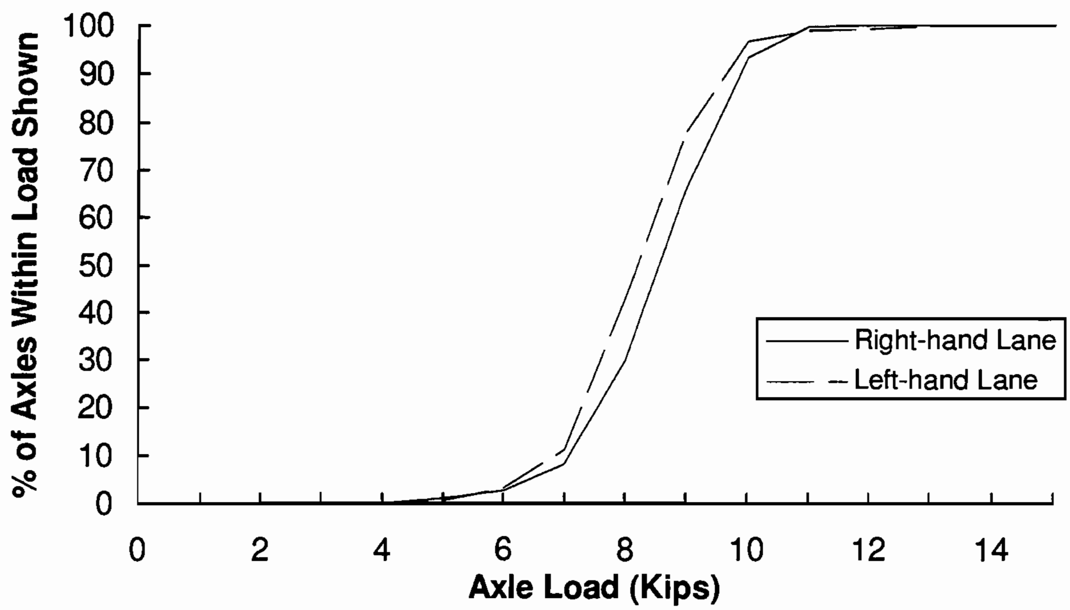

Figure B.15 Steering-Axle Load Distribution by Lane: 5-axle Semi-trailer trailer Trucks (2S1-2)

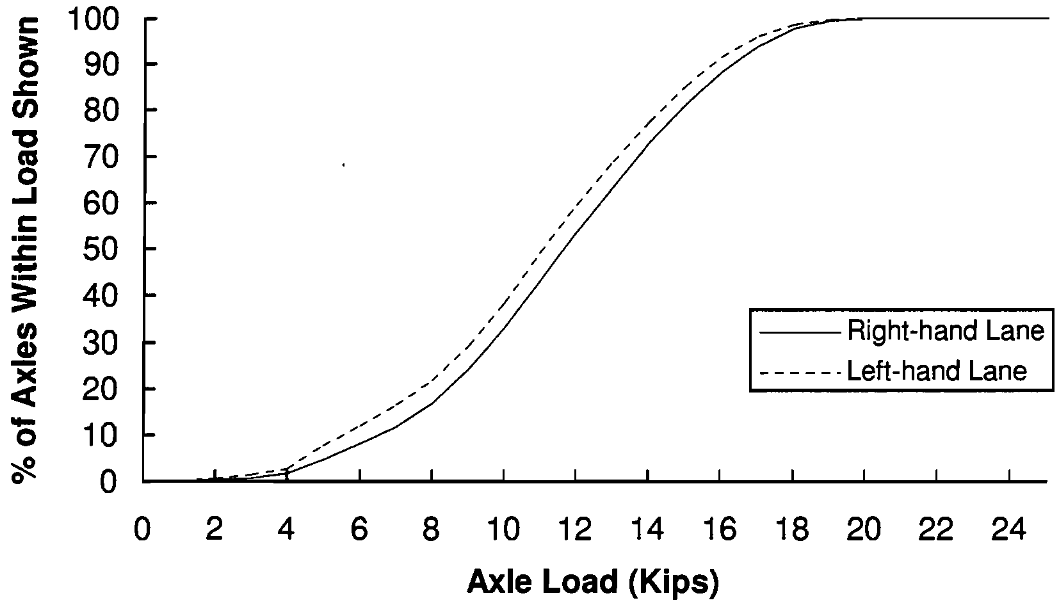

Figure B.16 Single-Axle Load Distribution by Lane: 5-axle Semi-trailer-trailer (2S1-2)

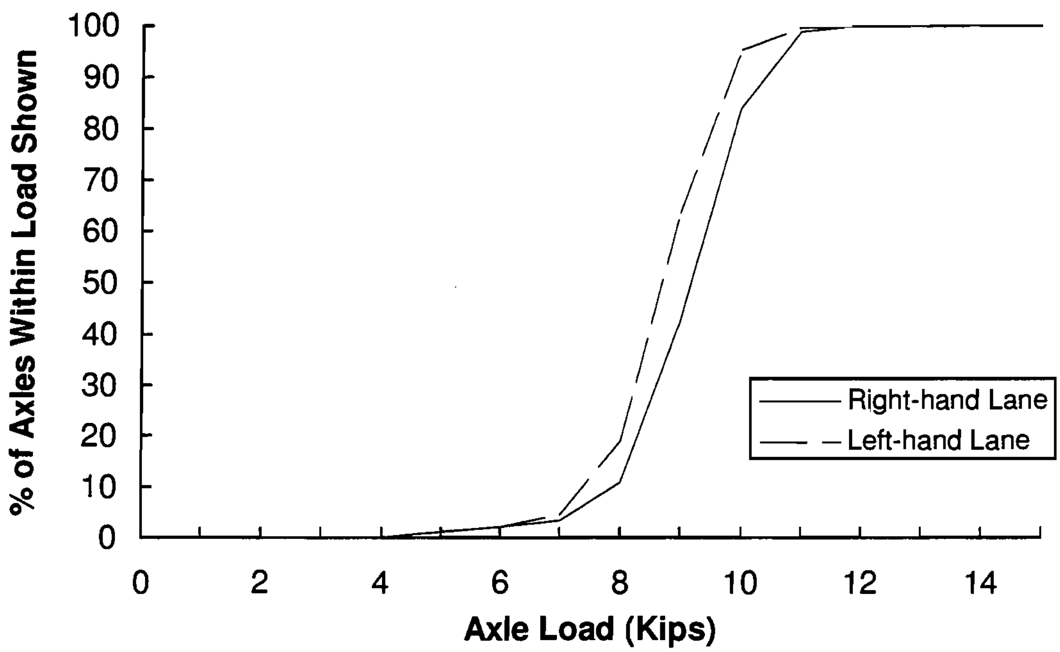

Figure B.17 Steering-Axle Load Distribution by Lane: 5-axle Semi-trailer Trucks (3S2 Spread)

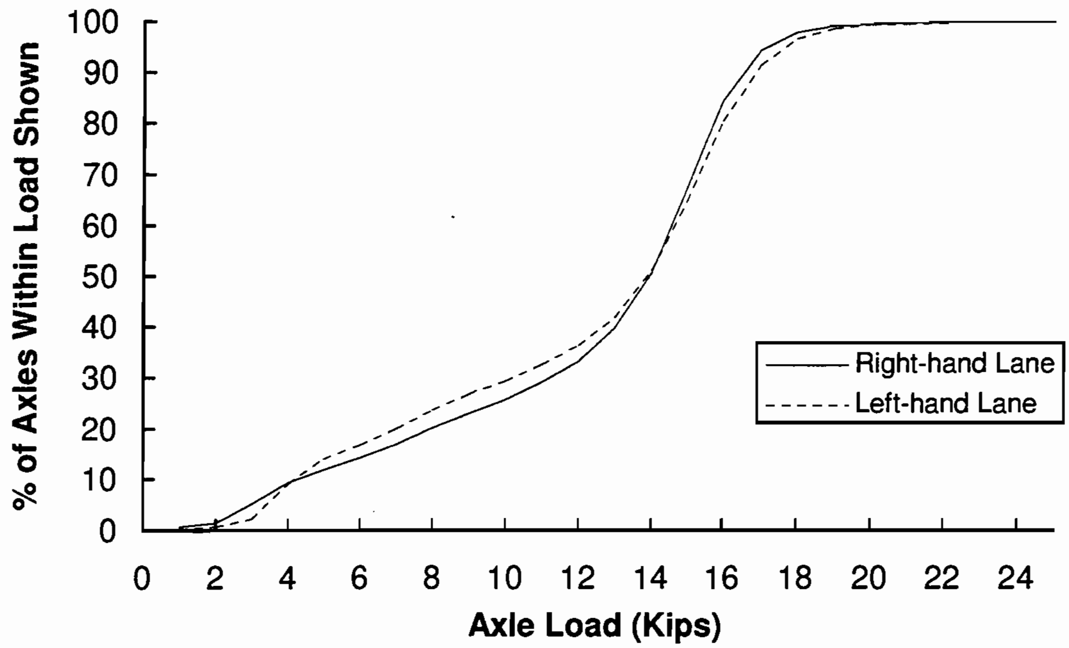

Figure B.18 Single Axle Load Distribution by Lane: 5-axle Semi-trailer Trucks (3S2 Spread)

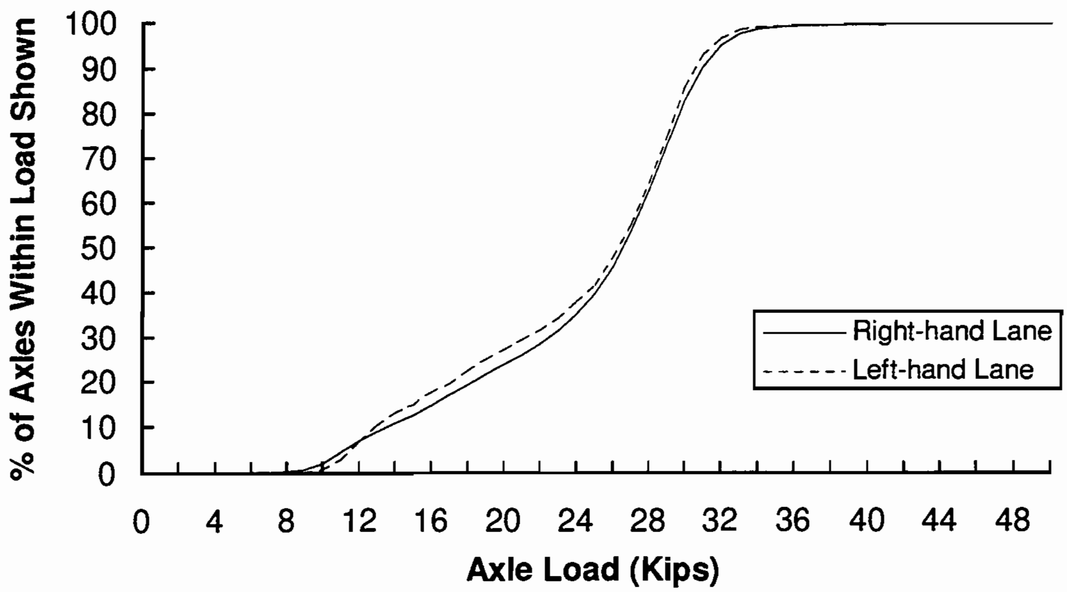

Figure B.19 Tandem Axle Load Distribution by Lane: 5-axle Semi-trailer Trucks (3S2 Spread)

APPENDIX C.

Traffic-Load Forecasting Macro Procedure

This appendix is included to facilitate the use of the esalform macro presented in Chapter 5 of this report. The following steps describe the general procedure and highlights areas where site-specific data should be used in generating traffic loading forecasts.

Before running the esalform macro, the esalform.xlm, esalfile.xls, wghtdst1.xls, and wghtdst2.xls files must be placed in a directory on the hard drive or on a floppy disk. It should be noted that a backup copy of these files should be made and the original copy of these four files should be kept on a separate disk and should not be used to run the program. A brief description of each file is contained below:

esalform.xlm - This file is a Micrsoft Excel macro that contains the executable code for performing the actual traffic loading forecast. It must be run from Microsoft Excel.

esalfile.xls - This file is a temporary spreadsheet file that is replaced each time the macro is run. It contains the input data and the final traffic loading data.

wghtdst1.xls - This spreadsheet file contains the axle load distributions for the vehicle classes in the right-hand lane. The first 55 rows contain the raw number of axles recorded while the following 55 rows contain the axle load distribution. Although not used in the forecast, a cumulative distribution is included in the last 55 rows.

wghtdst2.xls - This spreadsheet file is the same as above except for the vehicle classes in the left-hand lane.

Once Microsoft Excel is running, the esalform.xlm file can be opened by choosing the 'File: Open' menu command. To execute the esalform.xlm macro, choose the 'Macro: Run' menu command. If the "R1C2" cell is in the Reference line of the 'Run Macro' dialog box (Fig C.1) choose 'OK'.

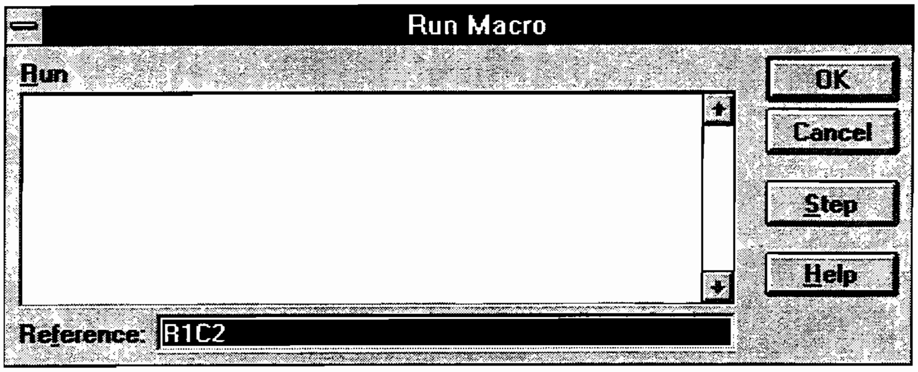

Figure C.1 Run Macro Dialog Box

Once the macro is started the user will be prompted to input both the pavement and traffic variables necessary to perform the traffic loading forecast. The first such input is the pavement type (Fig C.2), either Flexible or Rigid. By entering the number 1 (or choosing 'OK' to select the default of 1) a flexible pavement traffic loading forecast will be performed. Entering the number 2 will result in a rigid pavement traffic loading forecast.

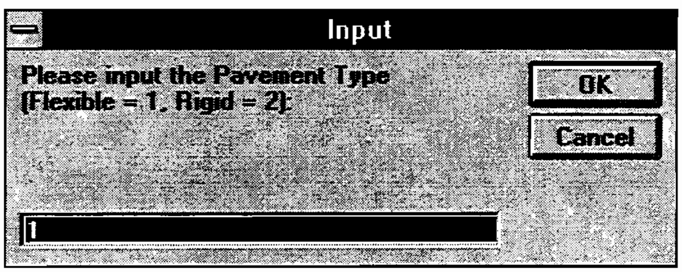

Figure C.2 Pavement Type Dialog Box

Depending on whether a flexible or rigid pavement was selected, the user will be prompted to input either the Structural Number (Fig C.3) or the Rigid Pavement Slab Thickness (Fig C.4). The default Structural Number is 5 and the default rigid pavement slab thickness is 6 inches, but either can be changed by the user by entering a different Structural Number or a different rigid pavement slab thickness in inches.

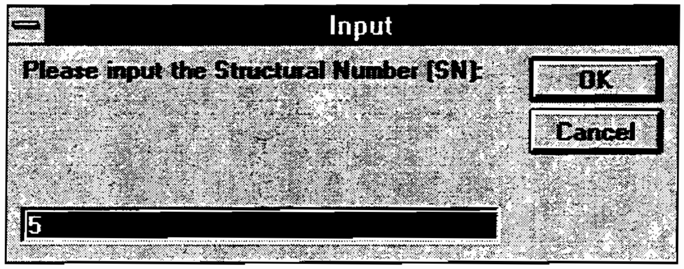

Figure C.3 Structural Number Dialog Box

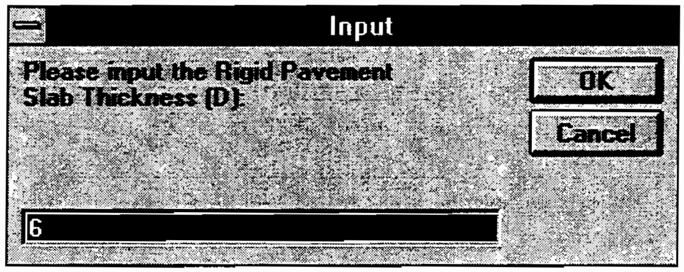

Figure C.4 Rigid Pavement Slab Thickness Dialog Box

The final pavement variable required is the terminal serviceability of the pavement (Fig C.5). The default is 2.5 but can be changed by the user.

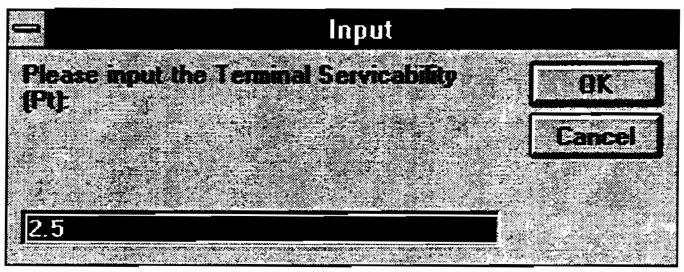

Figure C.5 Terminal Serviceability Dialog Box

All remaining input variables are traffic related. The first traffic variable is the annual average daily traffic (AADT) (Fig C.6). This AADT is a combination count of both directions of the highway in question. The AADT for the first year of the forecast should be input here.

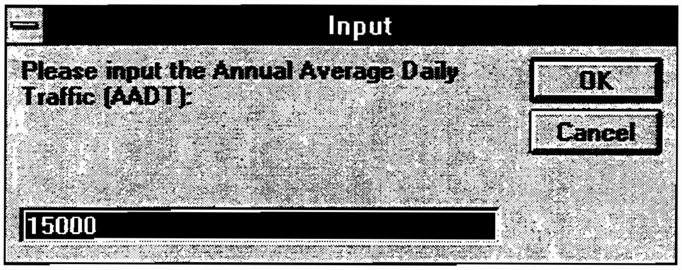

Figure C.6 AADT Dialog Box

The next input will distribute the AADT between the two travel directions of the highway. The percentage of the AADT traveling in the design direction should be input here and if the percentage is not known, the default of 50% should be used.

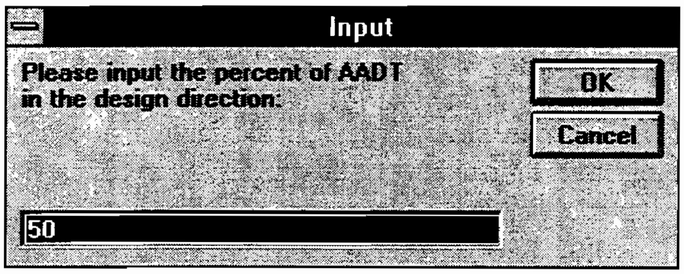

Figure C.7 AADT Directional Split Dialog Box

The next input will distribute the percentage of the AADT in the design direction between the right-hand and left-hand lanes. Typically pavement designs have placed 100% of the traffic in the design direction in the right-hand lane and used the resulting traffic loading forecast for both lanes. With lanewise traffic volume, classification, and loading information available, this program is able to provide separate loading forecasts for each lane separately. The default input of 70% will place 70% of the design direction AADT in the right-hand lane and the remaining 30% in the left-hand lane. To place 100% of the traffic in the right-hand lane, enter 100% in the input box (Fig C.8) and choose 'OK'.

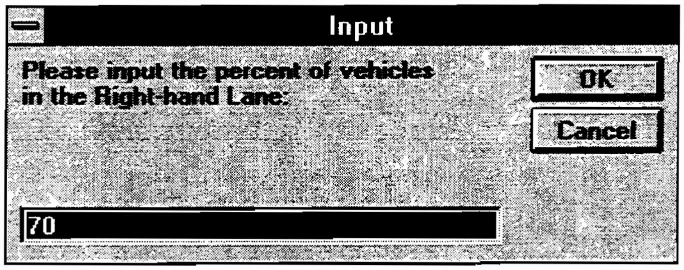

Figure C.8 Lanewise Distribution Dialog Box

The traffic loading analysis period is the next input (Fig C.9). Typically pavement design use a 20 year analysis period and this is the default. Any analysis period may be entered by the user.

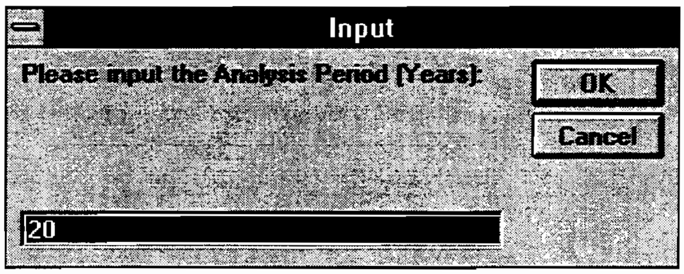

Figure C.9 Analysis Period Dialog Box

To account for the overall yearly growth in AADT the user is allowed to input a yearly compounding growth rate (Fig C.10). This growth rate will be applied to the AADT for each year of the analysis period entered above. By entering zero, the program will assume no growth and a simple growth factor equal to the analysis period will be used. Otherwise the relationship described in Chapter 5 will be used to account for the yearly compound growth. An annual growth rate of 2.5% is the default parameter.

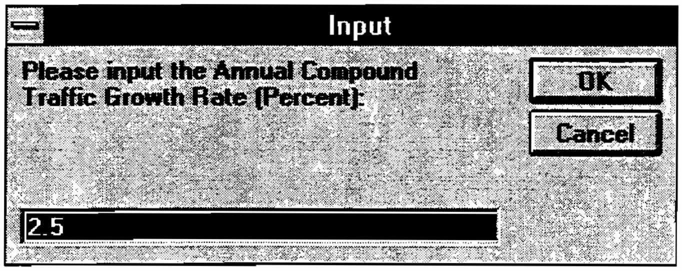

Figure C.10 Annual Growth Rate Dialog Box

The last input variable is the traffic classification distribution. The user is prompted (Fig C.11) to either accept the default lanewise distribution by choosing 'OK', or to edit the distribution by choosing 'Cancel'. If 'Cancel' is chosen the program will go to the classification distribution table and pause to allow the user to make any changes in the distribution. If only a directional classification is available, it should be used for both lanes by copying the distribution to both the 'R-hand %' and 'L-Hand %' columns. Once the changes are made, the user can resume the program by choosing the 'macro resume' button if one is present on the screen or by choosing the 'Macro: Resume' command mane. If the distribution changes made to not total 100% the user will be warned and the program will return to the distribution for changes. Once the distribution equals 100%, the program will resume and will continue with no further user input required.

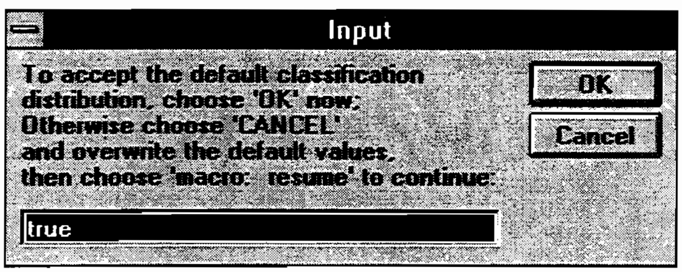

Figure C.11 Classification Distribution Dialog Box

After the conclusion of the macro, the total ESALs predicted by lane and for the design direction will be displayed (Table C.1). The spreadsheet will also identify the cumulative ESALs by vehicle class for each lane. If the user would like to keep the output, either the esalfile.xls file can be renamed and saved, or the summary statistics on the spreadsheet can be printed out in landscape format. Otherwise the next execution of the macro will overwrite the esalfile.xls file and any existing data will be lost.

The following is a sample forecast utilizing data listed below, the forecast output is included on the following page.

Pavement Type:	Rigid
Slab Thickness (D):	6 (inches)
Terminal Serviceability (pt):	2.5
AADT:	15,000
AADT Directional Split:	50 (Indicates 50% of the AADT is in the direction in question)
Lanewise Split:	70 (Indicates 70% of the directional traffic is in the right-hand lane)
Analysis Period (n):	20 (years)
Annual Growth Rate (g):	2.5 (%)
Vehicle Classification:	Default (Classification distribution recorded on southbound US 59)

Table C.1 Sample Traffic-Load Forecast, 1995-2015

Pavement	SN / D	Pt	Gt	B18		
Rigid	6	2.5	-0.17609	2.147879		
AADT	Dir. Dist.	Lane Dist.	n	g	G.F.	
15000	50	70	20	2.5	25.544658	
					ESALs	ESALs
		R-hand %	L-hand %		R-hand	L-hand
o-o	2-axle cars	55.54	63.31		184870	90314
o----o	2-axle trucks	21.63	26.57		247099	38422
o---oo	3-axle single-unit	1.07	0.91		203219	55935
o---o-----o	3-axle semi-trailer	0.28	0.17		48637	12656
o---oo------o	4-axle semi-trailer (3S-1)	0.08	0.03		30600	4918
o---o------oo	4-axle semi-trailer (2S-2)	0.52	0.16		100256	13221
o---o-----o-----o	4-axle double-trailer (2S-1-1)	0.24	0.03		68078	3647
o---oo------oo	5-axle semi-trailer (3S-2)	17.46	7.45		11296607	1665735
o---o-----o-o-----o	5-axle double-trailer (2S-1-2)	0.89	0.27		548804	61396
o---oo-----o-o	5-axle semi-trailer (3S-1-1)	1.90	0.88		1474206	290024
o---oo------ooo	6-axle semi-trailer (3S-3)	0.19	0.1		107546	24259
o---oo-----o-o-----o	6-axle double-trailer (3S-1-2)	0.14	0.07		39093	8377
	7 or more axle trucks	0.06	0.05		40393	14426
				ESALs =	14,389,410	2,283,328
			Total ESALs by Direction =			16,672,737