# FINAL REPORT

# PROJECT PLANNING MODELS FOR FLORIDA'S BRIDGE MANAGEMENT SYSTEM

Contract No. BC 352-9

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16. Abstract


The project planning tool for Florida's bridge management system will require updated user cost models specific to Florida in terms of truck weight and height characteristics, and also user costs for moveable bridge openings on Florida roadways. Using laser-based devices, truck height measurements were done at nine sites on Florida roadways. Truck weight data was collected simultaneously with the truck height data at three FDOT weigh stations, and also from FDOT's historical weigh in motion (WIM) data. For Pontis implementation at both network and project levels, truck height and weight histograms, and best-fitting linear and nonlinear functions were developed to estimate the probability of truck height or weight exceeding a specific value, for three categories: interstate roadways; non-interstate roadways; and all roadways. Bridge users cost models were formulated for both strengthening and raising improvements. Suitable data were collected, on bridge opening frequency and duration of opening at six moveable bridge sites on Florida highways. On-site data were collected on automobile and vessel traffic, including vehicle queue length and vessel height distributions. Queue models incorporating both vehicles and vessels, were developed in which the vehicular delay was modeled as a bottleneck occurrence on the roadway, where the service flow rate of vehicles is reduced due to a blockade. Decision-making templates were developed to correctly assign performance measures and priorities to moveable bridge replacement projects, including vessel and vehicular traffic future projection of 20 years. Recommendations are presented on implementing the user cost model in Pontis at both network and project levels. A project-level decision support software tool has been developed, incorporating Pontis network-level results along with all the products of the earlier research, to give FDOT bridge engineers a clear picture of the economic health of a bridge and the economic implications of scoping and timing decisions for structure maintenance, repairs, rehabilitation, improvement, and replacement. A comprehensive user manual as well as the software is included with this report. Named the Florida Project Level Analysis Tool, the highly graphical software sheds light on the scoping and timing decisions inherent in bridge life cycle decisions. Because it fills a significant gap in Pontis, other states have already expressed interest in implementing it.


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# METRIC CONVERSIONS

$$ft \times 0.3048 = m$$

$$ft^2 \times 0.09290 = m^2$$

$$yd^2 \times 0.000008361 = m^2$$

$$yd^3 \times 0.7646 = m^3$$

# DISCLAIMER

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# EXECUTIVE SUMMARY


The Florida Department of Transportation (FDOT) is implementing the AASHTOWare Pontis® Bridge Management System (BMS) as a decision support tool for planning and programming maintenance, repairs, rehabilitation, improvements, and replacement for more than 6,000 bridges on the state highway network. A BMS stores inventory and inspection data in a database, and uses engineering and economic models to predict the possible outcomes of policy and program decisions.

Previous Department research in the areas of user costs and agency costs have identified the remaining analytical needs for implementation of the economic models of Pontis, and have made significant progress in the development of these models. With the success of these research efforts, it was time to investigate several additional modeling issues and to develop methods for applying the results of the earlier research in actual FDOT bridge management decision-making. FDOT selected Florida State University (FSU), with subcontract support from Paul D. Thompson, to develop a project planning tool and also fill in other remaining gaps in order to make Pontis a valuable planning tool for FDOT bridge engineers. The project planning tool will require updated user cost models specific to Florida in terms of truck weight and height characteristics, and also moveable bridge openings on Florida roadways.

Extensive literature review was initially done, including study and updating of pertinent previous research done by FDOT. Descriptive truck height and weight models were developed for user cost modeling of bridges on Florida highways. Using laser-based devices: a handheld range finder, and installed vehicle scanners at FDOT weigh stations, truck height measurements were done at nine sites on Florida roadways, with the sites reflecting a variation in geographical location and roadway functional class. Truck weight data was collected simultaneously with the truck height data at the three FDOT weigh stations (Interstate Highways), by integrating the vehicle scanner into the weigh-in-motion (WIM) data collection system. Historical WIM data on truck weight were also obtained form FDOT, for additional eight sites, with all the 11 sites for truck weight data reflecting geographical location diversity and functional class variation. The data were analyzed for each site, to develop truck weight and height histograms and also generate curves, including piece-wise linear and nonlinear functions.

The results indicate that about 80% of the trucks, on the interstate roadways have heights between 13 ft. and 14 ft. For the non-interstate roadways, truck heights were observed to have an approximate triangular distribution, starting from 7 ft. to 13.5 ft. at a peak about 15%. The truck weight data showed interstate roadways carrying trucks with their gross weights represented by a triangular distribution from 4,000 lbs. to 80,000 lbs. at a peak of about 25%. The non-interstate data showed a left- skewed distribution, from 4,000 lbs. to 80,000 lbs., and a peak of about 25% of trucks with 30,000 lbs. weight. A combined data of the truck weight for all roadways indicated an approximate uniform distribution (15%) between 4,000 lbs. and 80,000 lbs. For Pontis implementation at both network and project levels, best-fitting linear and nonlinear functions were developed to estimate the probability of truck height or weight exceeding a

specific value, for three categories: interstate roadways; non-interstate roadways; and all roadways. User cost model input parameters were updated for vehicle operating costs and travel time cost. Bridge user cost models were formulated for both strengthening and raising improvements; the procedure was implemented in Microsoft Excel spreadsheets.

Also developed in the study was a user cost model for moveable bridge openings, which quantifies the economic impact of bridge openings, specifically the lost time due to delay of motorists. Working with the pertinent FDOT Offices, the researcher obtained suitable data on bridge opening frequency and duration of opening, for six moveable bridge sites on Florida highways. The locations were geographically spread, and also reflective of the variation in traffic and functional class of the roadways. On-site data were collected on automobile and vessel traffic, including vehicle queue length and vessel height distributions.

Queue models incorporating both vehicles and vessels, were developed in which the vehicular delay was modeled as a bottleneck occurrence on the roadway, where the service flow rate of vehicles is reduced due to a blockade, which in this case would be the opening of the bridge. Decision making templates have been developed to correctly assign performance measures and priorities to moveable bridge replacement projects, including vessel and vehicular traffic future projection of 20 years. Bridge replacement models considered higher moveable bridge options as well 65 ft. fixed bridges, with input data such as bridge geometrics, number of bridge openings, agency life cycle costs, and user delay costs. Also, the researcher has suggested modifications to the Pontis software that would be required in order to implement the user cost model at both network and project levels, for movable bridge openings, by considering vehicular and vessel delays, and the load carrying capacity of existing bridges.

To aid in the implementation of several recent FDOT research efforts, a project-level decision support software tool has been developed, incorporating Pontis network-level results along with all the products of the earlier research, to give FDOT bridge engineers a clear picture of the economic health of a bridge and the economic implications of scoping and timing decisions for structure maintenance, repairs, rehabilitation, improvement, and replacement. A comprehensive user manual as well as the software is included with this report.

The decision support tool is designed to be compatible with, and take advantage of, the existing Pontis network-level models, but is intended to be used as a part of project-level decision-making. This means adapting the Pontis economic definitions and life cycle cost model so that they are most useful in the context of individual structures, adding a few additional sub-models to address certain project-level concerns, and building a display tool that is informative for scoping and timing decisions.

Named the Florida Project Level Analysis Tool, the highly graphical software sheds light on the scoping and timing decisions inherent in bridge life cycle decisions. Because it fills a significant gap in Pontis, other states have already expressed interest in implementing it.

# 1. Research Background

This section presents the current status of knowledge both in terms of research activities and industry practice in developing project planning models in bridge management systems (BMS), including modeling of user costs and locating sources of pertinent data required to develop the models. But first, an overview of the project planning model study is presented.

## 1.1 Introduction and Research Objectives

The Florida Department of Transportation (FDOT) is currently in the process of implementing the AASHTO Pontis Bridge Management System (BMS) to support network-level and project-level decision making in the headquarters and district offices. Pontis is an integral part of a Department-wide effort to improve the quality of asset management information provided to decision makers. Pontis is also the most popular BMS in the industry, selected by approximately 40 state DOTs for their BMS. The essence of a BMS is to provide a decision support tool to maximize the benefit of investment in bridge structures by combining engineering, economic, and management analysis to determine the most cost effective mix of routine maintenance, periodic maintenance, rehabilitation, replacement, and other actions over the life of the structures. The credibility and usefulness of this information is also essential for satisfaction of the requirements of the Government Accounting Standards Board Statement 34 (GASB 34) regarding the reporting of capital assets.

The BMS requires various types of data to function, including the following: deterioration data, user cost data, and agency cost data. Bridge elements typically deteriorate over time. The deterioration is a function of many factors, including: material, environment, design practices, construction practices, and maintenance practices. Deterioration data are derived from condition information collected from periodic inspections of the bridges. The data collected by bridge inspectors are stored in a large database and are used to write inspection reports that form the basis for a variety of decisions and analyses concerning the bridge inventory

The most effective action recommended to correct bridge deficiencies will be influenced by the need to minimize user (public) costs. Also required for the BMS to function is agency cost data. Agency, or FDOT costs typically include the engineering, right-of-way, construction, and maintenance costs associated with the various bridge projects. As part of the FDOT's implementation effort on Pontis, two projects have been successfully completed to provide these required data: "Development of User Cost Data for Florida's Bridge Management System" and "Development Of Agency Maintenance, Repair & Rehabilitation (MR&R) Cost Data For Florida's Bridge Management System."

These previous Department research projects have identified the remaining analytical needs for implementation of the economic models of Pontis, and have made significant progress in the development of these models. With the success of these research efforts, it is now time to investigate several additional modeling issues and to develop methods for applying the results of the earlier research in actual FDOT bridge management decision making. The objectives of this study are briefly narrated in the following paragraphs.

As indicated in the final report of the Florida DOT User Cost Study, truck height and weight histograms were the only area where the study was unable to find satisfactory Florida-specific data.

These data are important for Pontis implementation, especially for local roads. It will be necessary to develop descriptive models of truck height and weight relevant to bridge management. The truck weight data would also be compared to the blanket vehicle groups used by the FDOT Road Use Permits Office, comparing moments and shears caused by typical permitted overweight vehicles. As also indicated in the final report of the Florida DOT User Cost Study, Florida is interested in developing a user cost model for movable bridge openings, to help justify replacement projects for its large inventory of movable bridges.

The major objective is to develop a project planning model that incorporates the results of the tasks described in the preceding paragraph, and of earlier FDOT research. This model, to be implemented in a user-friendly framework in Microsoft Excel for convenient use, will describe and perform all the calculations necessary to develop estimates of bridge project costs and benefits compatible with Pontis. The models already developed under the FDOT user cost and agency cost studies will be included in this overall framework. Engineers will be able to use the product as a planning tool by viewing model recommendations, adjusting them as necessary to incorporate practical concerns, viewing the calculated cost and performance effects of the work and saving the final selections. The model will automatically load relevant data from Pontis.

## 1.2 Literature Review and Relevant Data

The literature review was started with a computerized search of the Florida State University libraries' catalogs including databases such as the Applied Science and Engineering, Compendex and Engineering Index, and the Elsevier Science Journals. Also utilized in the search was the Online Computer Library Center (OCLC) through First Search services (Dissertation Abstracts). Various search engines were also utilized to search the Internet for related documents, including Internet-based libraries maintained by the Federal Highway Administration (FHWA) and the United States Department of Transportation (USDOT), i.e., the National Transportation Library and the Transportation Research Information Service (TRIS), Transportation Research Board (TRB), and Florida Department of Transportation (FDOT).

### 1.2.1 Truck Height and Weight Models

In Pontis, the user cost model predicts user benefits in terms of the functional improvements; bridge deck widening and approach alignment improvement, bridge raising and bridge strengthening. In order to make these predictions for the State of Florida bridge network it is imperative that Florida specific truck height and truck weight models are developed for the Florida bridge inventory.

A truck height model in a BMS indicates a distribution of various heights of trucks in a traffic stream on the highway, expected to pass under the bridges. The model is eventually utilized to measure the additional cost incurred by vehicles (predominantly trucks) that are forced to make a detour when they encounter a bridge that has inadequate under clearance. In a manner similar to the height model, the truck weight model in a BMS indicates a distribution of gross weight of trucks on the highway. It is used to measure the additional cost incurred by trucks that have to make a detour at a bridge that has been posted due to load capacity deficiency. Pontis calculates the vehicle operating costs and travel time costs associated with vehicles that have to take detour routes due to the load capacity and height deficiencies of the bridge, and assumes that this cost is saved if functional improvements are undertaken.

Several studies were found relating to general issues and modeling of truck weights, including March (2001), FDOT (1998), Najafi et al (1999), FHWA (1999), Battelle (1999), Fekpe and Blow (2000), Wang and Liu (2000), Moses et al. (1990), and Nowak et al. (1994). The most relevant source of information was the recent study reported in Thompson et al. (1999, 2000), describing the results of a user cost modeling effort for Florida bridges. Studies related to user cost models for bridges include Schelling (1985), Chen and Johnston (1987), Moses (1992), Ben-Akiva and Gopinath (1995), while work zone user cost model during roadway construction is described in Ellis and Herbsman (1997). Two Internet web sites, both hosted by the Federal Highway Administration (FHWA), were found to be very useful on information related to traffic volume and truck weight (FHWA 2001), and freight management and operations (FHWA 2002). Several articles are available for free download, on these web sites, as well as valuable links to other useful sites.

Contacts were made with various FHWA (David Jones) and FDOT offices (Barry Mason of Motor Compliance Office and Rick Reel (Traffic Office) to discuss availability of pertinent data on truck height and weight, including data collection at Weigh Stations. Since there are no WIM sites on local roads, the FHWA's Vehicle Travel Information System (VTRIS) software and VMT data were also reviewed as potential sources of relevant data.

Measurement of truck heights by modern technologies was investigated through a review of overhead laser scanners such as Schwartz Electro-Optics Inc.'s Autosense II and III sensors and Autosense II's evaluation by Harlow and Peng (2001); Hexamite's ultrasonic devices (2001); Laser Technology, Inc.'s Impulse laser rangefinder (2001); and laser-based "light curtains" vehicle scanners by Scientific Technologies, Inc. (STI) (2001).

Sources of truck costs related to the user cost models include the AASHTO Red book (1977), NCHRP 133 (1972), MicroBencost in NCHRP 7-12 (1993), FHWA's HERS (1996), and the American Truck Association (ATA) / FTA publication "American Trucking Trends" (2002).

### 1.2.2 User Cost of Moveable Bridge Openings

The State of Florida is also interested in developing user cost models for its large inventory of movable bridges in order to justify improvements and replacement projects. A user cost model relating to moveable bridge openings would also measure the additional cost incurred by vehicular traffic due to the opening of the bridge for vessels, and additional costs incurred by vessels due to delays caused by the closing of the bridge for vehicular traffic.

The identified work in the area of movable bridge user cost models were related to the formulation of queuing and delay models for vehicular and vessel traffic at movable bridge openings. A significant number of literature sources were found relating to the maintenance and inspection procedures for moveable bridges, but these procedures are not related directly to the collection and analysis of user cost data compatible with the Pontis BMS software requirements. The most relevant document found was Delgani et al. (1993) reporting a case study in Florida, estimated delays to boats and vehicular traffic caused by movable bridge openings, by empirical analysis. The estimation was used in the economic analysis to rank the proposed replacement facilities to the existing movable bridge. Replacement options identified include providing a fixed span bridge with higher vertical clearance, providing a tunnel, considering various height options for a higher level movable bridge and installation of movable concrete barriers that can be moved to provide increased vehicular capacity over the bridge during the peak hours. The Delgani et al. (1993)'s methodology is described in more details in section 3 of this report, presenting the results of the user cost models for moveable bridges.

### 1.2.3 Project Planning Models

Many efforts have been documented on developing network level models of bridge management, including recently by Gurenich and Vlahos (2000). Project level models have also been recently presented by Sinha et al. (2000), Soderqvist and Veijola (2000), Marshall et al. (2000), and Banks (2000). But only one project planning model, specific to Pontis bridge management system, was found, discussed by Anderson and Kivisto (2000) for the Minnesota bridge management system. A review of these prior efforts has been made, with the pertinent features evaluated. The beneficial aspects will be incorporated into the proposed project planning model for Florida bridges.

# 2. Truck Height and Weight Models

This section presents descriptive models of truck height and weight relevant to bridge management in general, and for Pontis in particular. This task included working with the pertinent FDOT offices and weigh-in-motion sites, to collect data, and the analysis of these data. As indicated in the final report of the Florida DOT User Cost Study (Thompson et al. 1999), truck height and weight histograms were the only area where the study was unable to find satisfactory Florida-specific data. These data are important for Pontis implementation. Sites were selected to reflect variation in locations and functional classes of highways, using the major Florida geographical regions, and the statistical distributions of the functional classes of bridges' primary roadways, including under pass roadways, as indicated in the Florida bridge inventory. A preliminary schedule was generated for dates of data collection based on the traffic time-based variations as indicated in the FDOT traffic records. Truck height data were collected at nine sites using both handheld laser range finders and infra-red "light curtain" based vehicle scanners, installed at weigh stations. Using weigh-in-motion data, truck weight information was collected at 11 sites, including axle weights and spacing data. Simultaneous weight and height data were obtained at three sites, along three major highway (Interstate) corridors in Florida. The data were analyzed to develop truck weight and height histograms and also generate curves, including piece-wise linear functions of the probability of truck height or weight exceeding a specific value. Finally, a preliminary study was done to develop moment envelopes due to truck axle loadings.

## 2.1 Site Selection

In consultation with the FDOT Permit Office and the Office of Motor Carrier Compliance (OMCC), the researcher located weigh-in-motion sites relevant to this study, on Florida roads, using the regional (geographical) locations and highway functional classification as the major criteria. Using FDOT traffic data, the researcher first reviewed the potential sites, identifying the current vehicle classification (percent cars, trucks, etc.) for each functional class. A statistical distribution of bridge data was then conducted to evaluate the important functional roadway classes on which bridges are located with respect to both the primary roadway and the under pass roadway. The primary roadway was relevant for truck weight data while the traffic for truck height limitations are indicated by the under pass roadway. Using daily and hourly traffic distribution data, the researcher identified dates of the year, i.e. day of the week and time of the day, which would ensure a statistically unbiased sample. Due to unavailability of daily traffic data at many pertinent potential sites, and delay on getting access and equipment installed at the sites, the scheduled data collection dates could not be strictly followed.

According to Florida Traffic Information (FTI) (1999), the FDOT Transportation Statistics Office collects traffic data at over 14,000 Traffic Monitoring Sites (TMS) throughout the state of Florida. The data is compiled annually and published as a CD-

ROM called FTI for the public domain. The TMS sites fall into two categories, Portable Traffic Monitoring Sites (PTMS) and Telemetered Traffic Monitoring Sites (TTMS). The TTMS are permanent and provide continuous monitoring of the distribution and variation of traffic flow. Eight reports can be generated for each TTMS and can be printed or exported to a spreadsheet format. These reports are: Annual Average Daily Traffic, 200 Highest Hour Annual Vehicle Classification, Peak season Factor Category, Historical AADT, Hourly Continuous Count, Weekly Axle Factor category, and Volume Factor Category summary. There are currently 386 TTMS sites statewide. For all other locations traffic statistics are estimated from the PTMS sites. PTMS sites do not provide permanent continuous monitoring. They are set up turned on or off for short periods, for example 1 or 2 weeks depending on the traffic monitoring objectives of the FDOT.

The FTI contains a TTMS Microsoft Access database. The records were exported to an Excel file. The TTMS were grouped by functional classification of their roadway and the parameters AADT and T-factor were looked at. The T-factor is the fraction of trucks in the AADT. By simple Excel spreadsheet manipulations the TTMS with the highest number of trucks (average daily tuck traffic – ADTT) were identified for each functional class of roadway. This was done by multiplying AADT by T-factor. This generated a list of potential data collection sties, including Weigh-In-Motion (WIM) sites for limited-access roadways and traffic collection points for the lower class roads. The 200 Highest Hour Report for the TTMS's with the highest ADTT for each functional classification of roadway was then studied. The 200 Highest Hour Report is an annual report that provides traffic count information, including dates, for the highest 200 hours of the year at all TTMS's. The information for each site includes location, direction of travel, hour of data, Directional Distribution (D), K-factor, the proportion of AADT occurring in hour with the Design D ($D_{30}$) and Design K ($K_{30}$), indicating the respective values for the 30$^{th}$ highest hour of the design year. By plotting AADT by date the traffic stream trend for the year was clearly depicted (Fig 2.1). By inspection of the graph the periods of peak flow, median flow and off-peak flow were clearly identified. From Figure 2.1 the study period proposed for FTI WIM station 9901 was the month of December. The median period, August through September was used for an alternative schedule in the event of unforeseen circumstances. From the preceding process the study locations and periods were chosen for each functional class of roadway.

To account for geographic distribution the state of Florida was considered as four regions; the Northwest (panhandle), northeast, southwest and southeast. Study locations for each functional class were chosen to cover as many geographic zones as possible. The FDOT and FWHA classify roadways into twelve functional classes based on their mobility and access characteristics (Table 2.1).

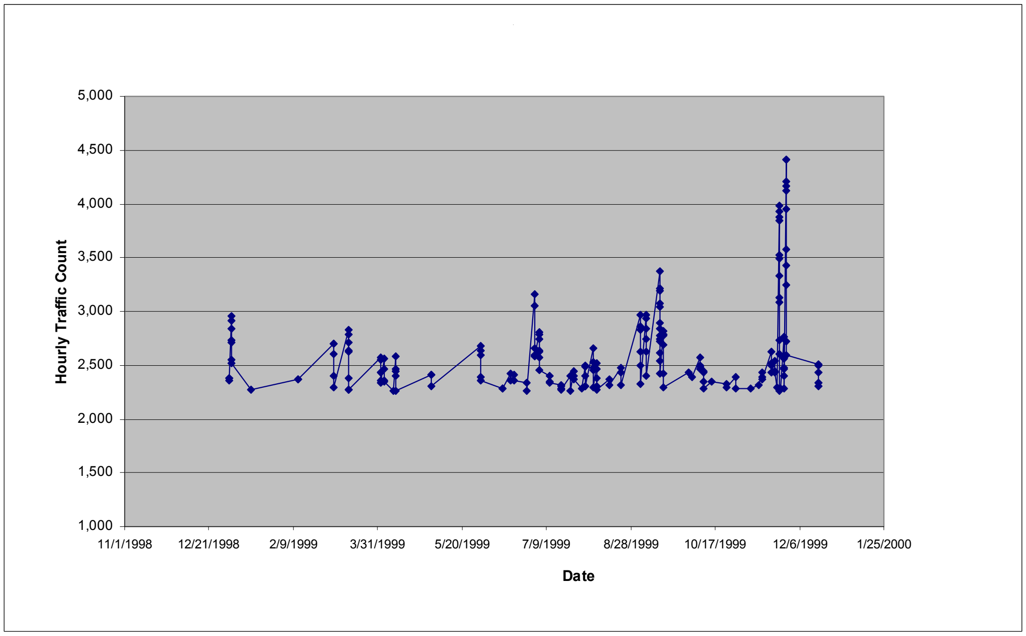



Figure 2.1. 200 Highest Hours of Traffic for 1999 for FTI WIM Station 9901, Lake City, Florida.

Table 2.1: Roadway Functional Classification (Source: FHWA/FDOT)

| | **Functional Class** | ***Description*** |
|---|---|---|
| Rural | 01 | Principal Arterial-Interstate |
| | 02 | Principal Arterial-Other |
| | 06 | Minor Arterial |
| | 07 | Major Collector |
| | 08 | Minor Collector |
| | 09 | Local |
| Urban | 11 | Principal Arterial-Interstate |
| | 12 | Principal Arterial-Other Freeways or Expressways |
| | 14 | Other Principal Arterial |
| | 16 | Minor Arterial |
| | 17 | Collector |
| | 19 | Local |

According to the FDOT's OMCC Office, Florida has 18 highway weigh stations statewide for the enforcement of truck weight and height regulations. The types of weighing facilities vary in sophistication. On lower classification routes they consist mainly of an electronic static scale, onto which trucks suspected of being over weight limit are directed at the discretion of local law enforcement officers. The most sophisticated facilities however are on interstate highways. These consist of a truck enforcement station built into the highway alignment. All trucks are required to enter the station, maintaining a speed of 45 miles per hour. A legal limit height scanner set at 13.5 feet above the pavement surface checks for compliance of legal height but does not measure the exact height of the truck. The vehicle then passes over a weigh-in-motion plate for legal load compliance. An overhead traffic signal directs violation vehicles and vehicles close to the legal limit onto a static scale where they are re-checked for compliance. Vehicles that are well within the legal limit are directed onto a bypass lane leading directly to a re-entry into the highway.

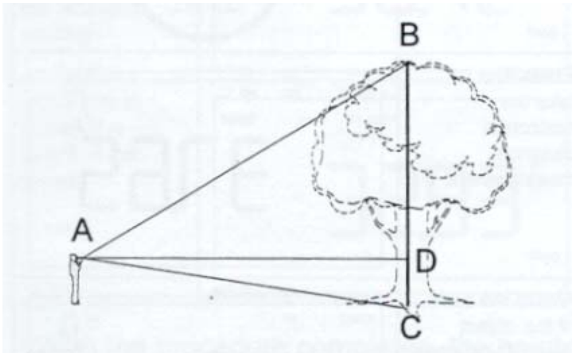
## 2.2 Equipment Selection

From the literature review it became apparent that a truck height survey of this nature had never been undertaken before in the state of Florida. Thompson et al (1999) described a methodology by which truck height data can be collected using a surveyor's transit at weigh-in-motion sites. An initial experiment with this method at one the eventual sites (FTI# 559908 – US 319 Capital Circle, Tallahassee, Florida), failed to capture a representative sample of truck heights on the roadway. This was because the setup of the instrument, the sighting, and focusing of the object required a significant amount of time in which many trucks would be missed. Secondly at weigh-in-motion sites, vehicles are traveling at about 45 mph. Sighting a freight truck traveling at that speed and measuring its height becomes an extremely difficult task and an impractical method. Even at lower speeds, it was still not practical to use the surveyor's transit. Two methods were therefore adopted -- the use of a hand held laser range finder, and an automated vehicle scanner, based on the "light curtain" principle. Calibration was done for both equipment (rangefinder and vehicle scanner); the results are shown in Appendix B, along with other pertinent information on the data collection effort.

### 2.2.1. Laser Range Finder

The laser range finder is capable of measuring truck height of vehicles pulled onto the static scale and vehicles in the bypass lanes. The laser range finder, shown in Figure 2.2, is also capable of accurately measuring truck heights on all routes at any time of day or night for vehicles traveling up to 60 miles per hour. In this study it was used at weigh stations on both major roads and traffic data sites on local roads. The device uses a laser beam to determine distances and angles and calculates the height of the object above a chosen reference point (Figure 2.3). The equipment provides a versatile, economical and accurate measurement within the requirements of this study.


Figure 2.2: Impulse™ Laser Range Finder Level [ASC Scientific, 2002]




*Height measurement: The instrument first calculates AD, then measures angles CAD and DAB. It then calculates BD and DC. The height is the sum of BD and DC.*

Figure 2.3: Laser Range Finder Operation [Impulse 1998]

The measurement procedure is very simple as demonstrated in Figure 2.3, from the manufacturer's manual – First, use the range finder to sight the truck directly (horizontally), then sight the lowest part of the truck (tire or pavement surface), and finally sight the highest point on the truck. The data collection process involved two persons, one of whom operated the instrument and took the readings while the other recorded the values and truck classification data. In using the range finder at two lane undivided local roads, some geometric adjustment had to be made to account for the roadway cross slope and lane/median width. The height measurements were undertaken by first establishing a datum with the closest lane to the observer. This involves fixing the horizontal distance from the instrument to the center of the near lane. This was done by sighting a target (measuring staff) at the center of the near lane. The data was entered on a pre-designed data entry sheet. The initial height measurements had to be corrected to make up for the differences in the fixed horizontal distances from the instrument positions and the actual truck positions on the road lanes. As shown in Figure 2.4 for a two lane road, the recorded values for trucks in Lane 1 are the actual truck heights while the corrected heights of trucks in Lane 2 were adjusted for roadway cross slope and lane width (Lane 1 Truck Height = AC; Lane 2 Truck Height, $A'C' = (H + 12) \tan \Theta_2$ )
At weigh stations, truck traffic is regulated by law to travel at 45 mph or lower, and also in a single lane. However for non-weigh station sites, the method describe above presented some limitations, because there was no restriction or interference with the traffic stream. For trucks traveling at excessive speeds, this made sighting of topmost parts of the trucks very difficult and for some trucks impossible. Cases of two or more trucks following each other closely resulted in some trucks being omitted during the period of the data collection. Overall the method can be considered accurate and useful. Figure 2.5 shows a demonstration using the range finder.

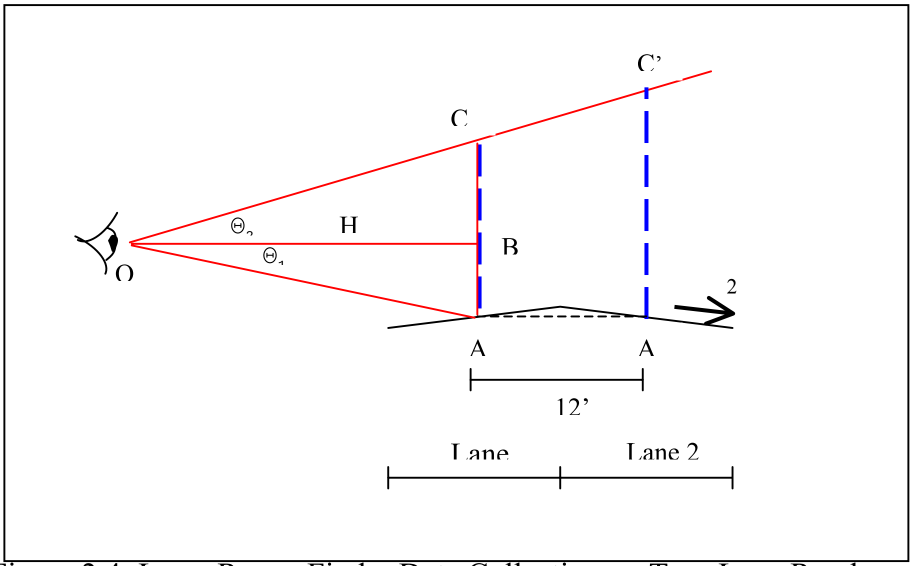



Figure 2.4: Laser Range Finder Data Collection on Two-Lane Road


Figure 2.5: Demonstration of Range Finder at Weigh Station

**2.2.2. Automatic Vehicle Scanner "Light Curtain"**

This is a high speed profiling scanner device designed to perform vehicle detection and classification. As shown in Figure 2.6, it consists of a transmitter, receiver, a controller, two tower environmental enclosures and interconnecting cables. It also has a power supply and controller assembly, which drives the transmitter, interprets the receiver's data, and control the various outputs. The transmitter houses a linear array of LED's (light emitting diodes) while the receiver has a linear array of photo detectors. The receiver detects which transmitter beams are blocked when an object passes through the scanning zone; the specific beams blocked will be interpreted by the system software to estimate the object height. The device comes in various scanning lengths, 2, 3, 4, 5, and 6 feet, each of which coming with beam spacing of 0.1 in, 0.5 in and 0.75 in. The scanner used on this project was the model VS6500, 6 ft. high (scanning length) with beams at the spacing of 0.5 inch, and designed for a beam range of 80 ft. Though it was capable of classifying vehicles, the scanner was programmed on this project just to estimate highest point on a vehicle, as the vehicle height.

The scanner was installed at three weigh stations on the Florida major corridors, on interstate highways 10, 75, and 95. The transmitter and receiver towers, were mounted onto the legal height detection system at a height of 10 feet above the pavement surface, with a beam range of approximately 30 ft. This position places the scanner almost directly above the weigh-in-motion plates at each station, enhancing a simultaneous collection of both the weight and truck data. The temporary reconstruction at each weigh station involved removing the top sections (single ray infrared scanner) of the existing vehicle over-height detectors, and the using the existing supporting frame to mount the new vehicle scanner towers. The towers were aligned to ensure correct transmission. New cables were installed to connect the new scanner to the existing data collectors in the weigh station.

Software programs were written to collect truck height and weight data and also integrate the data with the existing data collection system at each weigh station. The assembly is shown in Figures 2.7 and 2.8 for one of the weigh stations. This system enabled continuous uninterrupted monitoring of the traffic stream, detecting every vehicle of height between 10 and 16 feet. The main assumption here is that, based on a previous review of FDOT permit records, almost all trucks will lie within this scanner height range. The scanner system was integrated into the weigh station enforcement system so that the output data files recorded truck axle weights, vehicle classification and vehicle height for every vehicle passing through the weigh station. The output files were downloaded from the system periodically, and remotely through the telephone lines, using the pcAnywhere™ software.

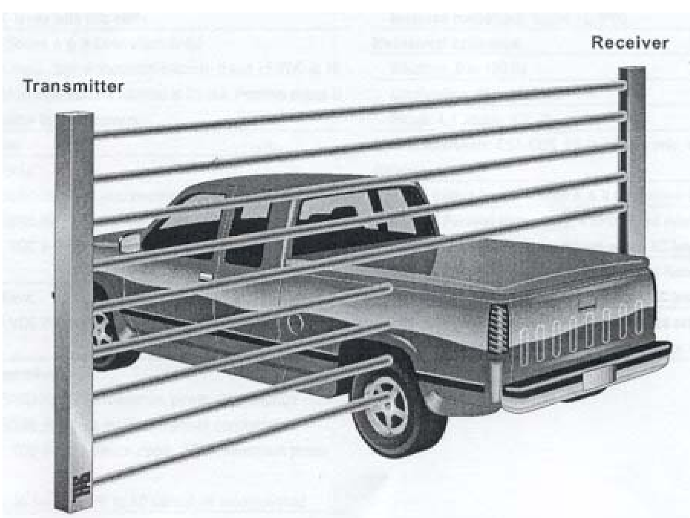


Figure 2.6: The Automatic Vehicle Scanner “Light Curtain” [STI 2002]

Figure 2.7 Typical assembly of a vehicle scanner tower, strapped to an existing height-detector pole.


Figure 2.8 Typical Installation of Vehicle Scanner at Weigh Stations

Table 2.2 lists the FTI stations (weigh-in-motion plates on the highway), and OMCC (weigh stations) Numbers at which height data was collected, the functional class of the roadway, and the description of the site. Due to low truck traffic and relaxed law enforcement at some weigh stations, sites were monitored with the laser range finder for a few days each, recording truck heights for a minimum of 8 hours a day, typically within the time frame 7:00am to 7:00pm. For the sites monitored by the vehicle scanner, the scanner collected data continuously without interruption, except for technical or maintenance reasons. The major maintenance problem faced was related to temperature and humidity effect on the scanner receiver glass, due to the fog formation during cold or rainy weather. During these times, the scanners had a tendency to indicate all vehicles as violating the legal height, thereby routing them for static measurements at the weigh station. The study was temporarily stopped and the problem corrected by the vendor installing better transformers to generate enough warmth to keep the glass of the receiver clear at all times. At the final site (I-95), the vendor was ordered to replace the unit with a new receiver capable of capturing data under all weather conditions. The collected data at the three sites were also screened to ensure no interruption or lapse in stream of height data collected.

The locations of the truck weight data collection points used to formulate the models for each functional class of roadway are summarized in Table 2.3 while the geographical locations of the various test sites are shown on Florida map in Figure 2.9.

Table 2.2: Truck Height Data Collection Sites

| Site ID | Functional class of roadway | Site type | Measuring device | Site Description | Collection Dates |
|---|---|---|---|---|---|
| OMCC #19 | 01 | Weigh station* | Laser range finder | I-75 Punta Gorda, FL | Aug. 12, 2002 to Aug. 16, 2002 |
| OMCC #11 | 02 | Weigh (Static) station | Laser range finder | US-19 Old Town, FL | June 19, 2002 to June 21, 2002 |
| OMCC #7 | 06 | Weigh (Static) station | Laser range finder | SR-121, MacClenny, FL | July 10, 2002 to July 11, 2002 |
| FTI 559908 | 14 | Weigh-in-motion | Laser range finder | US-319, Tallahassee, FL | June 27, 2002 to June 28, 2002 |
| OMCC # 6 | 14 | Weigh (Static) station | Laser range finder | US-441, Lake City, FL | June 17, 2002 |
| FTI 872515 | 19 | Traffic Monitoring Station | Laser range finder | SR-823, Miami-Dade, FL | Oct. 7, 2002 to Oct. 9, 2002 |
| OMCC #3 | 01 | Weigh station* | Vehicle Infrared Scanner | I-10 East, Sneads, FL | Nov. 14, 2002 to Dec. 14, 2002 |
| OMCC #5 | 01 | Weigh station* | Vehicle Infrared Scanner | I-75 South, White Springs, FL | Jan. 22, 2003 to Feb. 11, 2003 |
| OMCC #14 | 01 | Weigh station* | Vehicle Infrared Scanner | I-95 South, Flagler, FL | April 1, 2003 to May 22, 2003 |

* Site has both weigh-in-station plates and static weighing platforms.

Table 2.3: Truck Weight Data Collection Stations

| FTI Site ID | Functional Class of roadway | Site type | Data Type | Site Description | Collection Dates |
|---|---|---|---|---|---|
| 939935 | 02 | Weigh-in-motion | Truck Speed, Road Class, Axle Weight/ Spacing | US-27, Palm Beach County, FL | Feb. 2000 to Dec. 2000* |
| 899921 | 02 | Weigh-in-motion | Truck Speed, Road Class, Axle Weight/ Spacing | US-1, Jupiter, FL | June 2000 to Dec. 2000* |
| 299936 | 01 | Weigh-in-motion | Truck Speed, Road Class, Axle Weight/ Spacing | I-10, Lake City, FL | Dec. 2000* |
| 559908 | 14 | Weigh-in-motion | Truck Speed, Road Class, Axle Weight/ Spacing | US-319, Tallahassee, FL | June 2000 to Dec. 2000* |
| 549901 | 01 | Weigh-in-motion | Truck Speed, Road Class, Axle Weight/ Spacing | I-10, Monticello, FL | June 1999* |
| 509940 | 07 | Weigh-in-motion | Truck Speed, Road Class, Axle Weight/ Spacing | SR-267, Quincy, FL | Aug. 2002* |
| 599946 | 06 | Weigh-in-motion | Truck Speed, Road Class, Axle Weight/ Spacing | US-98, St. Marks, FL | Aug. 2002* |
| 489924 | 11 | Weigh-in-motion | Truck Speed, Road Class, Axle Weight/ Spacing | I-110, Pensacola, FL | Aug. 2002* |
| OMCC #3 | 01 | Weigh station | Truck Height, Speed, Road Class, Axle Weight | I-10 East, Sneads, FL | Nov. 14, 2002 to Dec. 14, 2002 |
| OMCC #5 | 01 | Weigh station | Truck Height, Speed, Road Class, Axle Weight/ Spacing | I-75 South, White Springs, FL | Jan. 22, 2003 to Feb. 11, 2003 |
| OMCC #14 | 01 | Weigh station | Truck Height, Speed, Road Class, Axle Weight/ Spacing | I-95 South, Flagler, FL | April 1, 2003 to May 22, 2003 |

* Collected by FDOT Traffic Statistics Office

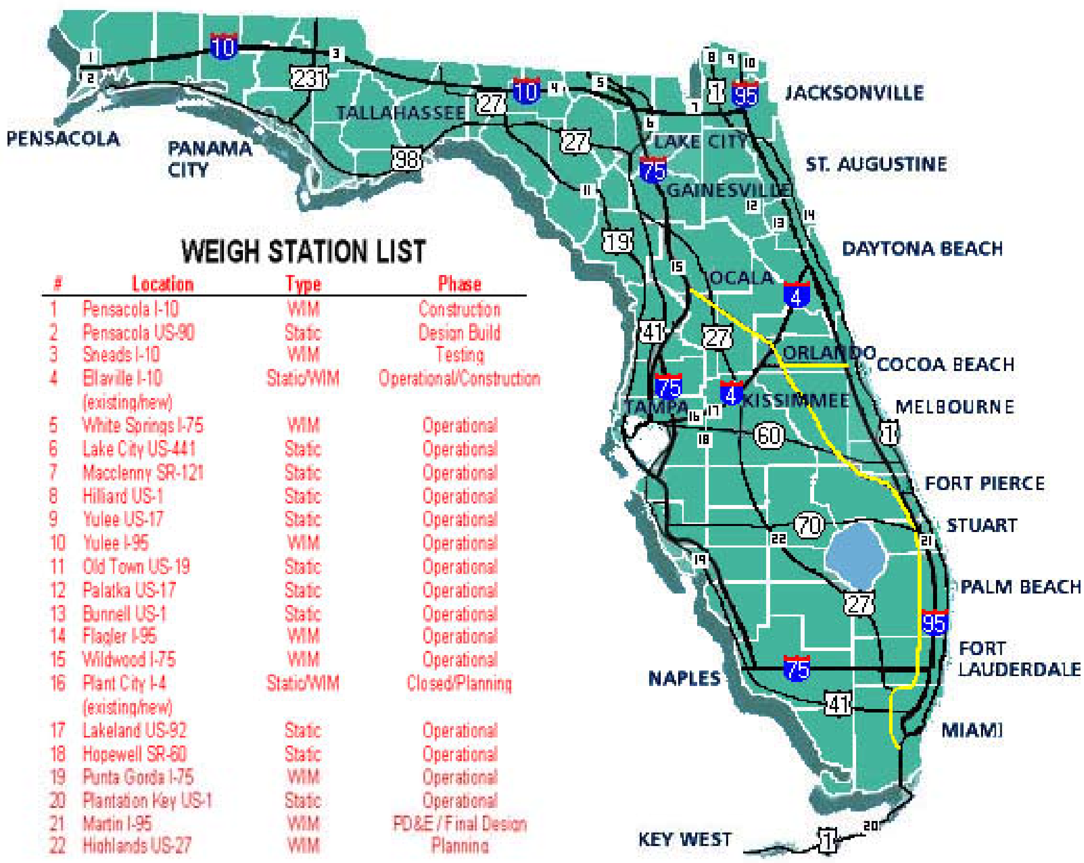



Figure 2.9 Map showing the locations of the FDOT Weigh Stations (Source: OMCC website)

## 2.3. National Bridge Inventory (NBI) Data

The National Bridge Inventory (NBI) is a database maintained by the FHWA, on over 600,000 bridges on public roads throughout the United States, with the Florida inventory comprising over 14,000 records. For each recorded bridge the NBI has 122 unique coded fields that describe the bridge in terms of age, location, structural characteristics, traffic characteristics, operating characteristics, and maintenance and inspection history (FHWA 1995). Structural characteristics include type and materials of construction, load-rating capacities, and number of spans. Traffic characteristics include ADT, percentage truck traffic, and future AADT projections. Operating characteristics include number of lanes on the bridge, type of service under the bridge, vertical under clearance, and horizontal clearance on the bridge. Other information coded consists of unique identification numbers for each bridge.

The research team acquired the 2002 NBI for Florida as a text file from FDOT with the focus at this stage being to learn what information could be derived from the records. Item 5 of the NBI requires the specification of an inventory route for every bridge. This is the route for which the applicable inventory data are to be recorded. The inventory route may be on the structure or under the structure. The inventory route being on the bridge or under the bridge is specified in item 5A – Type of record. Item 5A is coded 1 for the on route and 2 for the under route. If there is more than one under route these are labeled starting from 2 then by alphabetic letters starting from A. Separate data sets were created for pertinent NBI bridge data related to primary on-routes, and under roadways, ignoring non-highway bridges

For the under routes further sorting was done by item21 – maintenance responsibility, and only bridges maintained by the state highway agency or the state toll authority were considered. The inventory was then sorted by item 42B – type of service under the bridge, and only those with highway service were considered. This yielded an under-route inventory of 2,397 bridges maintained or owned by the state highway agency or the state toll authority. Figure 2.10 shows functional class 19 urban local streets have the highest incidence of under passing a highway bridge, for the state maintained bridge network for the year 2002.
Under clearance deficiency problems are of particular concern for bridges below 14 feet (Thompson et al. 1999). The graph in Figure 2.11 shows that only 1.40% of Florida bridges have less than 14 feet under clearance.

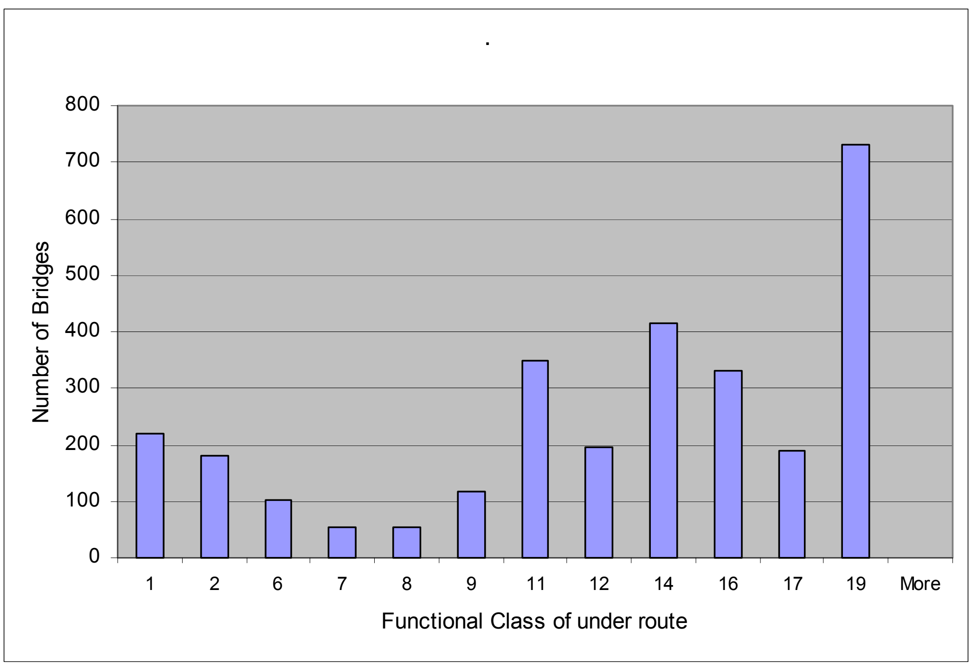



Figure 2.10: Under Routes by Functional Classification in Florida State Highway System, NBI2002

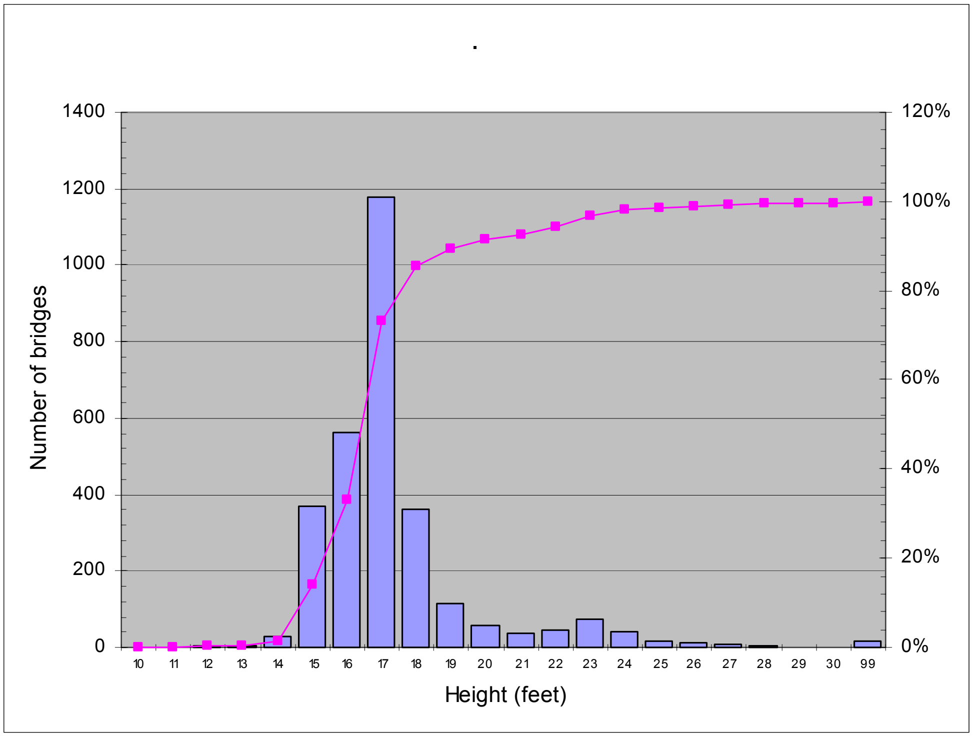



Figure 2.11: Under Clearance Distribution For Florida State Highway System Bridges, NBI 2002

The initial over route inventory was sorted by item 21 - maintenance responsibility in order to remove bridges not maintained by the state highway agency or the state toll authority. The inventory was then sorted by item 42A – type of service on the bridge. Codes representing highway (service) on the bridge were maintained and the rest, codes 2,3,9,0 were removed. NBI Items 41 - bridge posting, and 103 - temporary structure designation, were investigated for the over route inventory, indicating the presence of closed bridges and temporary bridges.
The above procedure yielded an over route inventory of 5,551 bridges maintained by the state highway agency or the state toll authority. Figure 2.12 shows the distribution of over routes by functional classification of the roadway. As shown in Figure 2.13, the distribution of operating rating shows that just over 2.53% of the network bridges has an operating rating lower than the legal truck weight limit.

The distribution of operating rating by functional class (Figure 2.14), however does not indicate a marked deficiency in operating rating generally in Florida, however it does show that local streets and minor collectors in rural and urban areas tend to have lower operating ratings. The Average Annual Daily Truck Traffic (ADTT) is highest on rural and urban interstate routes (Figure 2.15). The plot of ADTT versus operating rating shows there is no consistent trend that weaker bridges (lower operating rating) are subjected to the higher truck volumes (Figure 2.16).

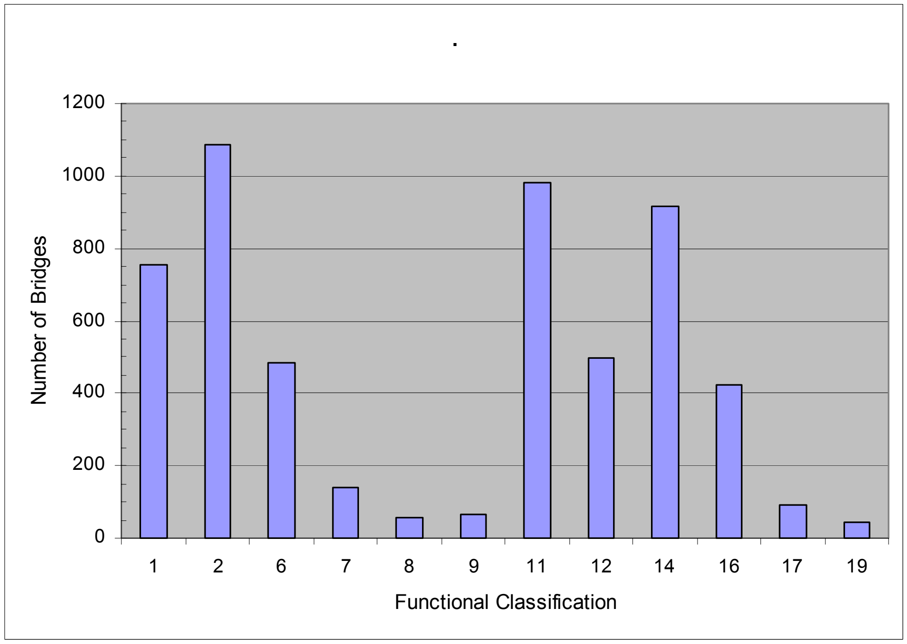



Figure 2.12: Over Routes on Bridges By Functional Class in Florida State Highway System, NBI 2002

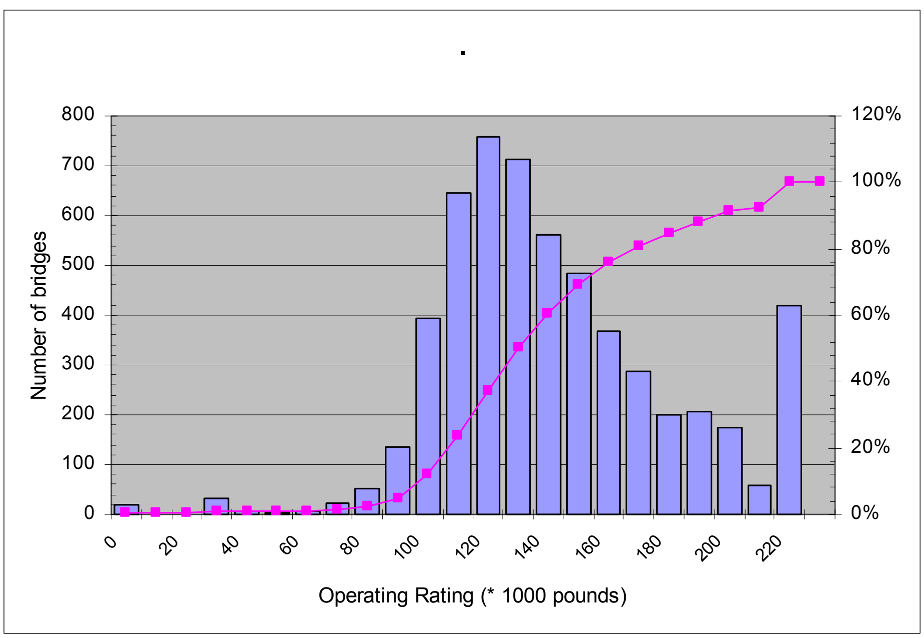



Figure 2.13: Operating Rating Distribution For Florida State Highway System Bridges, NBI 2002

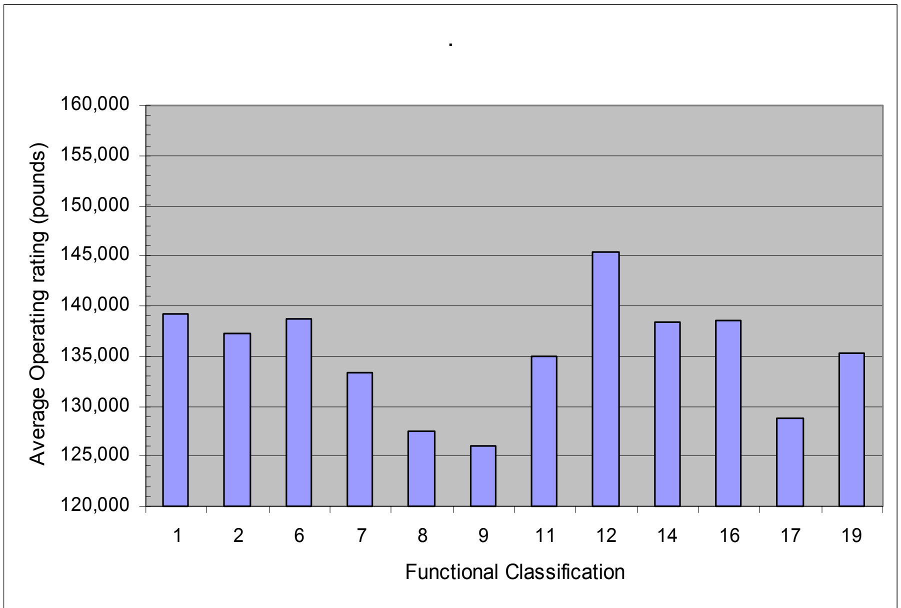



Figure 2.14: Operating Rating By Functional Class for Florida State Highway System Bridges, NBI 2002 (Over Routes)

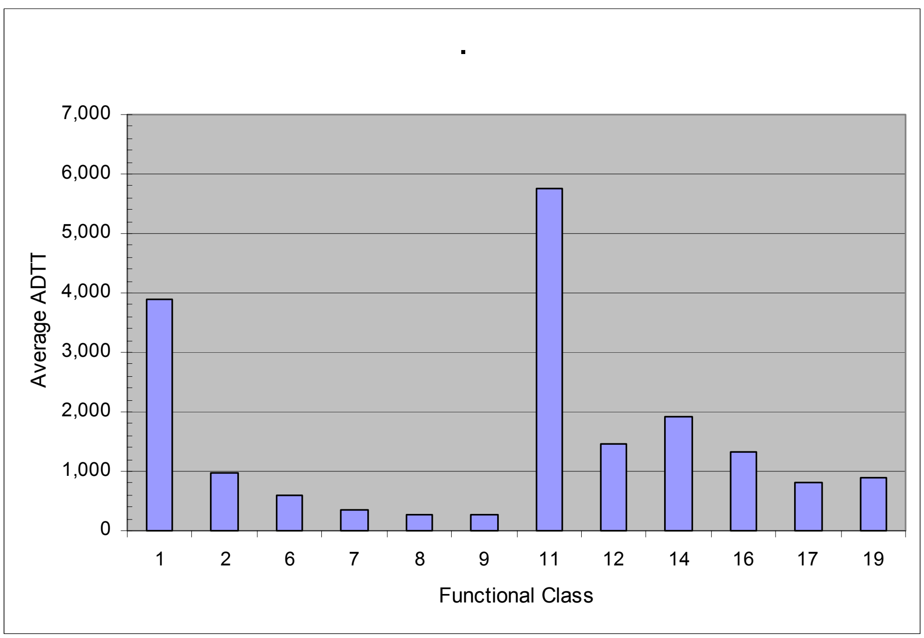



Figure 2.15: Florida State Highway System Truck Traffic By Functional Class, NBI 2002

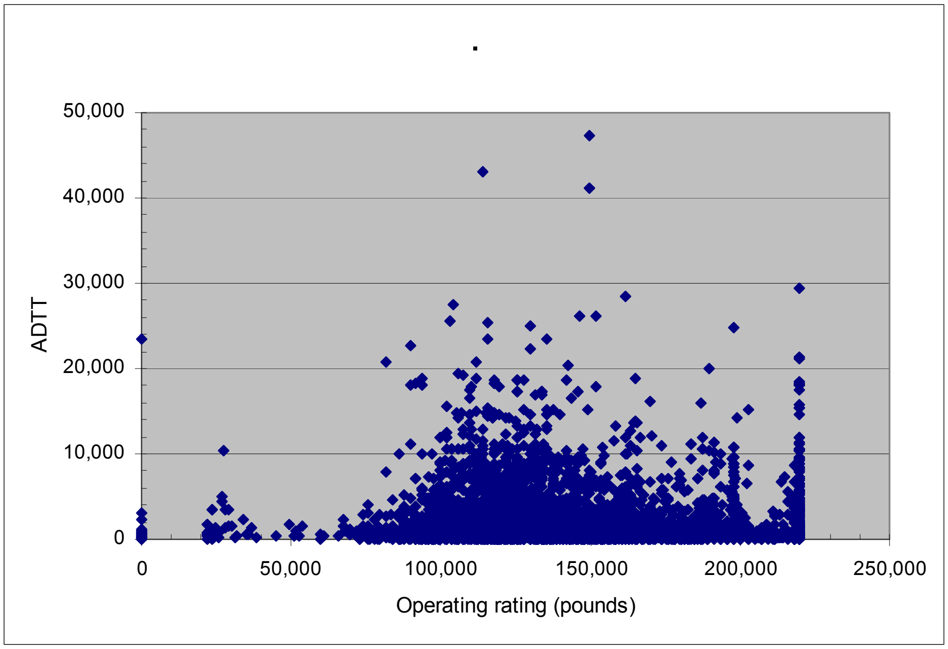



Figure 2.16: ADTT versus Operating Rating for Florida State Highway System Bridges, NBI 2002

## 2.4. Truck Height Distributions

As mentioned earlier, truck height data were compiled from the measurements taken by the STI's VS6500 vehicle scanner and the laser range finder. The measurements for each site were compiled and combined for the period of monitoring. Histograms and cumulative frequency charts were drawn to represent the data. The cumulative frequency chart represents the proportion of trucks below a given height. This study seeks to determine the number of trucks taller (greater) than a given height. Such a reverse cumulative frequency distribution was drawn for each site. The reverse cumulative frequency chart gives the proportion (probability) of vehicles greater than a given (bridge) height and must therefore detour. In this study the state highway system bridge inventory was analyzed by roadway functional class. Height distributions obtained at various locations on the Florida network, were chosen to represent the functional classes and all bridges in a functional class were assigned this truck height distribution. This is an improvement on the existing Pontis model that assigns one truck height distribution developed in California to the entire inventory.

To incorporate the truck height distribution into the user cost model its functional form must be determined and applied as a part of the user cost computation. The current user cost model in Pontis uses a stepwise linear function. Step functions were fitted to the truck height distributions by regression. In their study on Pontis user costs for Florida, Thompson et al (1999) recommended that the functional form of the distributions be changed if adequate data could be collected. The changes proposed by Thompson et al (1999) were that the existing function be given more detail or to change the type of function used. In this study both two options were investigated. The first was with regards to improving the detail by increasing the number of break points of the step function. The second option was to change the functional form, from a step function to a piecewise linear function or a curvilinear function. The latter would comprise piece wise linear or curve functions best fitting the data collected.

Truck height distribution functions were therefore formulated, including histograms, fitted Pontis step functions, and fitted piece wise functions. The Pontis step function values were estimated as averages between pertinent start and end values on the original data curve, and also with both the line segment and original curve having approximately same area under the curve. Using the data points on the reverse cumulative frequency curves, linear and nonlinear regression analyses were conducted to develop the best fitting piece wise functions for the truck height data. As shown in Figures 2.17 to 2.23, the truck height histograms and models were summarized in three forms: using the data collected on all roadways; using only the data for Interstate roadways; and the data for only Non-Interstate roadways. Tables 2.4 to 2.7 shows the recommended values of the step functions, and the equations derived for each segment of the piece wise functions, with the respective coefficient of determination ($R^2$). Appendixes C and E show the detailed results for all the data sites, including the histograms and the various fitted functions.

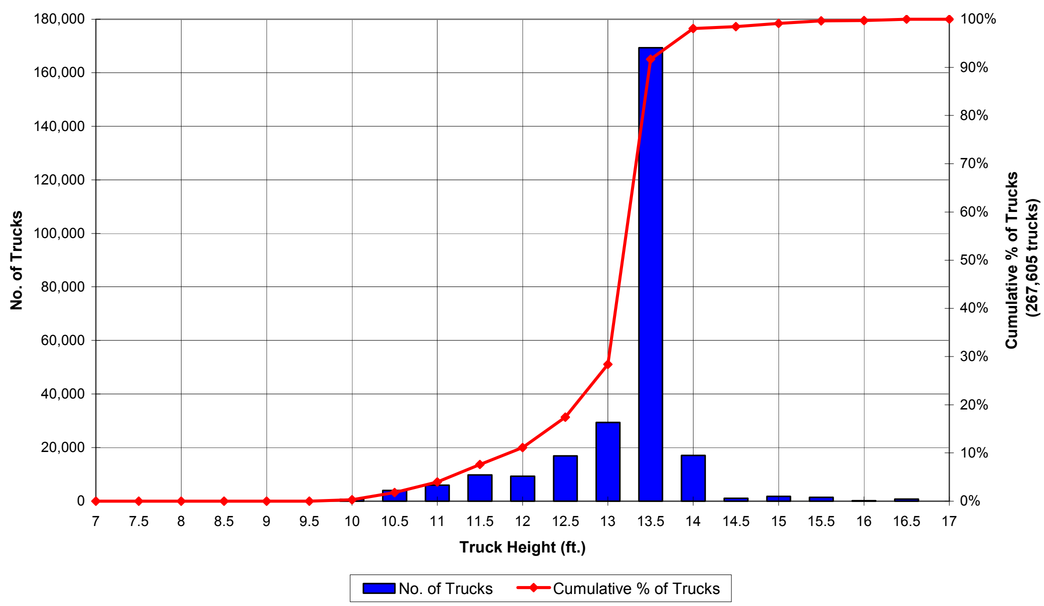



Figure 2.17 Truck Height Histogram for Florida Interstate Roadways

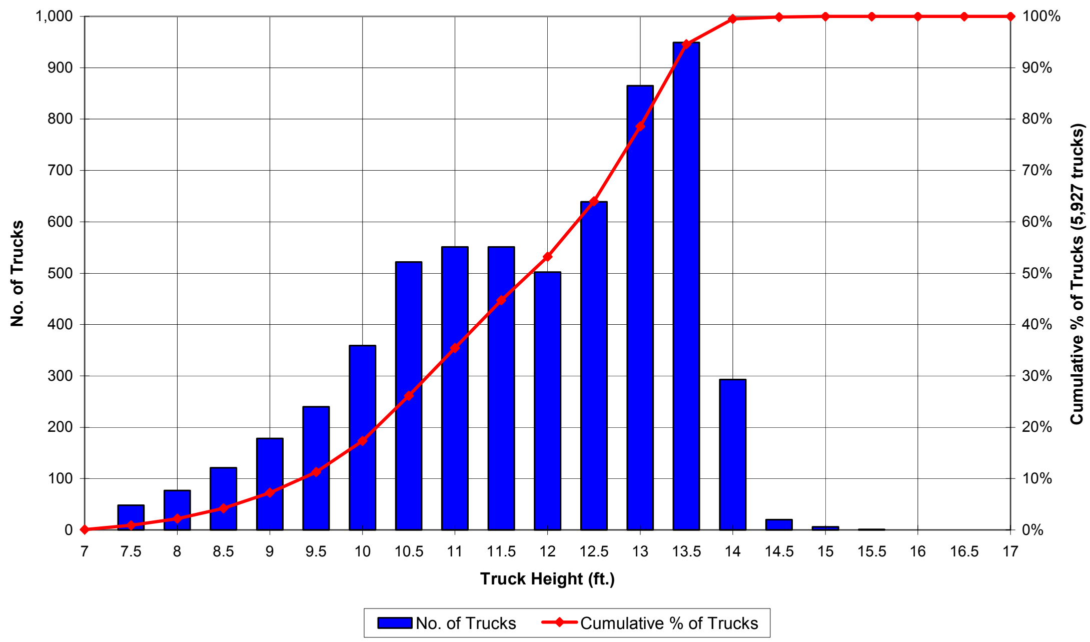



Figure 2.18. Truck Height Histogram for Florida Non-Interstate Roadways

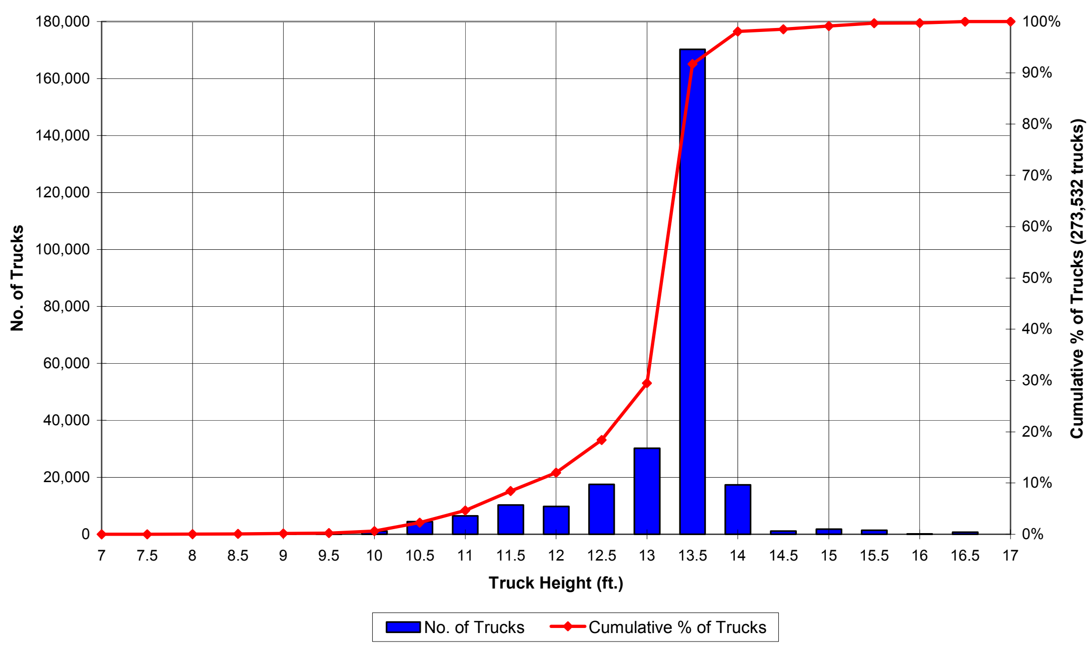



Figure 2.19 Truck Height Histogram for All Florida Roadways

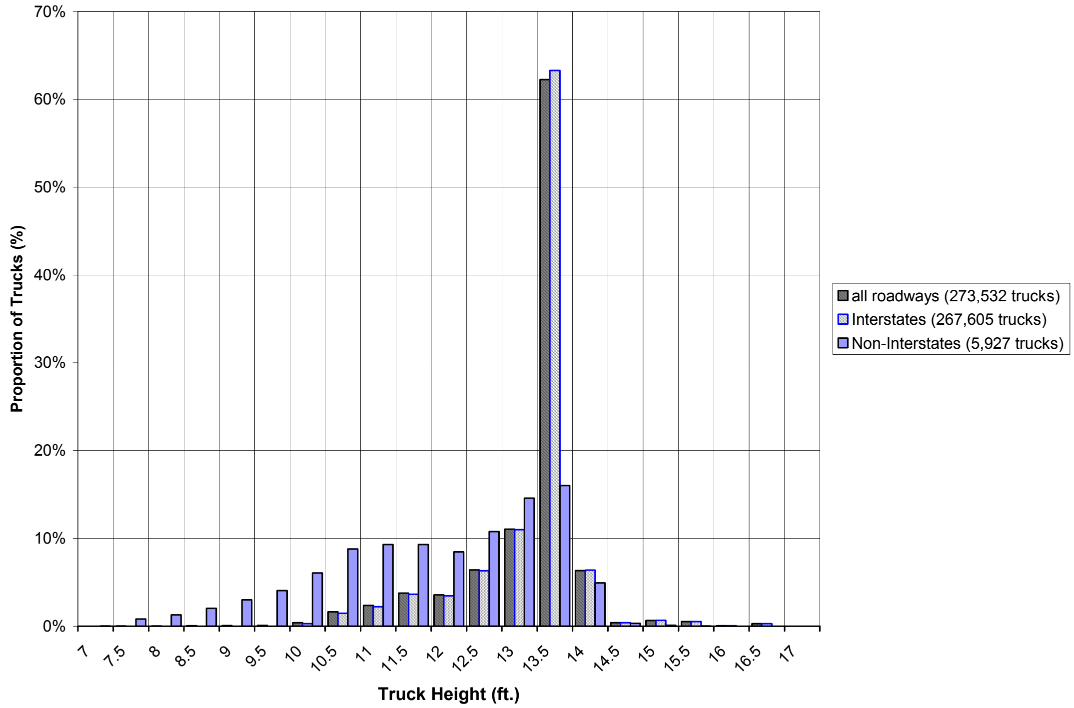



Figure 2.20 Comparison of Truck Height Histograms for Florida Highways

Table 2.4 Truck Height Step Functions (Pontis) for All/Interstate Florida Roadways

| Point | Height limit (ft.) | Percent Detoured |
|---|---|---|
| A | <= 0.0 | 0.000 |
| B | <=10.0 | 100.000 |
| C | < 12.0 | 93.700 |
| D | < 13.0 | 79.250 |
| E | < 14.0 | 36.200 |
| F | < 16.0 | 0.245 |
| | > 16.0 | 0.000 |

Table 2.5 Truck Height Step Functions (Pontis) for Non-Interstate Florida Roadways

| Point | Height limit (ft.) | Percent Detoured |
|---|---|---|
| A | <= 0.0 | 0.000 |
| B | <=7.0 | 100.000 |
| C | < 10.0 | 91.350 |
| D | < 12.0 | 64.750 |
| E | < 13.5 | 26.100 |
| F | < 14.5 | 2.750 |
| | > 14.5 | 0.000 |

Table 2.6 Truck Height Piecewise Curves for Florida Interstate Roadways

| Height Range (ft.) | Percent Detoured | Regression $R^2$ |
|---|---|---|
| < 9.65 | 100.00 | 1.000 |
| 9.65 -- 13.00 | $855.91 - 223.430\mathbf{x} + 22.199\mathbf{x}^2 - 0.742\mathbf{x}^3$ | 0.997 |
| 13.00 -- 14.00 | $(1.10E+56)\ \mathbf{x}^{-48.683}$ | 0.998 |
| 14.00 -- 16.10 | $14.567 - 0.905\mathbf{x}$ | 0.998 |
| > 16.10 | 0.00 | 1.000 |

Table 2.7 Truck Height Piecewise Curves for Florida Non-Interstate Roadways

| Height Range (ft.) | Percent Detoured | Regression $R^2$ |
|---|---|---|
| < 7.30 | 100.00 | 1.000 |
| 7.30 -- 13.50 | $-26.275 + 34.692\mathbf{x} - 2.389\mathbf{x}^2$ | 0.999 |
| 13.50 -- 14.00 | $138.860 - 9.886\mathbf{x}$ | 1.000 |
| > 14.00 | 0.000 | 1.000 |

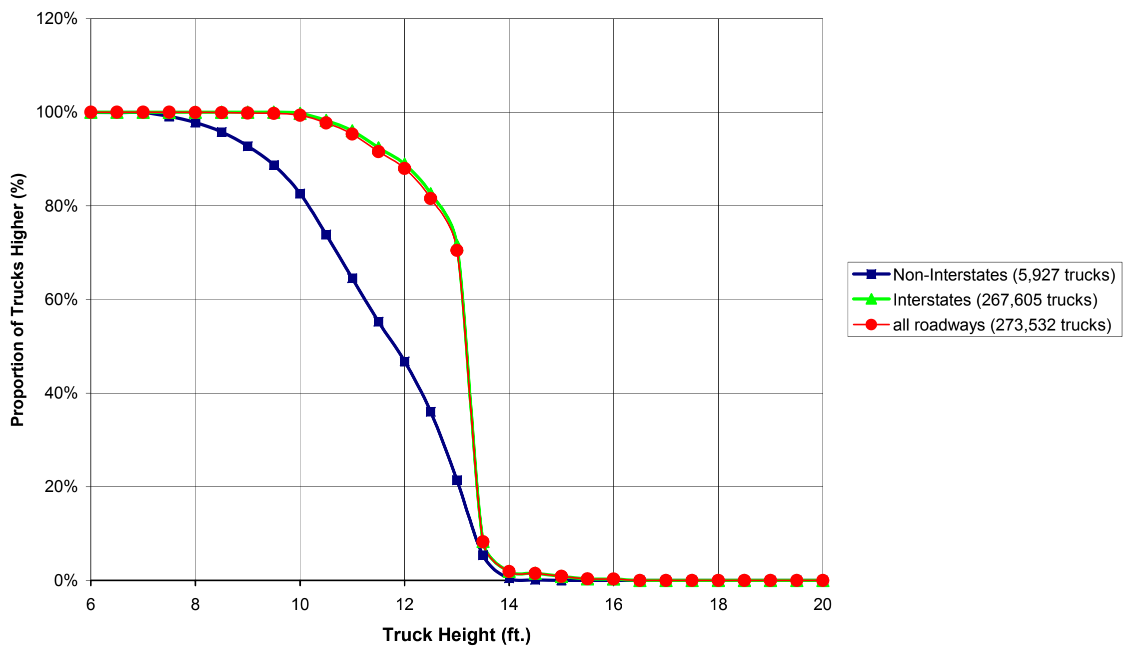



Figure 2.21 Truck Height Reverse Cumulative Curves for Florida Highway Types

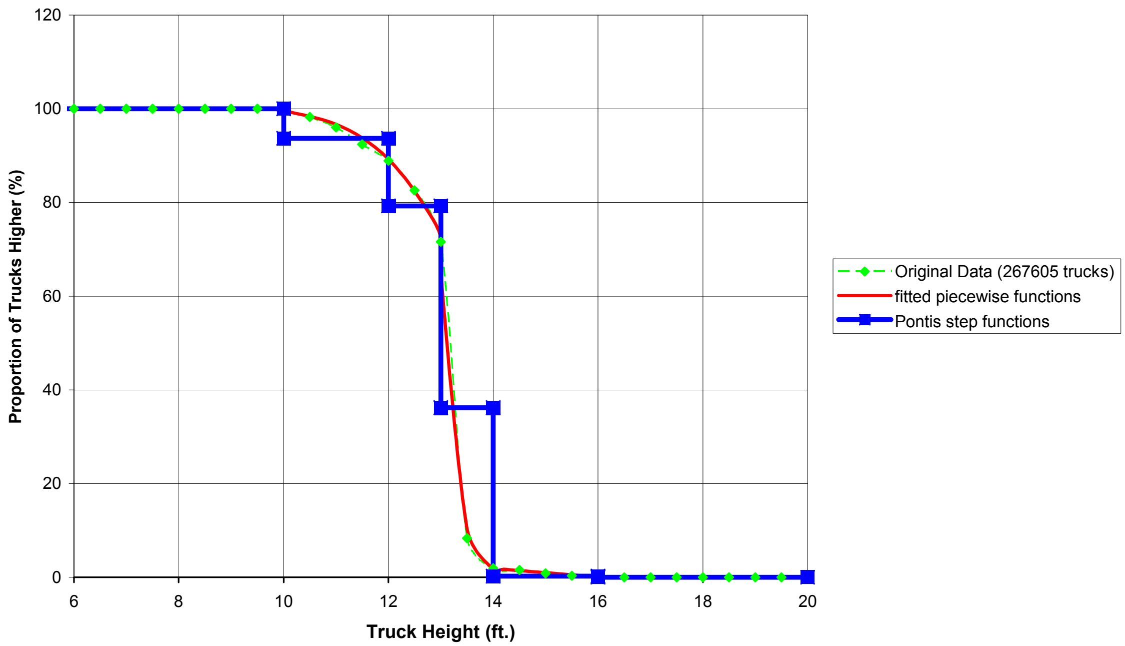



Figure 2.22 Truck Height Models for Florida Interstate Highways

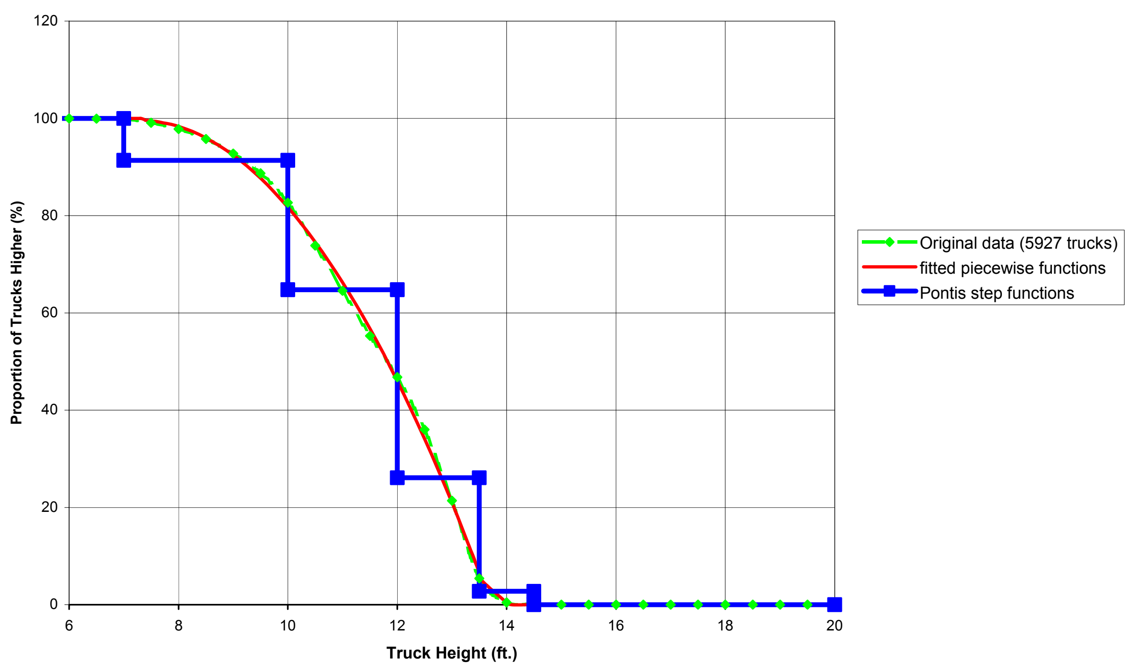



Figure 2.23 Truck Height Models for Florida Non-Interstate Highways

## 2.5. Truck Weight Distributions

Truck weight distributions are needed in the input for the benefit of the strengthening model. By comparing the truck weight distribution on the roadway or on a bridge, with the operating rating of the bridge, NBI Item 64, the number of vehicles that will need to detour can be estimated. . In this study the entire inventory was analyzed by roadway functional class. Weight distributions obtained at the various locations on the Florida network, were chosen to represent the functional classes and all bridges in a functional class were assigned this truck weight distribution. This an improvement on the existing Pontis model that assigns one truck weight distribution developed in California to the entire Florida inventory.

In this study existing truck weight data were compiled by FDOT at weigh-in-motion stations and highway enforcement weight stations on the state highway network. Two sources of weight data were used: (1) the weight data collected at the weight stations' WIM plates, simultaneously with the STI's vehicle scanner's height data (Sneads, Flagler, and White Springs); and (2) a set of previously recorded truck weight (WIM) data, labeled VTR files, for various locations, obtained from the FDOT's Office of Transportation Statistics. The data dictionary of the files obtained for both sources are included in Appendix B, indicating the fields such site ID, time, location, vehicle class, speed, length, and axle loads and spacing. For this study the value of interest was the gross vehicle weight (GVW). This study seeks to determine the number of trucks that exceed a given weight. So a reverse cumulative frequency distribution was drawn for each site; the reverse cumulative frequency curve indicates the proportion of trucks expected to be heavier than a given (bridge) allowable truck weight (Item 64 – operating rating) that must therefore detour.

Truck weight distribution functions were formulated, including histograms, cumulative frequency curves, reverse cumulative frequency curves, and fitted piece wise linear/nonlinear functions. Using the data points on the reverse cumulative frequency curves, linear and nonlinear regression analyses were conducted to develop the best fitting piece wise functions for the truck weight data. The Pontis piece wise linear functions were obtained, indicating the pertinent points required. As shown in Figures 2.24 to 2.31, the truck weight histograms and models were again summarized in three forms: all roadways; interstate roadways; and non-interstate roadways. Tables 2.8 to 2.13 shows the recommended points, and the equations derived for each segment of the piece wise functions, with the respective coefficient of determination ($R^2$). Appendixes D and E show the detailed results for all the data sites, including the histograms and the various fitted functions.

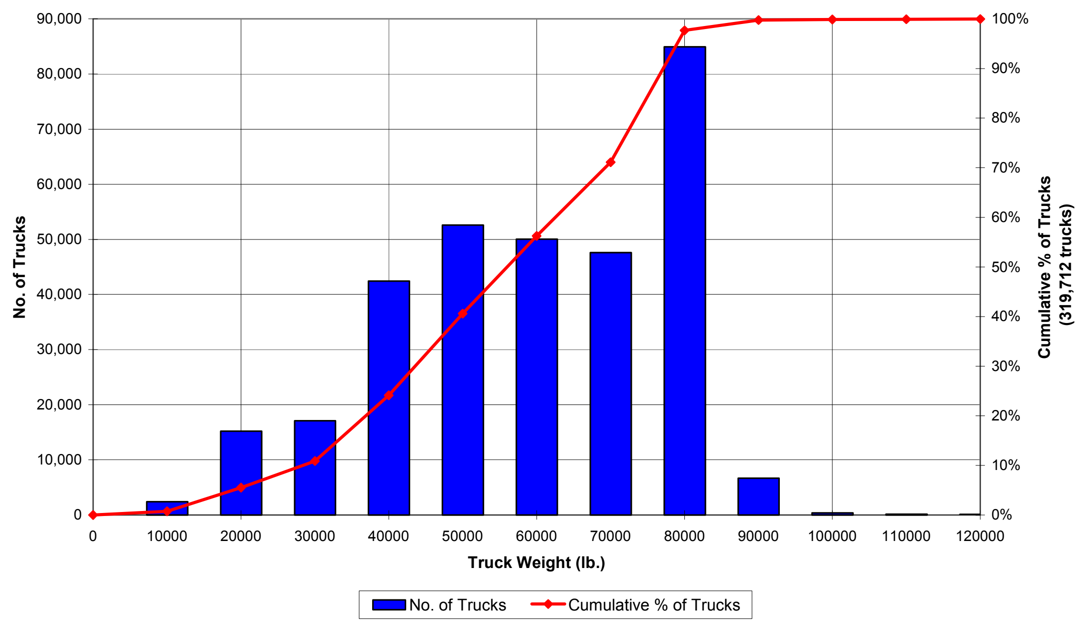



Figure 2.24. Truck Weight Histogram for Florida Interstate Roadways


Figure 2.25. Truck Weight Histogram for Florida Non-Interstate Roadways

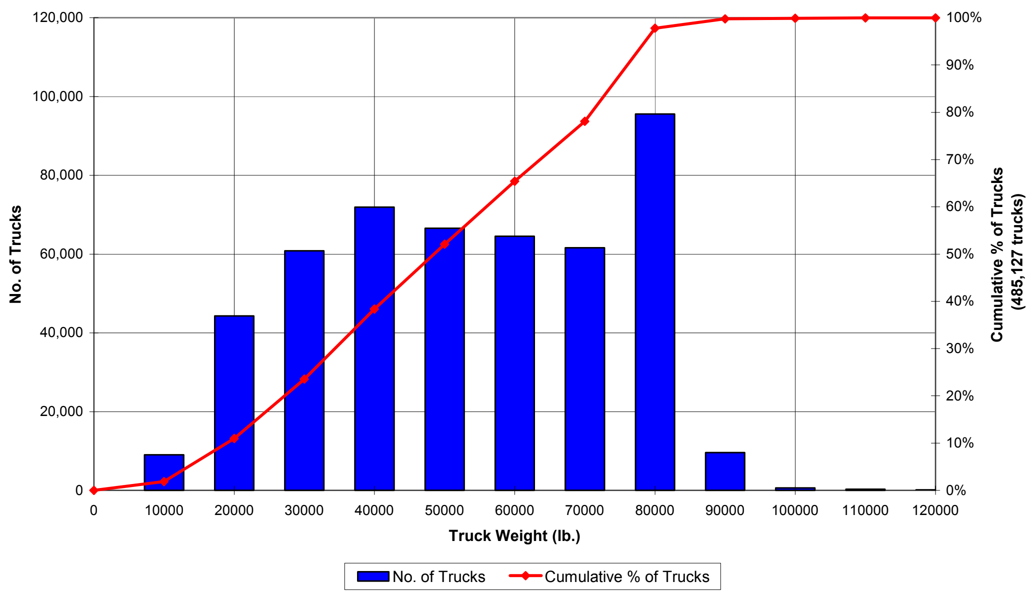



Figure 2.26. Truck Weight Histogram for All Florida Roadways

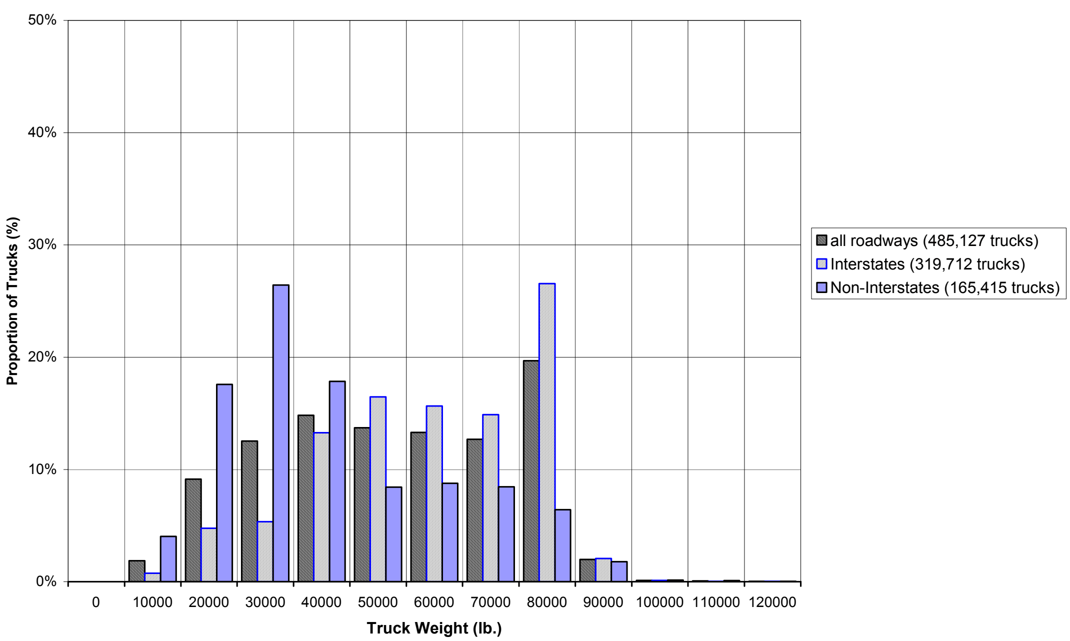



Figure 2.27. Comparison of Truck Weight Histograms for Florida Highways

Table 2.8. Truck Weight Piecewise Linear Data (Pontis) for All Roadways

| Point | Weight Limit (lbs) | Percent Detoured | Linear Regression $R^2$ |
|---|---|---|---|
| A | 10507.69 | 100.00 | 0.997 |
| B | 70000.00 | 21.90 | 0.816 |
| C | 89121.82 | 0.00 | |

Table 2.9. Truck Weight Piecewise Linear Data (Pontis) for Interstate Roadways

| Point | Weight Limit (lbs) | Percent Detoured | Linear Regression $R^2$ |
|---|---|---|---|
| A | 15072.73 | 100.00 | 0.948 |
| B | 60000.00 | 50.58 | 0.919 |
| C | 85394.40 | 0.00 | |

Table 2.10. Truck Weight Piecewise Linear Data (Pontis) for Non-Interstate Roadways

| Point | Weight Limit (lbs) | Percent Detoured | Linear Regression $R^2$ |
|---|---|---|---|
| A | 3552.94 | 100.00 | 0.967 |
| B | 50000.00 | 21.04 | 0.965 |
| C | 82056.67 | 0.00 | |

Table 2.11. Truck Weight Piecewise Curves for Interstate Roadways

| Weight Range (lb.) | Percent Detoured | Regression $R^2$ |
|---|---|---|
| < 10,000 | 100.00 | 1.000 |
| 10,000 -- 80,000 | $102.24 - (8.98E\text{-}05)\mathbf{x} - (1.43E\text{-}08)\mathbf{x}^2$ | 0.997 |
| 80,000 -- 91,100 | $18.98 - (2.08E\text{-}04)\mathbf{x}$ | 1.000 |
| > 91,000 | 0.00 | |

Table 2.12. Truck Weight Piecewise Curves for Non-Interstate Roadways

| Weight Range (lb.) | Percent Detoured | Regression $R^2$ |
|---|---|---|
| < 3,700 | 100.00 | 1.000 |
| 3,700 -- 85,000 | $107.26 - (1.97E\text{-}03)\mathbf{x} + (6.53E\text{-}09)\mathbf{x}^2 + (2.23E\text{-}14)\mathbf{x}^3$ | 0.986 |
| > 91,000 | 0.00 | |

Table 2.13. Truck Weight Piecewise Curves for All Roadways

| Weight Range (lb.) | Percent Detoured | Regression $R^2$ |
|---|---|---|
| < 3,600 | 100.00 | 1.000 |
| 3,600 -- 85,000 | $103.17 - (7.13E\text{-}04)\mathbf{x} - (6.86E\text{-}09)\mathbf{x}^2$ | 0.996 |
| 80,000 -- 91,200 | $18.09 - (1.98E\text{-}04)\mathbf{x}$ | 1.000 |
| > 91,200 | 0.00 | 1.000 |

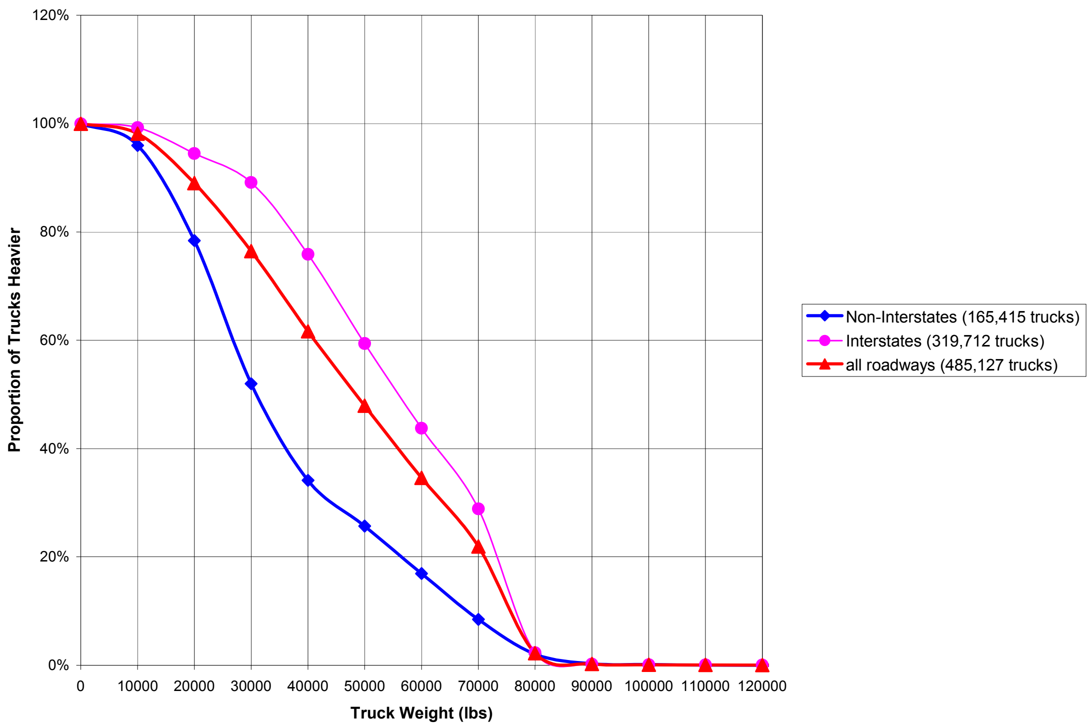



Figure 2.28. Truck Weight Reverse Cumulative Curves for Florida Highway Types

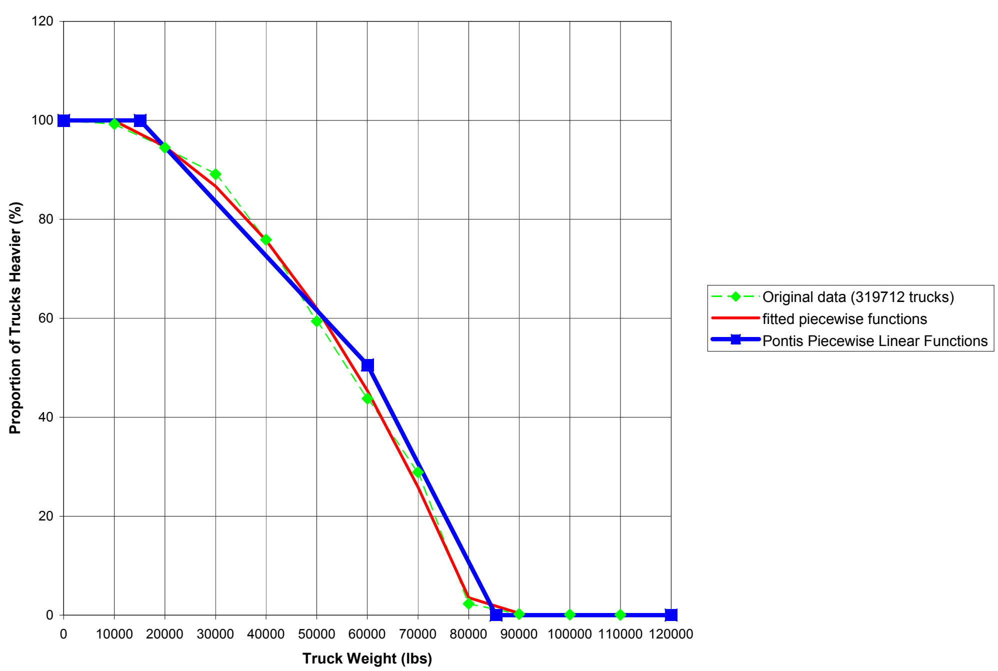



Figure 2.29 Truck Weight Models for Florida Interstate Highways

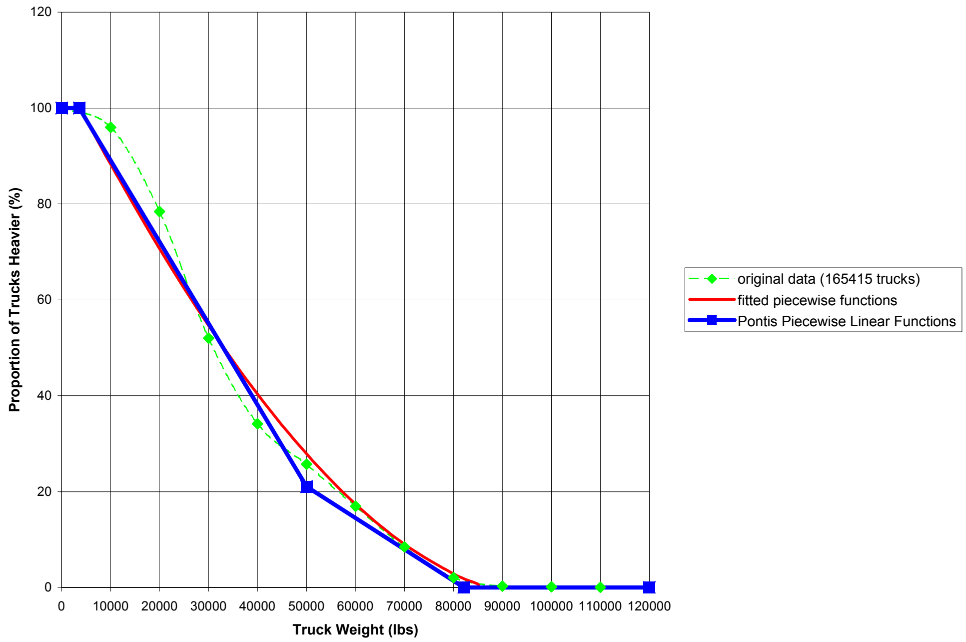



Figure 2.30 Truck Weight Models for Florida Non-Interstate Highways

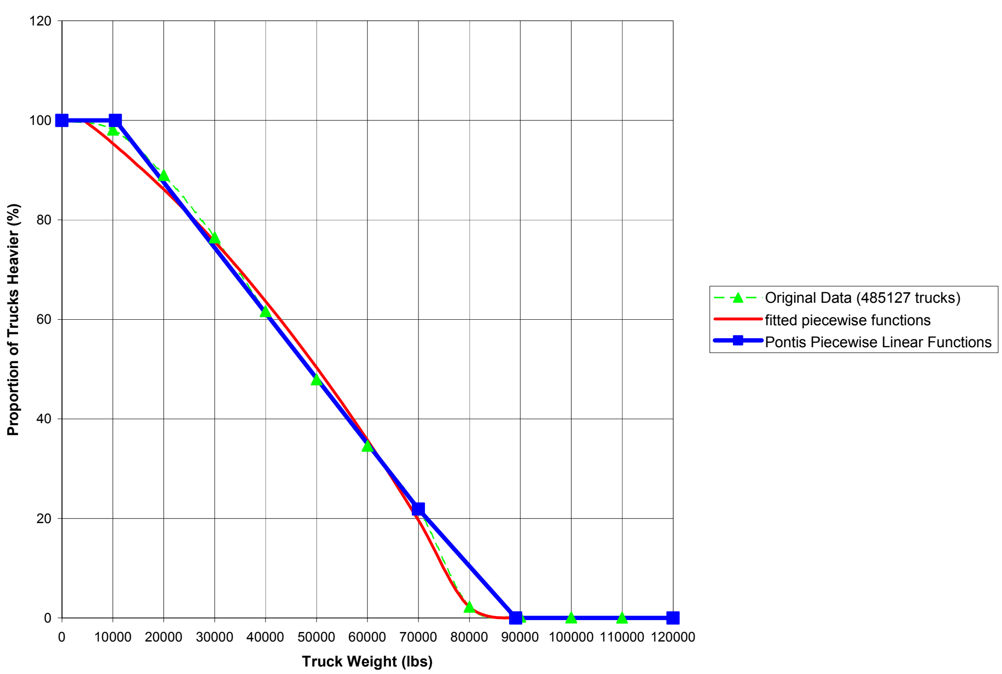



Figure 2.31 Truck Weight Models for All Florida Highways

## 2.6. Cost Data

User cost rates, also called unit cost parameters, are the dollar values assigned to each user cost component. User costs are calculated by multiplying the quantity of the various additional user cost components (VOC, delay, and accidents) incurred by the unit cost for those components. From equation 3.4, the cost parameter inputs are the vehicle operating cost per mile and the travel time cost also commonly referred to as the value of time. The current Pontis model does not have any Florida specific data (Thompson et al, 1999). Default values are currently used. The literature search however uncovered good sources of this information. The most notable among these being; Florida Trucking Association (FTA), Highway Economic Requirements System (HERS), AASHTO Red Book, North Carolina BMS, Indiana BMS and MicroBENCOST program by the Texas Transportation Institute (TTI). A more in-depth discussion of the above data sources can be found elsewhere.

Thompson et al, (1999), recommend FTA vehicle operating costs and the HERS value of time, for user cost estimation on the Florida network. Thompson et al, (1999) used these parameters in 1993 dollars and updated to them to 1999 dollars by applying the consumer price index (CPI). Table 2.14 shows the recommended unit costs for Florida by Thompson et al, (1999) in 1999 dollars.

Table 2.14: Unit Cost Parameters for 1999 [Thompson et al. 1999]

| **Vehicle Operating Cost $/vehicle mile** | **Travel Time Cost (Value of Time) $/vehicle hour** |
|---|---|
| 0.44 | 26.43 |

The FHWA Life-Cycle Analysis in Pavement Design-Interim Technical Bulletin (September, 1998) requires that previous year vehicular operating costs be updated to the current analysis year by the transportation component of the consumer price index (CPI). The technical bulletin also requires the value of time be updated to the current analysis year by the “all components” of the CPI. In this study, the Thompson et al (1999) recommended values were adopted and updated to the analysis year as per the FWHA requirements. The year 2002 unit costs are obtained are given in Table 2.15.

Table 2.15: Unit Cost Parameters for 2002

| **Vehicle Operating Cost $/vehicle mile** | **Travel Time Cost (Value of Time) $/vehicle hour** |
|---|---|
| $0.49 | $29.03 |

## 2.7. Speed Data

Speed data is a necessary input in the calculation of travel time delay costs resulting from detour. The detour speed for every bridge is required for estimation of the benefits of raising and strengthening. Florida does not have detour speed data for any of its bridges (Thompson et al, 1999). The old Pontis model for travel time on a detour route uses a default speed of 30 mph for the entire inventory. Though this value may be applicable to local streets and collectors (functional classes 8, 9, 17, 19) it is an over simplification. It was learned towards the end of this study that detour speed is now being collected in Pontis for every bridge in the Florida inventory, but it was late to incorporate the data into the study. Both roadway speed and bypass speed are currently being populated in Pontis databases from inspectors' observations. However, the following three methods were investigated during the study.

Method 1: Data on speeds for individual vehicles (trucks): This data is available from FDOT's historical weigh-in-motion data for trucks. A representative sample of data was compiled, for 1 month, 3 months, etc., and the average truck speed was computed. This process was repeated for a number of other weigh-in-motion stations of the same functional classification. An average of these period averages was computed to represent the functional class. The stations were chosen to take into account the geographic distribution. Detour speed for a bridge inventory route of each functional class was calculated as using the 80% adjustment factor originally recommended in Pontis, for estimating detour speed from the primary route speed.

Method 2: Binned Data on speed for the traffic stream: This is available from FDOT Office of Transportation Statistics. At each weigh-in-motion station the speeds of all vehicles are classified into speed ranges with average speed, 85$^{th}$ percentile speed calculated on a daily basis. For a functional class of roadway (bridge roadway) a number of stations were selected to represent geographic distribution. Over a given time period the daily average speeds or 85 percentile speeds were averaged to obtain a representative speed for that functional class. Again, the 80% adjustment factor was applied.
Method 1 would appear to be more desirable because the data in the truck files is for trucks only whereas the Method 2 type data captures the entire traffic stream and does not distinguish vehicle type. However an analysis of the data for weigh-in-motion station 559908, US-319, Tallahassee, FL (Fig 2.32) revealed that the speed distributions for both methods are almost identical.

Speed distributions were developed from the Binned Data, for a number of stations chosen based on availability of data, to represent the geographic regions of the state for each functional class (Table 2.16). The average speed and 85$^{th}$ percentile speed can be obtained from the speed distribution curve.

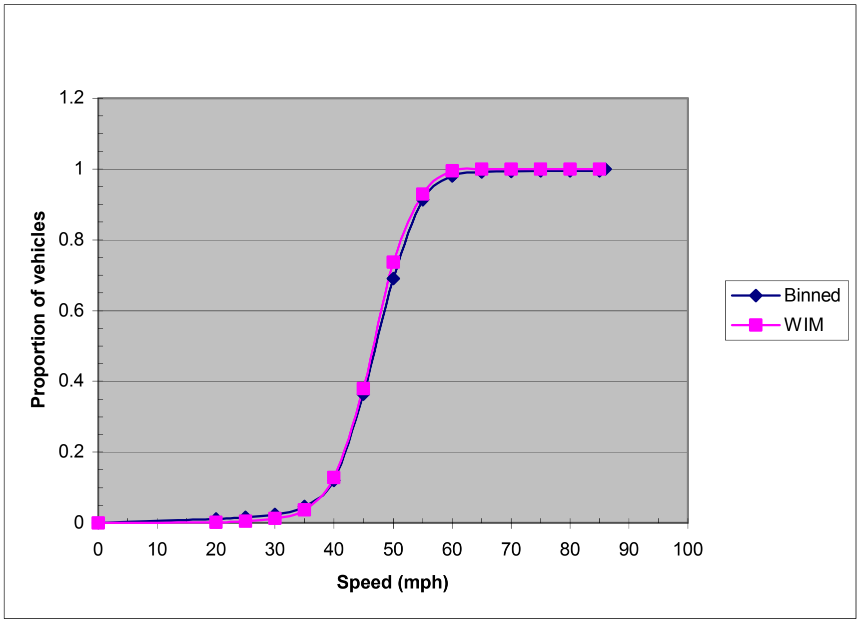



Figure 2.32: Comparison or Speed Data Sources, FDOT TTMS 9908, US-319, Tallahassee, FL, Functional Class 14, August 2002.

Table 2.16: Speed Data Collection Stations

| **Functional class** | **FDOT TTMS Site** |
|---|---|
| 01 | 299936, 549901, 010350, 700332 |
| 02 | 899921, 030270, 380280, 480348 |
| 06 | 480243, 720236, 120273, 880291 |
| 07 | 509940, 100276, 110136 |
| 11 | 860163, 100194, 550304 |
| 12 | 720216, 970430 |
| 14 | 559908, 720062, 870031, 460315 |
| 16 | 870258, 740182 |
| 17 | 860215, 880326, 160275 |

Method 3: speed data can be derived from the *Florida Traffic Information* (FTI) maintained by FDOT and updated annually. The FTI Access file has a summary table for all TTMS sites. The table has two columns for speed limit (maybe for the 2 directions). The stations were classified by functional class and county and the county average for each functional class was computed. (The converse yields the same the result, i.e. computing the functional class average for each county.). This approach was found to be the fastest and easiest way to gather speed data for the network, and was adopted for this study. It is noteworthy to point out that some counties do not have data for some functional classes. Furthermore, many counties have very limited data, yielding questionable averages. However, generally, the averages obtained for routes of a functional class in a given county were not alarmingly different from those of the same

functional class in other counties and functional classes. As with the preceding two methods the detour speed was assumed as 80% of the speed of the inventory route.

Some summary results are shown below in table 2.17 and Figure 2.33. It should be noted again, as mentioned earlier that Pontis now has available, the values for the detour speed for most bridges; it would therefore not be necessary anymore to go through the process just described above.

Table 2.17: Average Roadway and Detour Speeds

| Functional Class | Functional Class Average Speed (mph) | Functional Class Detour Speed (mph) |
|---|---|---|
| 1 | 69.80 | 55.84 |
| 2 | 60.21 | 48.17 |
| 6 | 55.36 | 44.29 |
| 7 | 53.75 | 43.00 |
| 8 | | |
| 9 | | |
| 11 | 63.39 | 50.71 |
| 12 | 63.13 | 50.50 |
| 14 | 49.70 | 39.76 |
| 16 | 45.50 | 36.40 |
| 17 | 35.00 | 28.00 |
| 19 | | |

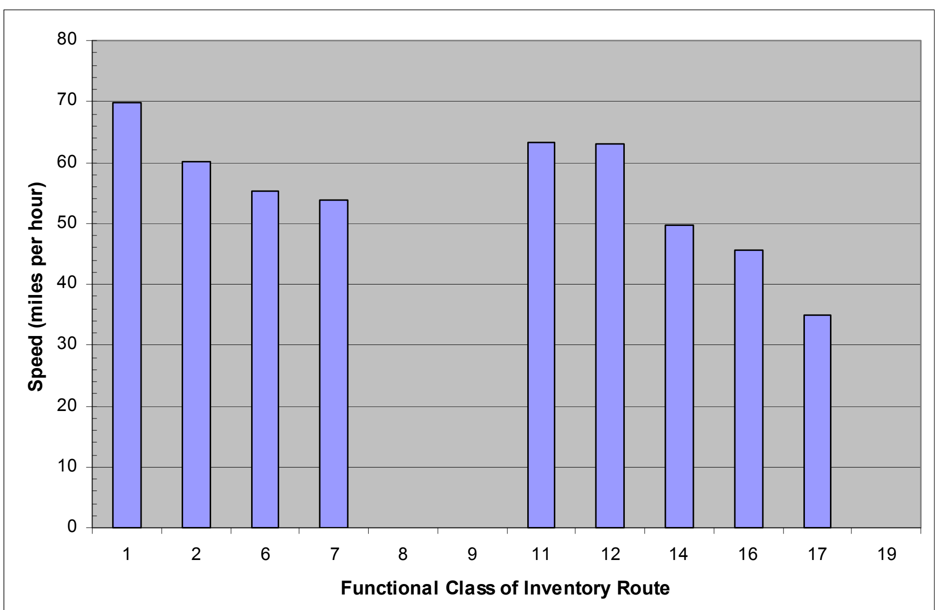



Figure 2.33: Average Roadway Speed By Functional Class

Table 2.18: FTI Speed Data For Functional classes 1, 2, and 6.

| | Fn Class 1 | | | Fn Class 2 | | | Fn Class 6 | | |
|---|---|---|---|---|---|---|---|---|---|
| County | AVE | MIN | MAX | AVE | MIN | MAX | AVE | MIN | MAX |
| 1 | 70 | 70 | 70 | | | | 45 | 45 | 45 |
| 2 | | | | 57.5 | 55 | 60 | | | |
| 3 | 70 | 70 | 70 | 60 | 60 | 60 | | | |
| 4 | | | | 60 | 60 | 60 | 60 | 60 | 60 |
| 5 | | | | | | | 55 | 55 | 55 |
| 8 | | | | | | | | | |
| 9 | | | | 65 | 65 | 65 | 55 | 55 | 55 |
| 10 | 65 | 65 | 65 | | | | | | |
| 11 | | | | | | | | | |
| 13 | | | | 57.5 | 55 | 60 | | | |
| 14 | 70 | 70 | 70 | 55 | 55 | 55 | | | |
| 15 | | | | | | | | | |
| 16 | | | | 65 | 65 | 65 | 55 | 55 | 55 |
| 17 | | | | | | | | | |
| 18 | 70 | 70 | 70 | | | | | | |
| 26 | 70 | 70 | 70 | | | | 45 | 45 | 45 |
| 27 | | | | | | | 60 | 60 | 60 |
| 28 | | | | 65 | 65 | 65 | 60 | 60 | 60 |
| 29 | | | | | | | 60 | 60 | 60 |
| 32 | 70 | 70 | 70 | | | | 55 | 55 | 55 |
| 33 | | | | | | | 60 | 60 | 60 |
| 34 | | | | 58.33 | 55 | 65 | 60 | 60 | 60 |
| 36 | 70 | 70 | 70 | 65 | 65 | 65 | | | |
| 37 | 70 | 70 | 70 | | | | 55 | 55 | 55 |
| 46 | | | | 60 | 55 | 65 | | | |
| 47 | | | | | | | 55 | 55 | 55 |
| 48 | 70 | 70 | 70 | 55 | 55 | 55 | 55 | 55 | 55 |
| 49 | | | | 55 | 55 | 55 | | | |
| 50 | 70 | 70 | 70 | 55 | 55 | 55 | | | |
| 51 | | | | | | | 55 | 55 | 55 |
| 53 | 70 | 70 | 70 | | | | 65 | 65 | 65 |
| 54 | 70 | 70 | 70 | 65 | 65 | 65 | | | |
| 55 | | | | 65 | 65 | 65 | | | |
| 56 | | | | | | | | | |
| 57 | 70 | 70 | 70 | 55 | 55 | 55 | 55 | 55 | 55 |
| 58 | | | | | | | 55 | 55 | 55 |
| 59 | | | | 55 | 55 | 55 | 55 | 55 | 55 |
| 60 | 70 | 70 | 70 | 55 | 55 | 55 | | | |
| 61 | 70 | 70 | 70 | | | | | | |
| 70 | 70 | 70 | 70 | 55 | 55 | 55 | 55 | 55 | 55 |
| 71 | | | | | | | | | |
| 72 | 70 | 70 | 70 | 60 | 60 | 60 | 55 | 55 | 55 |
| 73 | | | | | | | | | |
| 74 | 70 | 70 | 70 | 65 | 65 | 65 | | | |
| 75 | | | | 60 | 55 | 65 | | | |
| 76 | | | | 50 | 45 | 55 | | | |
| 77 | | | | | | | 55 | 55 | 55 |
| 78 | | | | | | | | | |
| 79 | | | | 65 | 65 | 65 | 45 | 45 | 45 |
| 86 | 70 | 70 | 70 | | | | | | |
| 87 | | | | | | | | | |
| 88 | | | | 55 | 55 | 55 | | | |
| 89 | | | | | | | | | |
| 90 | | | | 45 | 45 | 45 | | | |
| 92 | | | | 55 | 55 | 55 | | | |
| 93 | | | | 63.33 | 60 | 65 | | | |
| 94 | | | | | | | | | |
| 97 | | | | 69.17 | 65 | 70 | | | |
| | 69.8 | 65 | 70 | 60.21 | 45 | 70 | 55.36 | 45 | 65 |

## 2.8. Formulation of Models

### 2.8.1. Benefit of Raising

The model was formulated on an Excel spreadsheet of the NBI for the Florida bridge network for under routes maintained by the state highway agency and state toll authority. Columns were created for bridge under clearance and for each segment of the step function representing the truck height distribution obtained from the regression analysis. A column was created for the current (analysis) year average daily traffic (ADT). Columns were created for the proportion of trucks detoured, the detour cost per truck, and the user benefit of raising. A spreadsheet model was developed with the logical procedures for the major sub-procedures as shown in Figure 2.34.

Under clearance (Height): NBI Item 54B and Item 10 were used to calculate the under clearance as follows,

$$\text{Under clearance} = \begin{cases} \text{Item 10}, & \text{if Item 54B} = 0 \\ \text{Item 54B}, & \text{otherwise} \end{cases} \tag{2.1}$$

The NBI items required to calculate the current year ADT are Item 29-Average Daily Traffic, Item 30-Year of Average Daily Truck Traffic, Item 114-Future Average Daily Traffic, and Item 115-Year of Future Average Daily Traffic. For the current year Y,

$$ADT_Y = \begin{cases} \text{Item 29 * ((Item 114/ Item 29)\^{}((Y- Item 30)/(Item 115- Item 30))}, \\ \quad \text{if Item 29} > 0, \text{Item 30} > 1000, \text{Item 114} > 0, \text{Item 115} \geq Y \\ \text{Item 29}, \quad \text{otherwise} \end{cases} \tag{2.2}$$

The conditions for Item 29, Item 30, Item 114, and Item 115, were required to eliminate the effects of missing data and erroneous data entries.

To incorporate truck height distribution function, the piecewise (step) functions and the ranges were input so that the appropriate piecewise segment function would be applied to the under clearance depending on which range it fell. This was repeated in subsequent columns depending on the number of piecewise segments of the function. For a piecewise segment defined by the function P[under clearance], ranging from $HtRange_{MIN}$ to $HtRange_{MAX}$,

$$\text{Proportion of trucks detoured} = P_H[\text{under clearance}], \quad \text{if } HtRange_{MIN} \leq \text{Under clearance} \leq HtRange_{MAX} \tag{2.3}$$

For the curvilinear functional form of the truck height distribution, the assigning of ranges to the spreadsheet columns was not required and the polynomial function was input directly into the model.

To estimate the detour costs per truck, the vehicle operating cost (VOC), and values of travel time were tabulated separately. A detour speed column was created for each bridge and assigned the mean value computed for the functional class in the speed data analysis. Detour distance is available from NBI Item 19- Bypass/ Detour Length.

$$\text{Detour cost per truck} = \text{VOC} * \text{Item 19} + (\text{VOT} * (\text{Item 19/detour speed})) \tag{2.4}$$

The annual user benefit of raising for each bridge was obtained by multiplying the detour cost per truck by the number of trucks detoured. The NBI input necessary is item 109 – average daily truck traffic (%). For a bridge with under clearance restriction,

$$\text{Annual benefit of raising} = ADT_Y * (\text{Item 109/100}) * \text{proportion of trucks detoured} * \text{Detour cost per truck} * 365 \tag{2.5}$$

**2.8.2. Benefit of Strengthening**

The model was also formulated on an Excel spreadsheet of the Florida bridge network from NBI for over routes maintained by the state highway agency and state toll authority. Columns were created for bridge operating rating (lbs) and for each piecewise segment of the function representing the truck weight distribution obtained from the regression analysis. A column was created for the current (analysis) year ADT. Columns were created for the proportion of trucks detoured, the detour cost per truck, and the user benefit of strengthening.

The current year ADT was estimated as described earlier for the raising model. Truck weight distributions and models were also incorporated in a similar manner, to the raising model, in order to estimate the proportion of trucks expected to detour given each bridge's under clearance value.

$$\text{Proportion of trucks detoured} = P_W[\text{under clearance}], \quad \text{if } WtRange_{MIN} \leq \text{operating rating} \leq WtRange_{MAX} \tag{2.6}$$

Detour costs were also computed as follows:

$$\text{Detour cost per truck} = \text{VOC} * \text{Item 19} + (\text{VOT} * (\text{Item 19/detour speed})) \tag{2.7}$$

The annual user benefit of strengthening for each bridge was obtained by multiplying the detour cost per truck by the number of trucks detoured. The NBI input necessary is item 109 – average daily truck traffic (%). Considering that the operating rating is calculated based on the HS20 design truck, the listed values in NBI item 109 would have to be adjusted. The HS 20 design truck is an envelope vehicle for moments and shears, assumed adequate to represent most legal truck weights in the United States. An operating rating of 70,000 lbs. would be sufficient for a truck gross weight of 80,000 lbs., the legal weight limit. Therefore it would be necessary to adjust the listed operating rating by as ratio of 80,000lbs. to 70,000lbs. or 1.11.

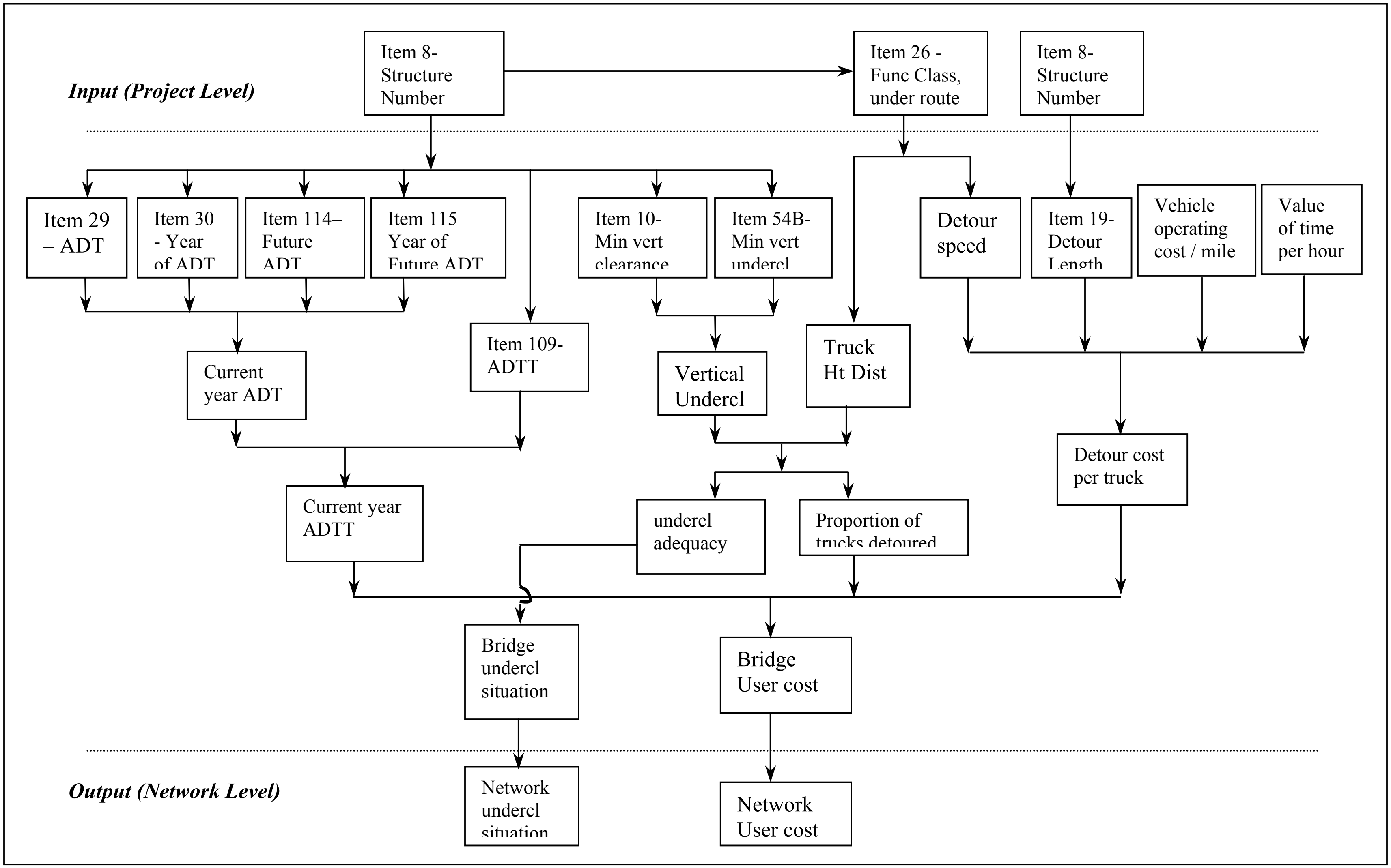



Figure 2.34: Procedure Flow Chart of Spreadsheet Model for Estimating the Benefit of Raising

For a bridge with load capacity restriction (inadequate operating rating),

Annual benefit of strengthening = $ADT_Y$ **((Item 109***1.11)/100) * proportion of trucks detoured * detour cost per truck * 365 (2.8)

The spreadsheet model developed for estimating user benefit of strengthening is illustrated by its procedure's flow chart is illustrated in Figure 2.35.

## 2.9. Moment Envelops Due to Truck Axle Loadings

Based on the permit procedure at FDOT, preliminary work was done to develop spreadsheets that would take axle loadings and spacing, as input, to develop moment envelopes. These spreadsheets were not rigorously validated, thus they are not reported for the study.

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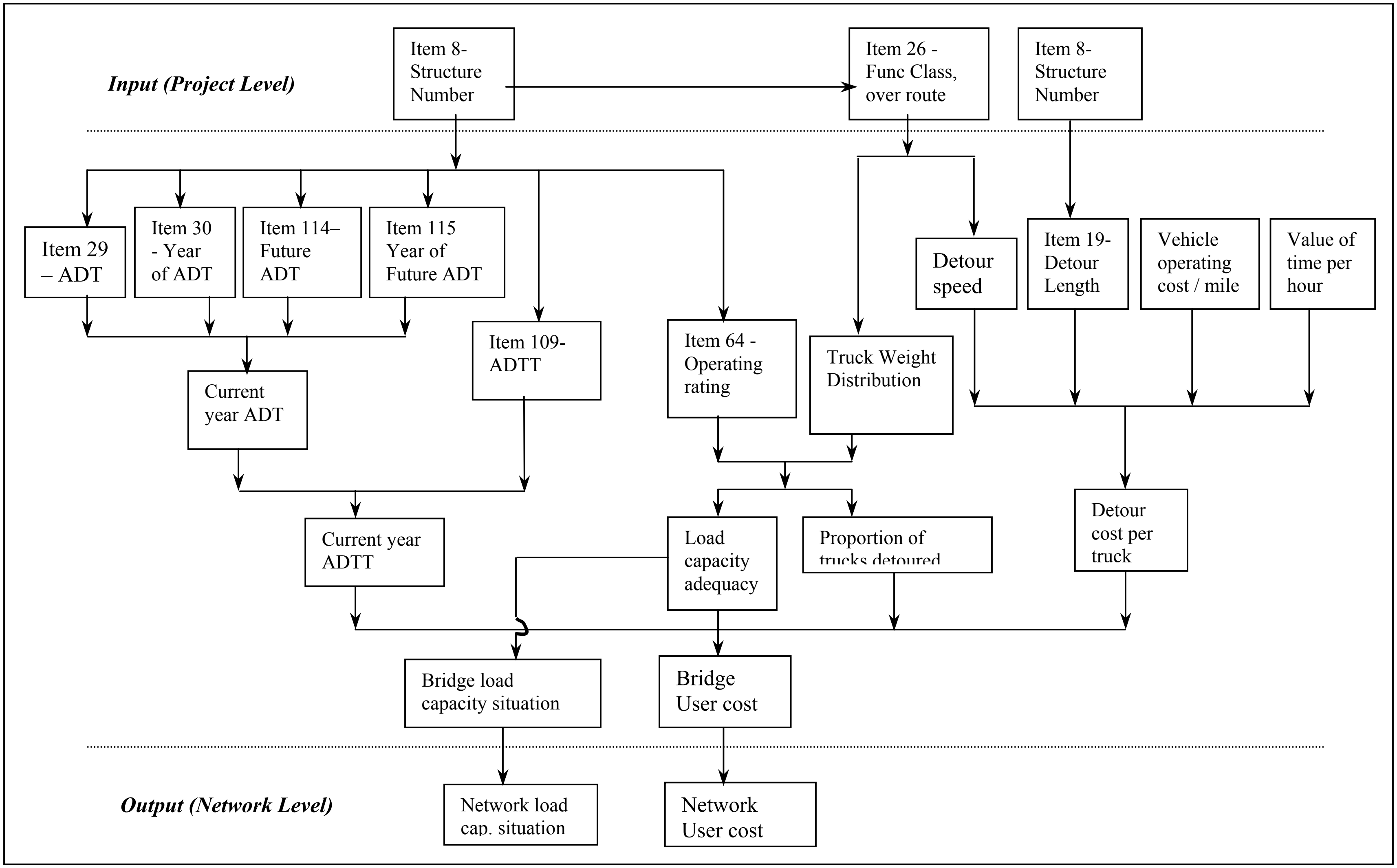


Figure 2.35: Procedure Flow Chart of Spreadsheet Model for Estimating the Benefit of Strengthening

# 3. User Cost Model Development

As indicated in the final report of the Florida DOT User Cost Study, Florida is interested in developing a user cost model for movable bridge openings, to help justify replacement projects for its large inventory of movable bridges. This section describes the development of the user cost model, which will quantify the economic impact of bridge openings, specifically the lost time due to delay of motorists. Working with the pertinent FDOT Offices, the researcher obtained suitable data on bridge opening frequency and duration of opening, for the various moveable bridges on Florida highways. On-site data were also collected on automobile and vessel traffic, including vehicle queue length and vessel height distributions. Decision making templates have been developed to correctly assign performance measures and priorities to moveable bridge replacement projects. Also, the researcher has suggested modifications to the Pontis software that would be required in order to implement the user cost model for movable bridge openings, by considering vehicular and vessel delays, and the load carrying capacity of existing bridges.

The main purpose of moveable bridges is to alternately permit two different intersecting traffic routes – vessel and vehicular and/or railway – in a practical manner. Vessels with heights greater than the vertical clearance under the bridge will have to queue up in a holding area till the bridge is opened while vehicles will also have to queue up during these openings. An increase in vehicular and vessel traffic therefore creates a greater demand for accommodation of the vehicular and vessel traffic as well as longer bridge openings for vessels that utilize the facility. This ultimately results in longer periods of delays to both vehicular and vessel traffic. These delays are quantifiable as costs to the users of the facilities and is termed as User Cost, which for this study will be defined as the monetary value of the extra travel time incurred by both vehicular and vessel traffic due to the opening of bridges (delay to vehicles) and due to bridge closure (delay to vessels). The study involved the identification, research and analysis of various factors that will be instrumental in the formulation of a template movable bridge user cost model for the FDOT to aid in any bridge replacement study.

## 3.1 Data Collection

The development of the user cost model required the review and input of vessel and vehicular traffic. Some information regarding the past and current vessel traffic was available from the FDOT Bridge opening logs. This information however did not provide data on the heights of vessels using the waterways or the duration of openings, which are critical for the analyses of different bridge replacement options. A vessel traffic survey was therefore required to be conducted, to obtain information such as the duration of each bridge opening, and the count of vessels and heights of vessels on queue before the opening.

A reconnaissance visit of some of the potential study sites was initially made to observe the operations of the moveable bridges and the characteristics of bridge openings and closures. It was also aimed at helping to select appropriate sites for the study. A total of six (6) bridge sites in FDOT District 2 were initially visited. The Impulse® Laser range finder, a hand-held laser equipment that has been selected for prior use in the measure of truck heights, was tried during the reconnaissance to ascertain its effectiveness for obtaining reliable data. As described in the next section, FDOT bridge opening logs were also obtained and reviewed.

### 3.1.1 Site Selection

A study and analysis of existing FDOT bridge opening logs aided in the selection of study sites where potentially relevant and adequate data could be obtained. The following primary data from the logs considered during this analysis, for each bridge site, were:

- Number of Vessels using the waterway over which the bridge spans
- Frequency of openings

The number of vessels and frequency of openings gives an indication of level of usage of the bridge opening. Sites that were selected based on the above considerations were further analyzed for final selection based on:

- The average daily traffic (ADT) on the roadway carried by the bridge
- Functional class of the inventory route and
- Geographical location of the sites in Florida

The ADT indicates the count of vehicles affected by the bridge openings. The model to be developed from the data collected is to be used as a state bridge management tool, therefore it is important that the data, as much as possible, should be representative of the functional classes of roadways, and also not be geographically biased.

Existing data on movable bridge openings were obtained from the FDOT bridge openings log sheets, which contained information such as the total monthly number of openings for each movable bridge and the total number of vessels for which the openings were made. The data obtained were for the period January 1981 to May, 2001.A ratio of the number of vessels to the number of openings was computed for each month of the entire period. The ratio of vessels per opening was used a measure of determining the level of vessel traffic through the bridge. A ratio of 2.00 vessels per opening was used as the minimum to sort out the data for further analysis. This procedure provided a list of 70, out of the total of 152 movable bridges, which have had at least a monthly average of 2 vessels demanding passage at each of its openings.

A detailed analysis was then made for each of these bridges to review based on a criterion such as the ratio of the actual monthly opening frequencies to the number of vessels, considering those with the ratios of 2.00 or more and how recent these ratios were. Several bridges were eliminated through this detailed analysis because their high rations of vessels per opening occurred mostly between 1981 and 1990 and have since been lower. Other bridges were also eliminated because despite having high ratios, the actual number of vessels and frequency of openings were low. The remaining bridges were sorted into their respective districts and subjected to a further selection process based on their roadway average annual daily traffic (AADT). Bridges with low vehicular volumes were eliminated because the selected study sites were intended to reflect areas where the impact of the openings is significant. Final selection was then made to reflect the geographical distribution of movable bridges in the state of Florida, and the frequency distribution of the state’s movable bridges relative to functional classes of the roadways carried by the bridge.

According to the 2002 inventory, Florida presently has a total of 152 movable bridges with 91% of them being of the bascule type, 7% swing and 2% lift. Of the state's seven (7) districts, district 4, with Fort Lauderdale as the district headquarters, has a total of 49 while District 3 has 1. Details of these distributions, which were obtained from the Florida Department of Transportation (FDOT) Bridge Management System's Bridge Inventory Report for June 2002, are given in the table 3.1.

Table 3.1 **-** Distribution of Movable Bridges (FDOT 2002)

| | **TYPE** | | | |
|---|---|---|---|---|
| **DISTRICT** | **LIFT** | **BASCULE** | **SWING** | *Total* |
| 01- BARTOW | 1 | 22 | 2 | *25* |
| 02- LAKE CITY | 1 | 8 | 1 | *10* |
| 03- CHIPLEY | 0 | 1 | 0 | *1* |
| 04- FT. LAUDERDALE | 0 | 47 | 2 | *49* |
| 05- DELAND | 0 | 12 | 4 | *16* |
| 06- MIAMI | 0 | 27 | 1 | *28* |
| 07- TAMPA | 1 | 21 | 1 | *23* |
| *Total* | *3* | *138* | *11* | *152* |

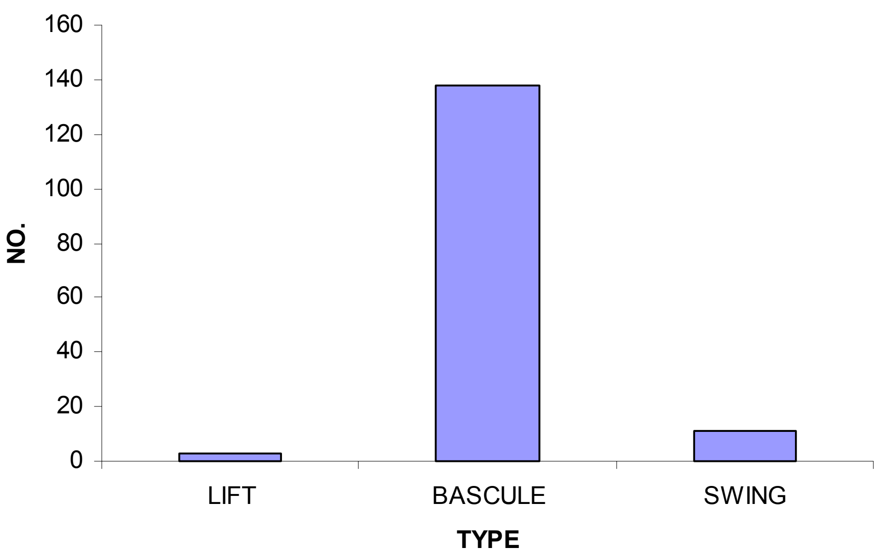


Figure 3.1 Distributions of Florida Moveable Bridges by Type

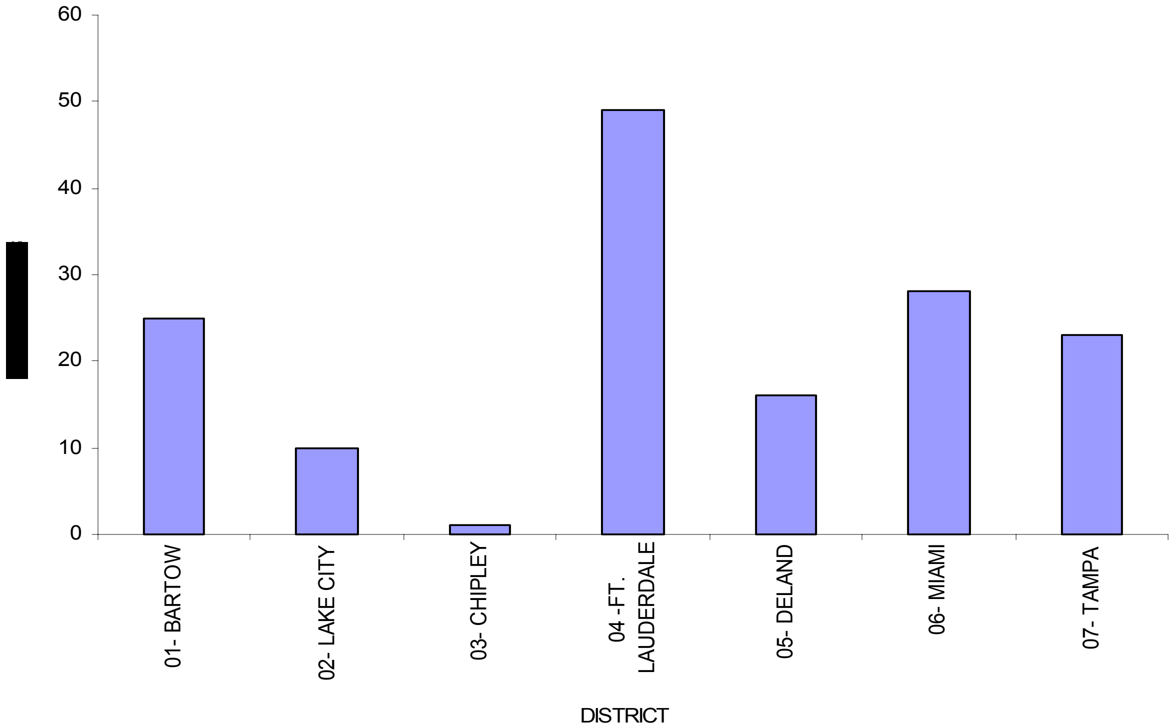


Figure 3.2 Distributions of Florida Moveable Bridges by FDOT District

A study of the functional classes of the roadways carried by movable bridges showed that most movable bridges, about 86%, carry roadways that serve urban vehicular traffic roadways i.e. Roadways with functional classes between 11 and 19. The distributions of the functional classes are shown in table 3.2 and figure 3.3 below. A study of the bridge opening log sheet data also indicated that most of the bridges carrying rural roadways, i.e. roadways with functional classes between 1 and 10 have relatively lower vessel traffic and consequently fewer openings.

For the study therefore bridges carrying urban roadways, i.e. Roadways with functional classes between 11 and 19 were selected, specifically functional classes 14, 16 and 17, which together represent about 80% of all the various roadways carried by movable bridges in the State of Florida.

Table 3.2 Distribution of Moveable Bridges by Roadway Functional Class

| Functional Class | Number | Percentage |
|---|---|---|
| 2 | 9 | 6.1 |
| 6 | 4 | 2.7 |
| 7 | 6 | 4.1 |
| 9 | 1 | 0.7 |
| 11 | 1 | 0.7 |
| 12 | 2 | 1.4 |
| 14 | 36 | 24.3 |
| 16 | 59 | 39.9 |
| 17 | 24 | 16.2 |
| 19 | 6 | 4.1 |

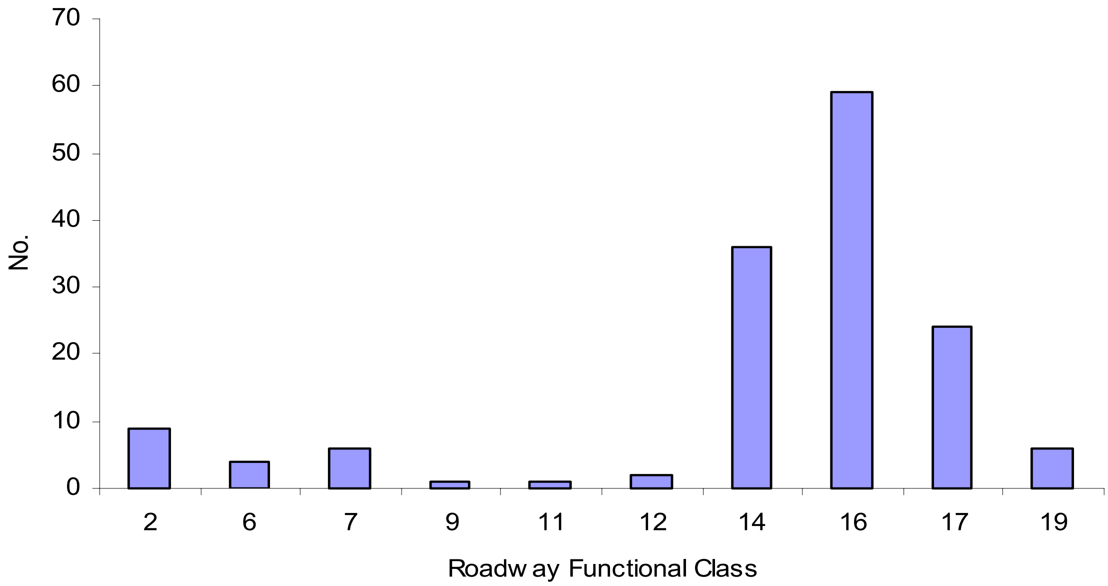


Figure 3.3 Distributions of Florida Moveable Bridges by Roadway Functional Class

Based on the criteria and methodology discussed above an analysis of the existing data obtained from the National Bridge Inventory (NBI), the FDOT Bridge Report, and the FDOT Movable bridges opening log, twelve bridges sites were selected as potential study sites out of which six were eventually used as data collection sites.

Table 3.3 List of Initial Potential Study Sites

| | Structure Number | Highway Agency District | County | Features Intersected | Functional Class of Inv. Rte. |
|---|---|---|---|---|---|
| 1 | 720005 | 2 | Duval | ORTEGA RIVER | 17 |
| 2 | 720068 | 2 | Duval | INTRACOASTAL WATERWAY | 14 |
| 3 | 720069 | 2 | Duval | INTRACOASTAL WATERWAY | 14 |
| 4 | 780074 | 2 | St. Johns | ICWW MATANZAS RIVER, ST AUG | 16 |
| 5 | 860011 | 4 | Broward | SR A1A OVER HILLSBORO | 16 |
| 6 | 860060 | 4 | Broward | I.C.W.W | 17 |
| 7 | 930004 | 4 | Palm Beach | I.C.C.W. | 14 |
| 8 | 930064 | 4 | Palm Beach | SR-806 OVER ICWW | 12 |
| 9 | 930157 | 4 | Palm Beach | INTRACOASTAL WATERWAY | 14 |
| 10 | 150050 | 7 | Pinellas | INTRACOASTAL WATERWAY | 16 |
| 11 | 150027 | 7 | Pinellas | JOHNS PASS BOCACIEGA BAY | 16 |
| 12 | 150076 | 7 | Pinellas | JOHNS PASS BOCACIEGA BAY | 16 |

Table 3.4 Inventory Data on Potential Study Bridge Sites.

| | Structure Number | Type of Design / Constr. | Highway Agency District | County | Features Intersected | Facility Carried By Structure | Location | Functional Class of Inv. Rte. | AADT (FTI) -2001 |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 720005 | Bascule | 2 | Duval | ORTEGA RIVER | SR-211 | SR 211 OVER ORTEGA RIVER | 17 | 5800 |
| 2 | 720068 | Bascule | 2 | Duval | INTRACOASTAL WATERWAY | US-90 W.B. (SR-212) | U.S.-90 / INTRACOASTAL WY | 14 | 19047 |
| 3 | 720069 | Bascule | 2 | Duval | INTRACOASTAL WATERWAY | US-90 E.B. (SR-212) | U.S.-90 / INTRACOASTAL WY | 14 | 19047 |
| 4 | 780074 | Bascule | 2 | St. Johns | ICWW MATANZAS RIVER. ST AUG | SR-A-1-A (LIONS) | IN ST. AUGUSTINE | 16 | 23000 |
| 5 | 860011 | Bascule | 4 | Broward | SR A1A OVER HILLSBORO | SR-A1A | OCEAN BLVD AT HILSBRO INL | 16 | 9900 |
| 6 | 860060 | Bascule | 4 | Broward | I.C.W.W | SR-844 (14 ST.CSWY) | 300' W OF A1A & E OF SR-5 | 17 | 13500 |
| 7 | 930004 | Bascule | 4 | Palm Beach | I.C.C.W. | SR-5 (US-1) | 1.6 KM SOUTH OF SR 786 | 14 | 24500 |
| 8 | 930064 | Bascule | 4 | Palm Beach | SR-806 OVER ICWW | SR-806 | 800' W OF A1A & E OF SR-5 | 12 | 12500 |
| 9 | 930157 | Bascule | 4 | Palm Beach | INTRACOASTAL WATERWAY | SR A1A | 200 M. W.OF SR 5 ON A1A | 14 | 21500 |
| 10 | 150050 | Bascule | 7 | Pinellas | INTRACOASTAL WATERWAY | SR-682 | 5.3KM WEST OF US 19 | 16 | 17900 |
| 11 | 150027 | Bascule | 7 | Pinellas | JOHNS PASS BOCACIEGA BAY | SR - 699 S.B. (GULF BLVD) | 2.7KM S OF SR 666 | 16 | 10500 |
| 12 | 150076 | Bascule | 7 | Pinellas | JOHNS PASS BOCACIEGA BAY | SR - 699 N.B. (GULF BLVD) | 2.7KM S OF SR 666 | 16 | 10500 |

Table 3.4 Inventory Data on Potential Study Bridge Sites (Continued).

| | Structure Number | Year Built | Nearest FTI Station | Nav. Vert. Clearance. (m) | Nav. Hor. Clearance. (m) | Opening Regulation |
|---|---|---|---|---|---|---|
| 1 | 720005 | 1927 | 72-0188 | 2.7 | 16.1 | No special regulations |
| 2 | 720068 | 1949 | 72-0062 | 11.2 | 27.4 | No special regulations |
| 3 | 720069 | 1949 | 72-0062 | 11.2 | 27.4 | No special regulations |
| 4 | 780074 | 1927 | 78-0114 | 7.6 | 23.1 | B/n 7am -6pm opens only on :00, :30, Need not open at 8:00 am, 12 noon and 5:30 pm |
| 5 | 860011 | 1966 | 86-0311 | 4.0 | 18.3 | B/n 7am &6pm opens :00: 15, :30, :45 |
| 6 | 860060 | 1967 | 86-0482 | 8.5 | 27.0 | B/n 7am &6pm opens: 15,: 45 |
| 7 | 930004 | 1956 | 93-0756 | 7.6 | 29.0 | Weekdays b/n 7-9am &4-7pm opens :00 &: 30, Weekends b/n 8am-6pm opens :00, :20, :40. |
| 8 | 930064 | 1952 | 93-0681 | 2.7 | 24.4 | Open on signal |
| 9 | 930157 | 1938 | 93-0087 | 5.2 | 2.4 | Open on signal |
| 10 | 150050 | 1962 | 15-3075 | 9.3 | 13.4 | B/n 7am &7pm opens :00, :20,: 40 |
| 11 | 150027 | 1971 | 15-0017 | 7.0 | 18.3 | Open on signal |
| 12 | 150076 | 1971 | 15-0017 | 6.0 | 18.3 | Open on signal |

Table 3.5 Vessel Traffic Data on Potential Study Bridge Sites*

| | Bridge No. | Monthly Volume | | | | | | Typical Peak Period | Avg. Daily Vessels | Avg. Daily Openings |
|---|---|---|---|---|---|---|---|---|---|---|
| | | L-lowest | YEAR | | | | | | | |
| | | H-Highest | 1997 | 1998 | 1999 | 2000 | 2001 | | | |
| 1 | **720005** | L | 743 | 830 | 968 | 99 | 861 | Mar-Nov | 45 | 31 |
| | | H | 1874 | 1953 | 1916 | 1696 | 1780 | | | |
| 2 | **720068** | L | 148 | 132 | 114 | 61 | 59 | Apr-May, Oct-Nov | 9 | 7 |
| | | H | 710 | 664 | 752 | 536 | 406 | | | |
| 3 | **720069** | L | 148 | 132 | 114 | 61 | 59 | Apr-May, Oct-Nov | 9 | 7 |
| | | H | 710 | 664 | 752 | 536 | 406 | | | |
| 4 | **780074** | L | 509 | 347 | 391 | 411 | 369 | Apr-May, Oct-Nov | 26 | 15 |
| | | H | 1252 | 1309 | 1417 | 1381 | 1219 | | | |
| 5 | **860011** | L | 1070 | 1966 | 2101 | 2502 | 2038 | All Year | 109 | 44 |
| | | H | 4070 | 4988 | 4369 | 4519 | 5312 | | | |
| 6 | **860060** | L | 1392 | 1099 | 1396 | 1408 | 1146 | Nov-May | 68 | 30 |
| | | H | 3041 | 2551 | 2700 | 2584 | 2693 | | | |
| 7 | **930004** | L | 432 | 558 | 522 | 518 | 446 | Nov-May | 33 | 21 |
| | | H | 1296 | 1327 | 1300 | 1436 | 1278 | | | |
| 8 | **930064** | L | 499 | 511 | 502 | 716 | 537 | Oct-May | 36 | 23 |
| | | H | 1183 | 1471 | 1647 | 1516 | 1669 | | | |
| 9 | **930157** | L | 454 | 706 | 585 | 600 | 571 | Oct-May | 38 | 22 |
| | | H | 1842 | 1784 | 1884 | 2016 | 2112 | | | |
| 10 | **150050** | L | 447 | 485 | 692 | 669 | 375 | Oct-Nov, Mar-May | 32 | 19 |
| | | H | 1499 | 2024 | 1345 | 1271 | 1426 | | | |
| 11 | **150027** | L | 583 | 594 | 691 | 691 | 630 | Oct-Nov, Mar-May | 34 | 26 |
| | | H | 1607 | 1304 | 1433 | 1515 | 1381 | | | |
| 12 | **150076** | L | 583 | 594 | 691 | 691 | 630 | Oct-Nov, Mar-May | 34 | 26 |
| | | H | 1607 | 1304 | 1433 | 1515 | 1381 | | | |

*Based on the summary of the available vessel traffic data for five (5) years; the lowest and highest monthly vessel traffic volume for each year have been indicated.

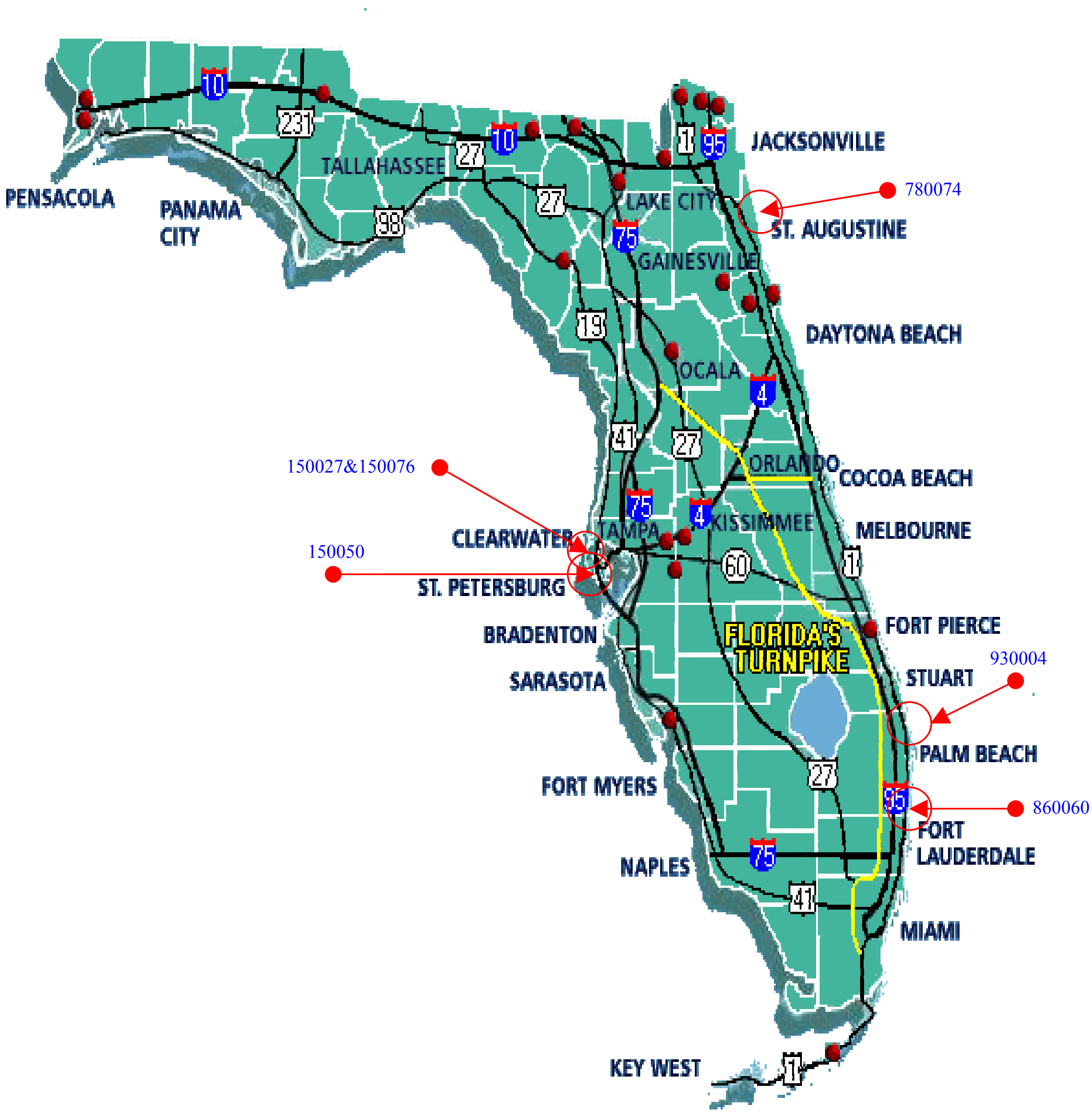


Figure 3.4 Final Selected Study Sites for Movable Bridge Survey

The Movable bridge inventory report indicated that about 40% of the movable bridges in Florida carry roadways over the Intracoastal Waterway, a brief description of which is given in the following paragraphs. The Intracoastal Waterway (ICCW) is a 2,640-mile federally and locally maintained system of natural water bodies and connecting canals paralleling the Atlantic and Gulf coasts of the United States. The purpose of the waterway is to provide a protected environment for vessels moving coastwise, particularly shallow-draft commercial and recreational vessels.

It was originally envisioned as a continuous navigable waterway that would stretch from Trenton, New Jersey through Miami, Florida, to Brownsville, Texas but the channel through northwest Florida that was needed to join the two coasts was never completed. Therefore the ICCW is now in two separate sections: the Atlantic Intracoastal Waterway (AIWW) and the Gulf Intracoastal Waterway (GIWW)._The Atlantic Intracoastal Waterway is a 1,391-mile channel between Trenton, New Jersey, and Miami, Florida. The channels from Trenton to the St. Johns River in Florida are 12 feet deep, 90 feet wide through lands and generally 150 or 300 feet wide in open waters. The channel south from the St. Johns River was authorized to be constructed as a 12-foot by 125-foot channel throughout, but was modified to a 10-foot depth from Fort Pierce south to Miami. By way of this waterway, one can travel the length of the Atlantic coast without ever actually venturing out into the Atlantic. Boats pass along rivers and streams and sounds and bays and swamps, all interconnected, and dredged to a close to constant depth.

The Gulf IntracoastalWaterway is a 1,100-mile long channel between Brownsville, Texas and St. Marks, Florida, south of Tallahassee. The channel is 150 feet wide and 12 feet deep. From Tarpon Springs to south to Fort Myers, a distance of 150 Miles, there is 9-foot channel that is officially known as the Intracoastal Waterway but generally considered as part of the Gulf Intracoastal Waterway. The construction of the waterway from St. Marks to Tarpon Springs was never constructed and as a consequence, boats operating between the two places must enter the open waters of the Gulf for 135 miles. From Fort Myers on Florida’s west coast to Stuart on the east coast, the 8-foot deep Okeechobee waterway provides a linkage between the GIWW and the AIWW. Maintenance of the State of Florida’s portion of the ICWW is provided by the Jacksonville District Corps of Engineers in cooperation with the Florida Inland Navigation District (www.aicw.org), which was created in 1927 by the Florida Legislature as the local sponsor of the Atlantic Intracoastal Waterway project from Jacksonville to Miami.


Figure 3.5 Atlantic Intracoastal Waterway

### 3.1.2 Equipment Selection

The equipment selected for collection of vessel height data was the Impulse® Laser Range Finder. As described earlier in Chapter 2, and also calibrated as shown in Appendix B, the rangefinder is a lightweight distance, angle and height-measuring device and could be operated either hand-held or fixed to a bracket and mounted on a tripod. It was found appropriate for measuring vessel heights due to its ease of operation and handling. It can be used to accurately measure distances and heights from distances up to over 500 feet from the object. It can also be effectively used to measure heights and distances of moving objects. These attributes made the Impulse an ideal for taking the heights of the vessels, which may be in motion through the bridge, and at some distance, depending on the width of the waterway at the bridge site. A calibration test was performed on the instrument to determine its level of accuracy by taking the height of a known object repeatedly at various distances from the object (see Appendix B).

## 3.2 Overview of User Cost Methodology

The research methodology used in this study is an adoption and modification of the Delghani et al (1993)*'s* empirical analysis method, developed as part of a project development study for estimating delays to boats and vehicular traffic caused by movable bridge openings. The estimation was used in the economic analysis to rank the proposed replacement facilities to the 25 feet movable bascule bridge carrying S.E. 17th Street across the Intercoastal Waterway (ICWW) in Fort Lauderdale, Florida.

### 3.2.1 Queue Model

A queuing process occurs at movable bridges during the closure and opening of the bridge, a queue of vessels on the waterway when the bridge is closed and a queue of vehicles on the roadway carried by the bridge when the bridge is opened. The vehicular delay can be modeled as a bottleneck occurrence on the roadway, where the service flow rate of vehicles is reduced due to a blockade, which in this case would be the opening of the bridge and would be equal to zero since no vehicle is serviced by the bridge during the opening.

The Adolf May (1990) bottleneck model can be formulated is follows:

Duration of queue, $t_q = r(s - s_r) / (s - q)$ (3.1)

Number of vehicles affected, $N = q * t_q$ (3.2)

Average number of minutes of vehicle delay, $d = r*(q - s_r) / 2q$ (3.3)

Total vehicle minutes of delay, $D = r * N / 2$ (3.4)

Where,
$q$ = average arrival rate of traffic (vehicles per minute) upstream of bottleneck
$s$ = saturation flow rate or capacity of uninterrupted flow in veh/ hr/ lane
$s_r$ = flow rate at bottleneck during blockade,= 0, when bridge is open to vessel traffic
$r$ = duration of blockade (bridge opening time in minutes)
$t_0$ = time for the queue to dissipate after the blockade is removed in minutes
$t_q$ = total elapsed time (mins) from when start of the blockade (bridge opening) until free flow resumes, i.e. $[t + t_0]$

### 3.2.2. User Cost Model

User cost models quantify in economic terms, the benefits to the user of functional improvements to a physical infrastructure. In bridge management systems, the user cost model predicts the benefits of improved safety (reduction of accident costs) and/or improved mobility (reduction of operating costs and reduction of travel time/delays) of functional improvements or replacement. For movable bridges a user cost model would be used to quantify or estimate, in economic terms, the potential user benefits of replacing a movable bridge. Primarily, delay periods estimated from the queue models for different replacement options would form the basis for a user cost analysis. There is also the functional deficiency of bridge load capacity, which is considered secondary in the user cost model, but addressed later in the report. The user cost analysis involved the following: (1) the review of the latest boat traffic data; and (2) the analysis of alternative bridge replacement options. The following primary data are therefore required: Value of travel time; Boat Traffic survey data; and vehicular traffic data.

### 3.2.3. Value of Travel Time

Roadway user costs are incurred by highway users traveling on that facility and those who cannot use the facility because of either agency or self-imposed detour requirements. User costs usually have three components: Vehicle Operating Costs (VOC), Crash Costs, and User Delay Costs.

For an analysis of User Costs due to moveable bridge openings the most relevant will be User Delay Costs. These are the costs of increased travel time incurred by the motoring public who are held up on either side of a moveable bridge during its opening times for the passage of tall vessels. Users cost rates refer to the dollar values assigned to each user cost component. The user delay cost rate used in the analysis is the dollar value of an hour of delay or travel time resulting from the opening of a moveable bridge. Based on the results of some previous research on travel time models, values of time were obtained and updated for the current year. The existing time models reviewed included the following: The Highway Economic Requirement System (HERS) Model, which is being used by the Federal Highway Administration (FHWA); and the MicroBENCOST, which is a model developed under the National Cooperative Highway Research Program (NHCRP).

The table below gives the values of travel times for estimated from these sources for August 1996.

Table 3.6 Listing Of Travel Time Values, August 1996

| Source | Units | Autos | Single Unit-Trucks | Combination Trucks |
|---|---|---|---|---|
| MicroBENCOST | $/Vehicle-Hour | 11.37 | 17.44 | 24.98 |
| HERS | $/Vehicle-Hour | 14.30 | 25.99 | 31.30 |

Based on the consideration of these potential sources, the ranges of the value of travel time per vehicle recommended for use in typical analyses, where distribution data on trip purpose and type are not known, are as given below:

Table 3.7 Recommended Values Of Time ($/Vehicle-Hour), August 1996

| Passenger Cars | Trucks | |
|---|---|---|
| | Single-Unit | Combination |
| $ 10- $13 | $17 - $20 | $21 - $24 |

A prior study by Thompson et al. (1999) on user cost on Florida bridges, with high emphasis on trucks, recommended using $26.43 per hour in 1998 dollars. Since the proportion of passenger cars is relatively much more than trucks on these routes for the movable bridges, it is suggested that different rates be used for each vehicle class. A time value of $12.00 per vehicle-hour was assigned to passenger cars and $18.5 per vehicle-hour to trucks. The average value of travel time used for the analysis was then calculated based on the percentage of passenger cars and trucks in the traffic stream.

Roadways carried by moveable bridges usually have vehicle weight restrictions and are therefore mostly traveled by passenger cars and very light trucks. According to the FDOT's Traffic Data (FTI), the percentage of these light trucks observed going over the moveable bridge during the survey are between 5 and 10 percent of the total number of vehicles in the traffic stream. The estimated percentage of trucks traversing various sections of each roadway around moveable bridges is available in the FTI database.

The 1996 study by the National Cooperative Highway Research Program (NHCRP) had estimated a unit cost for travel time, which was used in the MicroBENCOST model. The1996 value was therefore selected as the base year value for the estimation of present and future travel time costs. Present and future cost of travel times were then estimated from the ratio of Consumer Price Indices (CPI) of the year of estimation to that of the base year and is formulated as:

$$T = B * \frac{T\,(CPI)}{B\,(CPI)} \tag{3.5}$$

Where
T = Estimated travel time cost for present of future year
B = Travel time cost for base year (1996)
T (CPI) = Consumer Price Index for the present or future year
B (CPI) = Consumer Price Index for the base year

## 3.3. Vehicular and Vessel Characteristics Data

As discussed earlier, five geographically spread locations were selected for data collection, with one of the locations having twin bridges. The data collected included primarily vehicular and vessel characteristics relevant to the development of the user cost models, including vessel count in queue, vessel heights, vehicular count in queue, and bridge opening times and duration of each opening. At one of the locations -- Bridge ID 780074 (Bridge of Lions), very little vessel traffic was observed during the data collection period of January 10, to January 12, 2003. The data for this location is therefore not reported but the site is briefly described. Reasonable size of data were collected at the other four sites, listed as follows:

1. Bridge ID 860060 (N.E. 14th Street Causeway) for Study Period 12/13/02 – 12/19/02.
2. Bridge ID 930004 (Parker) for Study Period: 1/17/03 – 1/19/03.
3. Bridge IDs150027 & 150076 (St. John's Pass) for Study Period: 3/28/03 – 3/29/03.
4. Bridge ID 150050 (Pinellas Bay Way) for Study Period: 5/8/03 – 5/11/03.

This section has been organized to first present for each site, a site description, and then tabular and graphical summaries of collected data.

### 3.3.1 Bridge ID 860060 (N.E. 14th Street Causeway) for Study Period 12/13/02 – 12/19/02

Bridge No. 860060, named as the 14th street Causeway Bridge, is a double leaf bascule bridge that carries the four-lane divided SR 844, locally named as N.E. 14th Street, across the Atlantic Intracoastal Waterway (ICWW) at Pompano Beach in Broward County, FL. The street connects SR A1A and US1. SR A1A is located about 800 feet to the east of the bridge whiles US1 is about 2000 feet to the west of the bridge. There is a boat building and repair yard located about 800 feet north of the bridge. On the northwest side of the bridge is a recreational park and a parking lot with a boat ramp. There is another moveable bridge, No. 860157, about a mile south of the 14th street causeway and this carries SR 814, Atlantic Boulevard, across the ICWW. Opening times of the two bridges are staggered at 15 minutes intervals to allow vessels traversing one to arrive at the other in time for an opening.

The 14th street causeway bridge has the following opening regulations:
7am – 6pm: Bridge opens on the quarter hour and on the three-quarter hour. It also opens on demand for US Public Vessels, Tugs in Tow and Vessels in Distress.
6pm – 7am: Bridge opens on demand.
The Atlantic Boulevard Bridge opens on the hour and on the half-hour.

During the period of the data collection at the bridge, under clearance for the vessels using the waterway were recorded to be between 14 and 16 feet. This depended on the rise and fall of the tide.

The waterway serves both recreational and commercial vessels. A lot of small vessels such as personal watercraft, Rowing Dinghies, canoes, water-ski boats and small fishing boats which needed no opening were observed to use the waterway. Coast Guard and the local Sheriff Patrol boats were on regular patrols. Barges with heavy construction equipment and materials were observed to travel upstream and back for 3 days of the seven-day study. The bridge was usually opened upon their approach and stayed open for longer periods than the average for each of the openings due to the slow movement of the barges.

NE 14th Street is classified as a functional class 17 roadway, and is a collector for the minor arterial SR- A1A. It has an estimated Average Annual Daily Traffic (AADT) of 13500.
Queue lengths observed ranged from 500 feet to over 1200 feet. Vehicles at several times in the day backup all the way onto SR A1A, which being a two- lane highway have through vehicles being delayed by turning vehicles which have also been held up due to the bridge opening. Some vehicles traveling south on SR-A1A and initially intending to turn onto the N.E. 14th Street were seen backing out of the queue and rerouting back onto SR- A1A to use Atlantic Boulevard as a detour route.

An annual boat parade is held along the waterway every year and this year's event was held on the Sunday of the data collection week. Sundays are usually one of the busiest days for vessel traffic but the scheduled boat parade resulted in relatively low vessel traffic. An interaction with the bridge tender revealed that most boats were waiting upstream to join the parade. The bridge opened for the parade at 6 pm and stayed open for duration of about 75 minutes. A total of 52 vessels were observed as part of the parade but people who had lined up along the waterway to watch the parade (some coming in as early as 10 am) expressed some disappointment over the number of vessels that came through. An estimated number of 150 vessels were expected.

The summaries of recorded data from the boat survey are shown in Figures 3.6, 3.7 and 3.8 along with the effects of feasible replacement alternatives indicated.

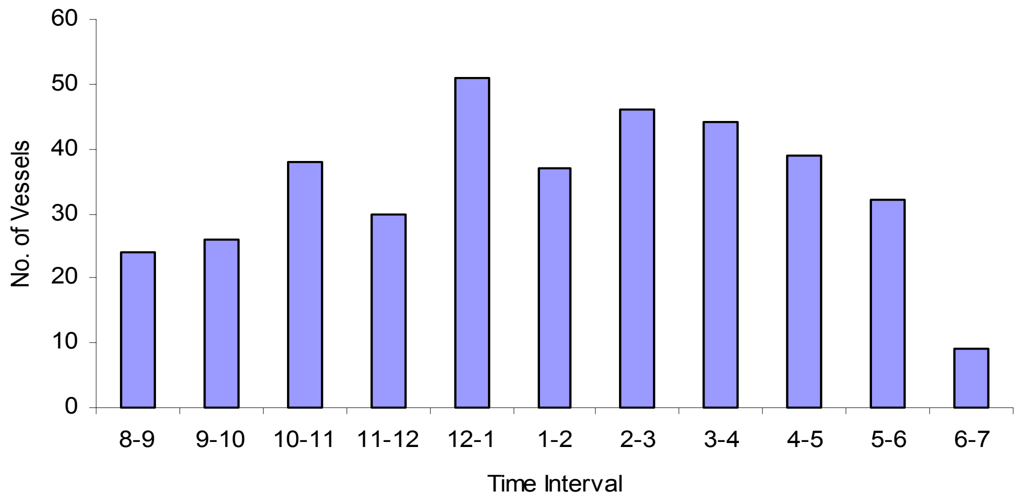


Figure 3.6 Hourly Distribution of all vessels – Bridge 860060

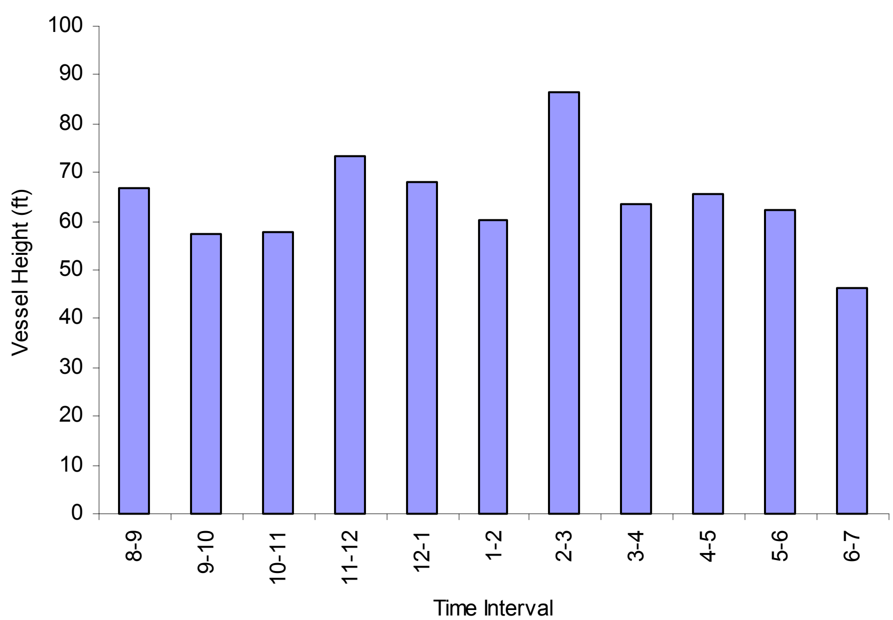


Figure 3.7 Hourly Distributions of Tallest Vessel Heights - Bridge 860060

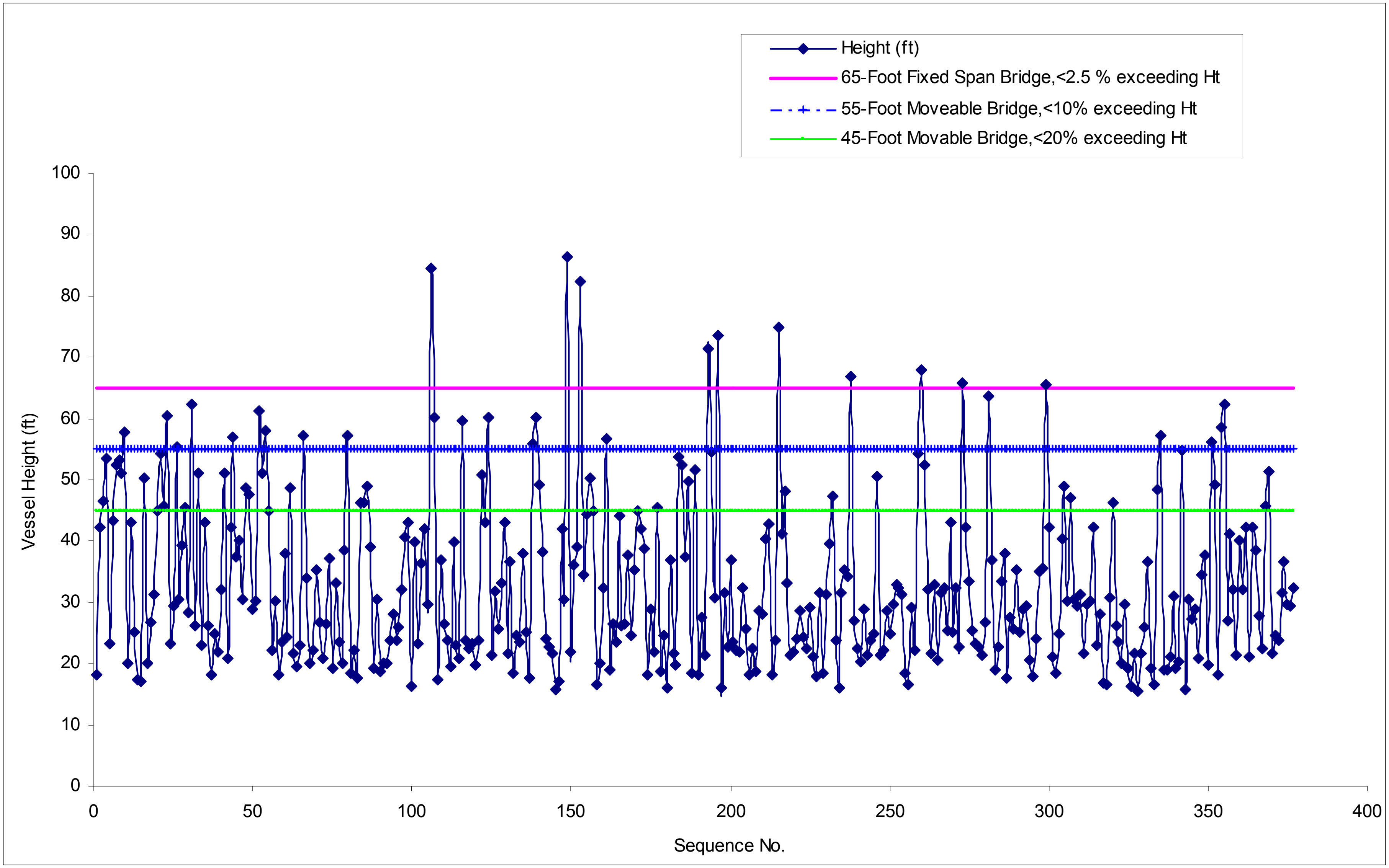


Figure 3.8 Bridge 860060 – Survey Results and Bridge Replacement options

### 3.3.2 Bridge ID 780074 (Lions) for Study Period 1/10/03 – 1/12/03

Bridge 780074, named as the Bridge of Lions is a single leaf bascule that carries SR A1A in St. Augustine in St. Johns County, across the Intracoastal Waterway (ICWW). It is about 1500 ft long with a two-lane undivided roadway over the bridge section, which opens up into four-lane divided sections on either side of it. Long high fixed bridges can be seen up north and down south of the bridge. The northern bridge carries SR A1A back across the ICWW at Vilano, while the southern bridge carries SR 312 across the ICWW.

During the period of data collection at the bridge, under clearance for the vessels using the waterway was recorded to be between 23 and 27 feet. This depended on the rise and fall of the tide. Generally, vessels under 23 feet required no opening but a couple of vessels with heights well below the under clearance available at their time of arrival at the bridge waited for the bridge opening. This resulted in extra or unnecessary delay to vehicles using the roadway.

The bridge is regulated to open on the hour and on the half-hour between 7 am and 6pm each day but opens on demand for Coast Guard vessels, Tugboats with Barges and for Commercial Vessels. It does not have to open at 8 am, 12pm and 5pm. It is manned 24 hours each day.

It has a wide holding area for vessels, and 45 vessels south of the bridge and 15 vessels north of the bridge were anchored for the entire 3-day study period. There's also a marine bay on the south side of the bridge. Information obtained from the bridge tenders indicated that vessel traffic was usually low at the time of year due to the cold weather. Sailors preferred to stay down south of the Intracoastal Waterway where it is warmer. This the researchers found out to be true, from the very low number of vessels and number of openings recorded during the study period. The temperature during the study days ranged between $37^0$ and $43^0$ F. Pedestrian traffic was quite high throughout each day; the walkway at the sides of the bridges seemed to be a favorite route for strollers and joggers. A lot of cyclists were also observed.

SR A1A, which is carried by the bridge, is classified as functional class 16, minor arterial roadway. It has an estimated Average Annual Daily Traffic (AADT) of 13500.Queue lengths observed ranged from 700 feet to over 1500 feet.

### 3.3.3 Bridge ID 930004 (Parker) for Study Period 1/17/03 – 1/19/03

Bridge No. 930004 is located in North Palm Beach, Florida and carries SR-5 (US-1), which is a 4-lane divided roadway, across the Intracoastal Waterway. It is a double leaf bascule bridge with a maximum under clearance of about 25 feet. The bridge is about 500 feet long with a waterway of about 350 feet wide.

Peak season of boat traffic on the waterway is between November and April and special opening regulations have been set up for that period as follows:

Monday – Friday: Bridge opens at 7:00, 7:30, 8:00,8:30 am
B/n 9:00am –4:30 pm, every 20 minutes on the hour
At 4:30 pm, 5:30pm, 6:00pm, 7:00pm
Weekends and Holidays: Bridge opens b/n 8:am –6:00pm, every 20 minutes.

There's a notice posted on the bridge that advises sailors to lower their antennae and out rigs to avoid unnecessary openings. Violators are then warned of a possible imposition of a $1,000 fine. The researchers observed a strict compliance with this order by sailors and the frequency of openings were greatly reduced.

During the period of data collection at the bridge, under clearance for the vessels using the waterway were recorded to be between 21 and 25 feet. This depended on the rise and fall of the tide. The weather forecast for the period of the study indicated unfavorable weather for sailing, resulting in a fewer boats on the water than anticipated for a weekend in that period of the year. The weather turned out to be much warmer than predicted but the previous warning had obviously resulted in a change of plans for the regular weekend sailors.

SR-5 (US-1) is classified as a functional class 14 roadway, a principal arterial and has a high estimated Average Annual Daily Traffic of 24500. Numbers of vehicles that were affected at each bridge opening ranged from 60 at periods of low flows, to over 200 at high flows.

There is an intersection about 600 feet north of the bridge and the traffic signal was observed to be coordinated with the traffic signal at the bridge. Through and southbound vehicles wishing to go over the bridge therefore experience an extended red time when the bridge is open. This results in very few vehicles occupying the roadway section between the bridge and the intersection and the intersection is free from any blockade by stopped vehicles. Vehicles turning into the intersection from the east and west are then coordinated to use the intersection.

Between 1:21 pm and 2: 30 pm on Friday, the researchers observed an irregular happening; on five different occasions the bridge opening signal gates dropped and stopped vehicles for periods ranging between 37 seconds to 3 minutes but the bridge did not open on those occasions. Two vessels were held up all this while. Then at 2:41 pm the gate signals dropped again but this time one leaf on the south side of the bridge was raised. The other leaf on that same south side was raised after about 7 minutes. The two leaves on the north side stayed down, so the vessels held up in the holding area had to use the half-opening on the south side. It took a total of over 15 minutes from the dropping of the gates to get the vessels to pass and to reopen the roadway for vehicles to resume their travel. Flow in the south direction resumed to normal after about 7 minutes but the north bound traffic completely broke down and normal flow did not resume till the next opening about 25 minutes later. This was due to the extended delay period coupled with the presence of the signalized intersection located about 600 ft from the bridge. This caused another extended delay during the next opening because vehicles had backed up from the intersection all the way onto the bridge and beyond. Therefore at the drop of the gates, the bridge tender had to wait for a couple of minutes to allow stopped vehicles on the bridge to be cleared before the bridge was opened.

The summary of recorded data from the boat survey are shown in Figures 3.9, 3.10 and 3.11, along with the effects of feasible replacement alternatives indicated.

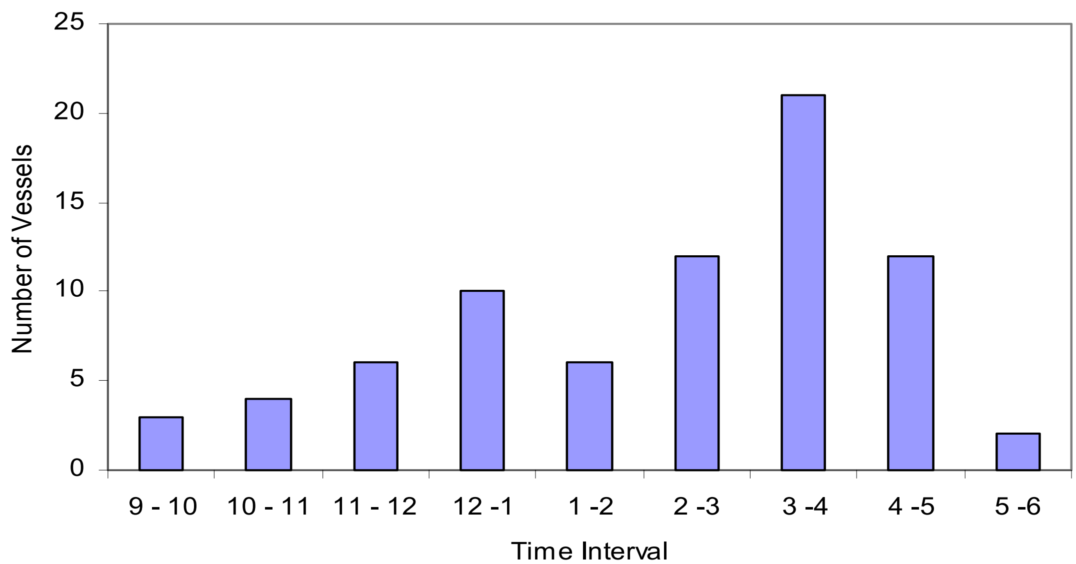


Figure 3.9 Hourly Distribution of all vessels – Bridge 930004

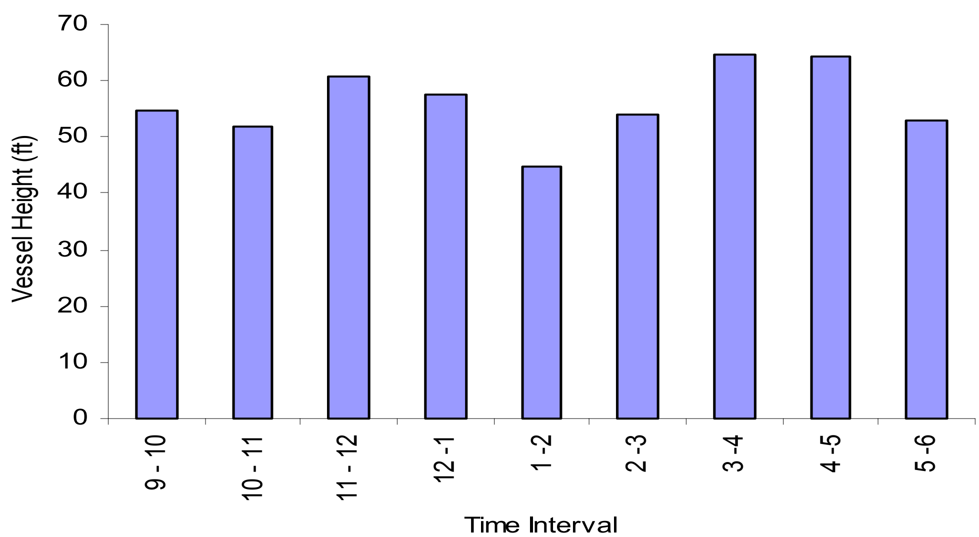


Figure 3.10 Hourly Distribution of Tallest Vessel Heights -Bridge 930004

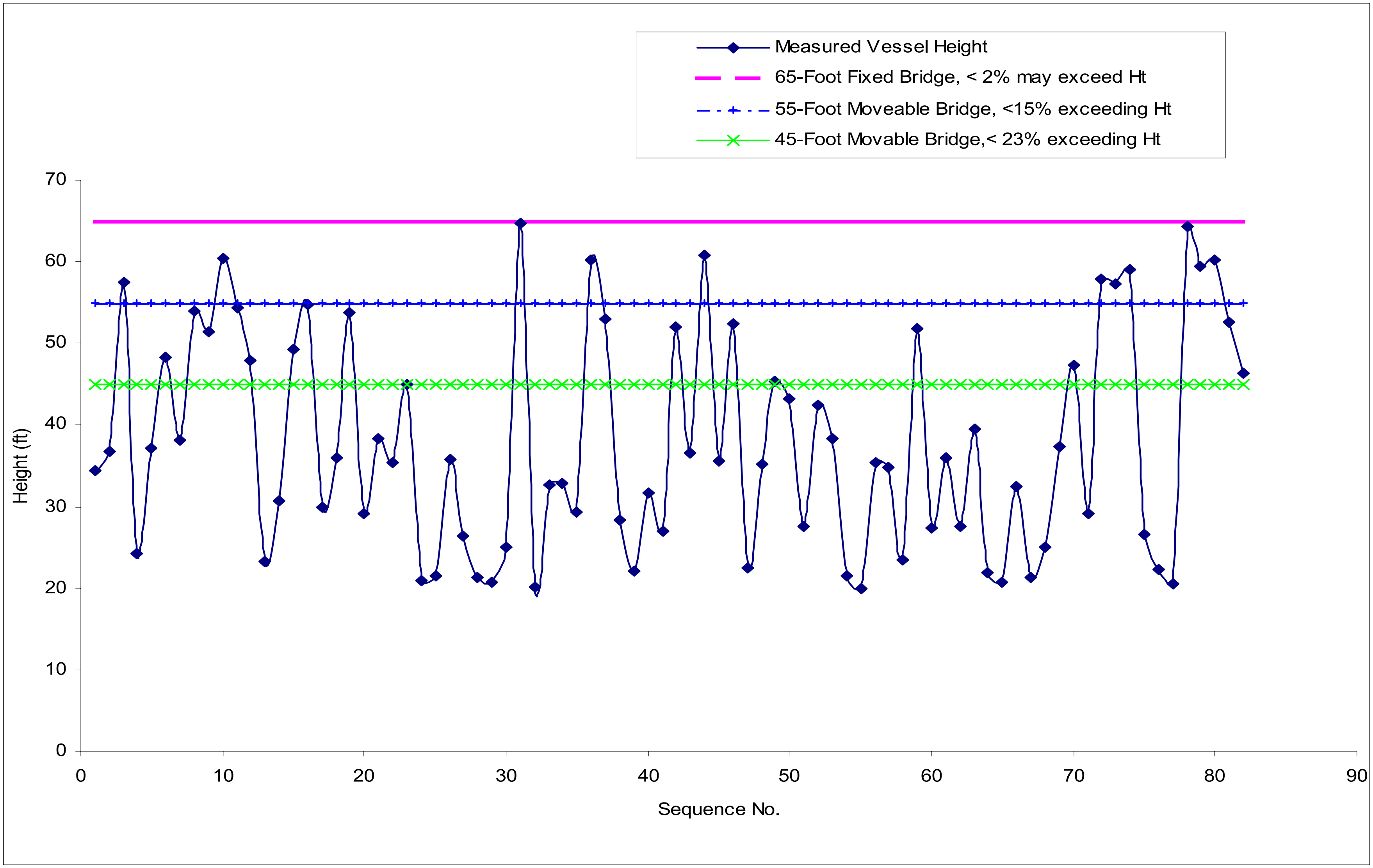


Figure 3.11 Bridge 930004 - Survey Results and Bridge Replacement options

### 3.3.4 Bridge IDs 150027 & 150076 (St. John's Pass) for Study Period 3/28/03 – 3/29/03

Bridge Nos. 150027 &150076, named as the St. John's Pass Bridges are located at Madeira Beach in Pinellas County, FL. They are a set of twin-span bascule bridges that carry the four-lane divided SR 699 across the Gulf Intracoastal Waterway (GIWW), at its outlet into the Gulf of Mexico. The St. John's Pass Bridge is regulated to open on demand at all times in the day. Vessels are therefore not usually held up in the holding area as the bridge is opened on their approach.

During the period of the data collection at the bridge, under clearance for the vessels using the waterway were recorded to be between 23 and 25 feet. This depended on the rise and fall of the tide. The waterway was observed to serve mostly recreational vessels. A lot of small vessels such as personal watercraft, rowing dinghies, canoes, water-ski boats and small fishing boats which needed no opening were observed to use the waterway. Coast Guard and local Sheriff Patrol boats were on regular patrols. The Gulf Boulevard, classified as a functional class 16 roadway, is a coastal route serving several beaches along the Gulf of Mexico in Pinellas County. It has an estimated Average Annual Daily Traffic (AADT) of 10,500 in each direction.

The bridges' opening regulations give no priority to vehicles even at peak hours, resulting in long queues. Queue lengths observed ranged from 1000 feet to over 2000 feet. Vessels randomly arrive and therefore at peak hours of vessel traffic the bridges were observed to stay closed for periods as short as five minutes in between openings. This greatly affects the flow of vehicular traffic and has some vehicles being delayed by two consecutive bridge openings.

There are plans underway to replace the existing bridges, details of which are given below:

- The proposed improvements involve replacing the existing bascule bridges with low-level, twin-span bascule bridges on the same alignment. The new bridges will increase the horizontal navigational clearance from 60 feet to 100 feet in width and will provide a 27-foot vertical clearance over the channel without acquiring additional right-of-way. The profile grade will be 5.6 percent and will meet the requirements of the Americans with Disabilities Act (ADA). The typical section includes two lanes of travel in each direction, 8-foot sidewalks, 10-foot outside shoulders, and 4-foot inside shoulders.
- Projected start date: Fall 2005
- Projected cost: $49.8 million

The summary of recorded data from the boat survey are shown in Figures 3.12, 3.13 and 3.14, along with the effects of feasible replacement alternatives indicated.

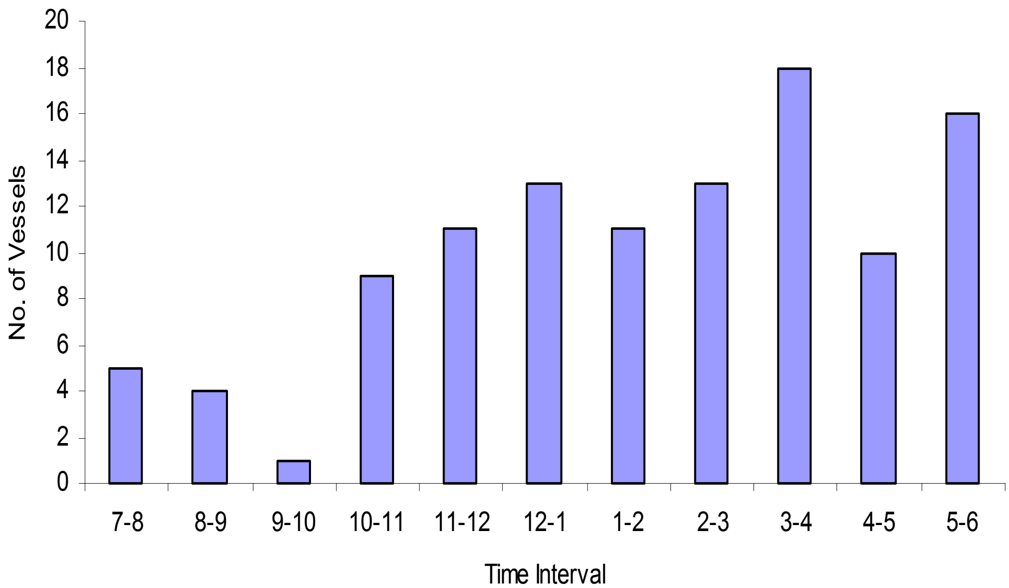


Figure 3.12 Hourly Distribution of all vessels – Bridge IDs 150027&150076

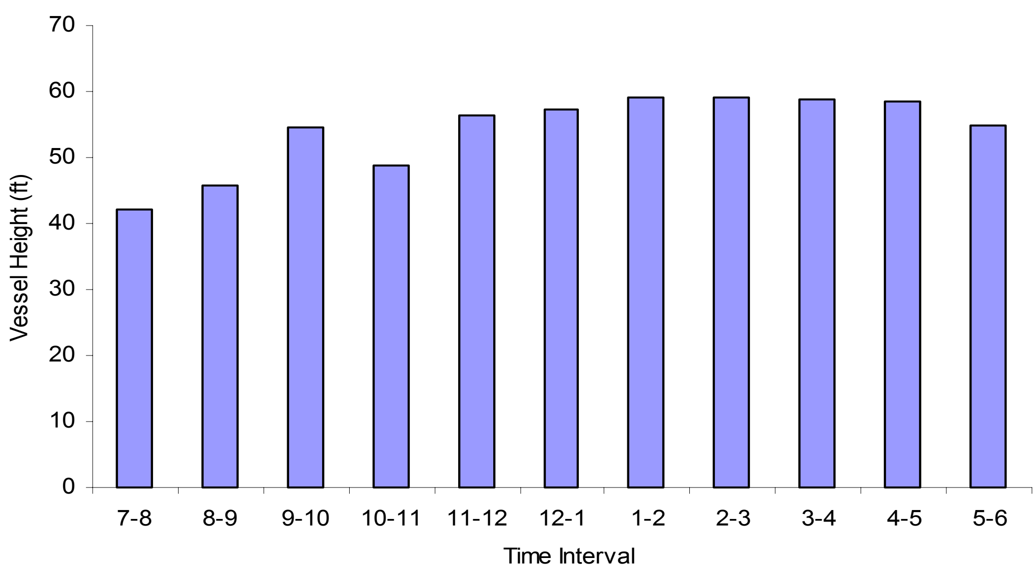


Figure 3.13 Hourly Distribution of Tallest Vessel Heights – Bridge IDs 150027&150076

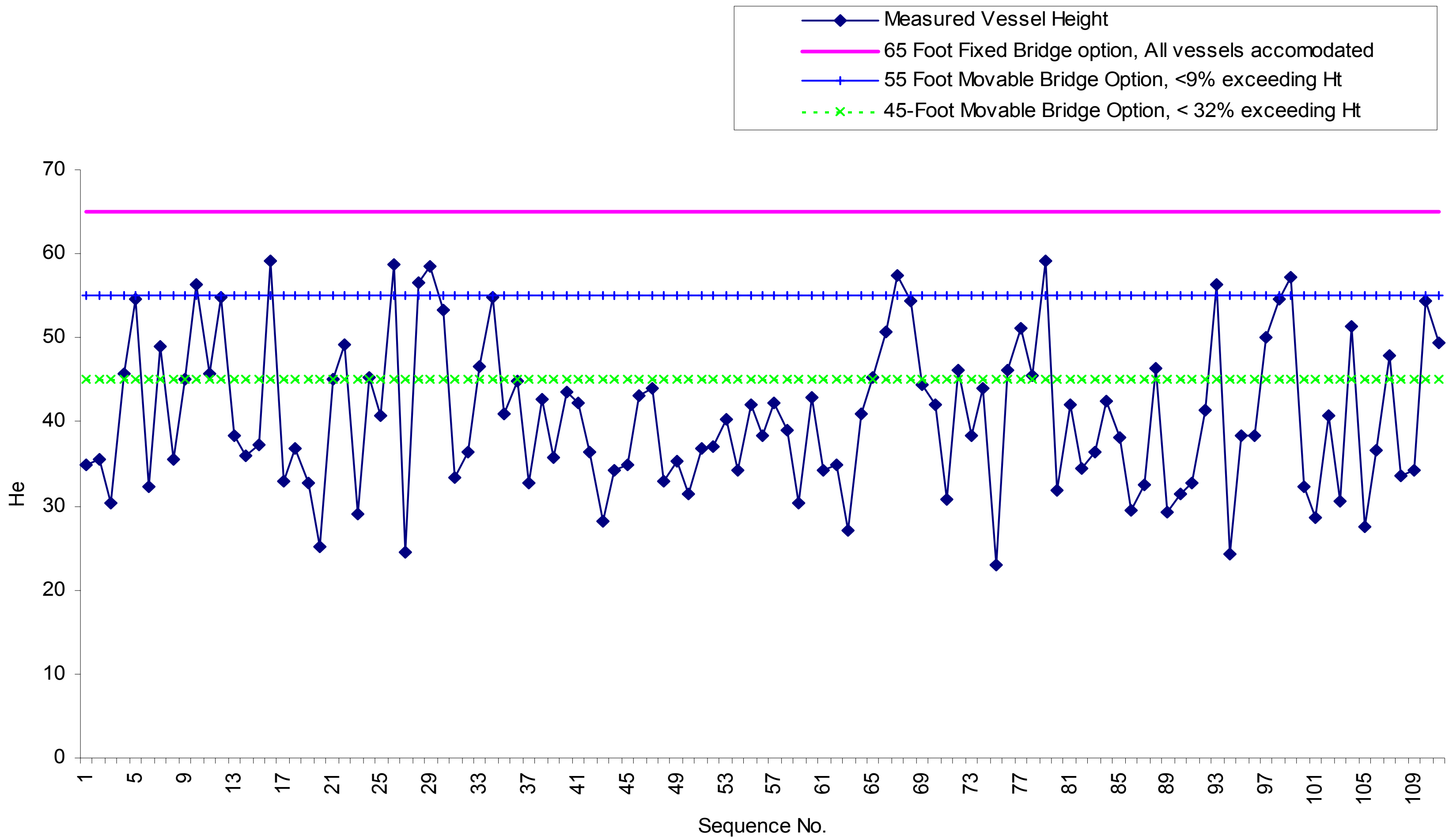


Figure 3.14 Bridge IDs 150027&150076 – Survey Results and Bridge Replacement options

### 3.3.5 Bridge ID 150050 (Pinellas Bay Way) for Study Period: 5/8/03 – 5/11/03

Bridge No. 150050, is a double leaf bascule bridge that carries the two-lane SR 682, locally named as Pinellas Bay Way across the Gulf Intracoastal Waterway at St. Pete Beach in Pinellas County, FL. There is a Toll Plaza located about 550 feet to the west end of the bridge.

The 14th street causeway bridge has the following opening regulations:
7am – 7pm: Bridge opens on the hour, 20 minutes on the hour and 40 minutes on the hour.
It also opens on demand for US Public Vessels, Tugs in Tow and Vessels in Distress.
7pm – 7am: Bridge opens on demand.

During the period of data collection at the bridge, under clearance for the vessels using the waterway was recorded to be between 21 and 22 feet. This depended on the rise and fall of the tide.

The waterway serves both recreational and commercial vessels. A lot of small vessels such as personal watercraft, rowing dinghies, canoes, water-ski boats and small fishing boats which needed no opening were observed to use the waterway. Coast Guard and local Sheriff Patrol boats were on regular patrols. One barge with heavy construction equipment and materials was observed during the survey period. The bridge was opened at its approach and stayed open for a longer period than average due to the slow movement of the barge.

The Pinellas BayWay is classified as a functional class 16 roadway and it has an estimated Average Annual Daily Traffic (AADT) of 15800. Queue lengths observed ranged from 800 feet to over 2000 feet. During peak periods stopped vehicles backed up into the signalized intersection located about 2500 feet to the east of the bridge and affected the operation of the intersection. Some through vehicles on the west side of the intersection sometimes remain stationary through the green phase because there would be no storage space if they had to move through the intersection. Operation at the Toll Plaza was also affected during the extended bridge openings when vehicular traffic backed up to the entrance of the plaza. Vehicles are forced to queue up in the 3 toll lanes, 2 of which were Electronic Toll Collection (ETC) lanes.

There are plans to replace the existing bridge, details of which are given below:

> SR 682 (Bay way) from west of SR 679 to the west toll plaza
> This project will construct a high level bridge (four lanes) to replace the existing two-lane drawbridge over the Intercoastal Waterway.
> Projected start date: Spring 2004
> Projected cost: $37 million

The summary of recorded data from the boat survey are shown in Figures 3.15, 3.16 and 3.17, along with the effects of feasible replacement alternatives indicated.

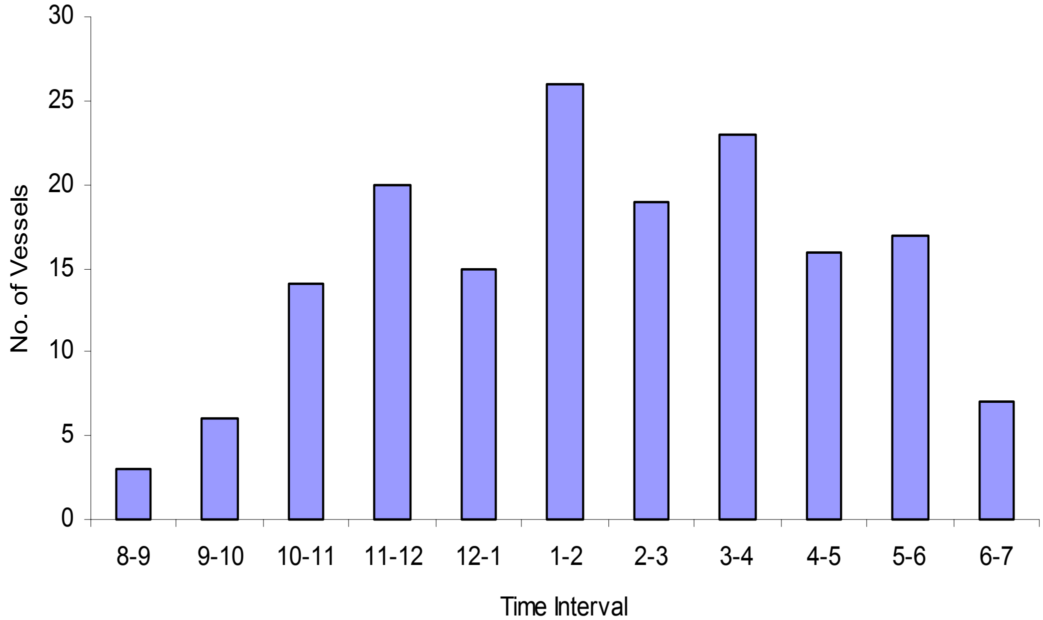


Figure 3.15 Hourly Distribution of all vessels – Bridge ID 150050

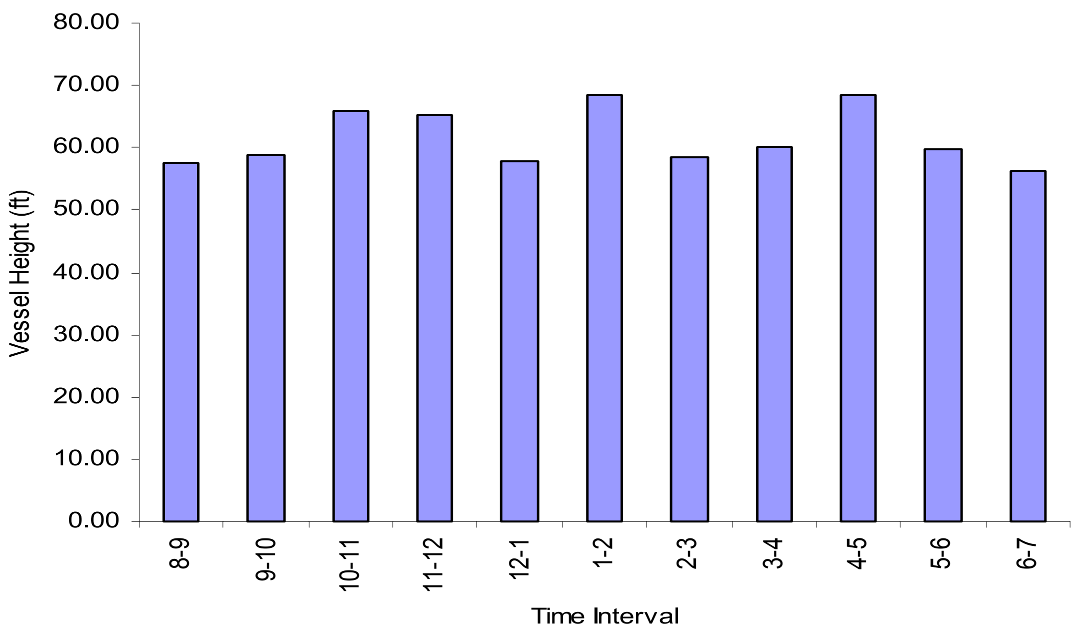


Figure 3.16 Hourly Distribution of Tallest Vessel Heights – Bridge ID 150050

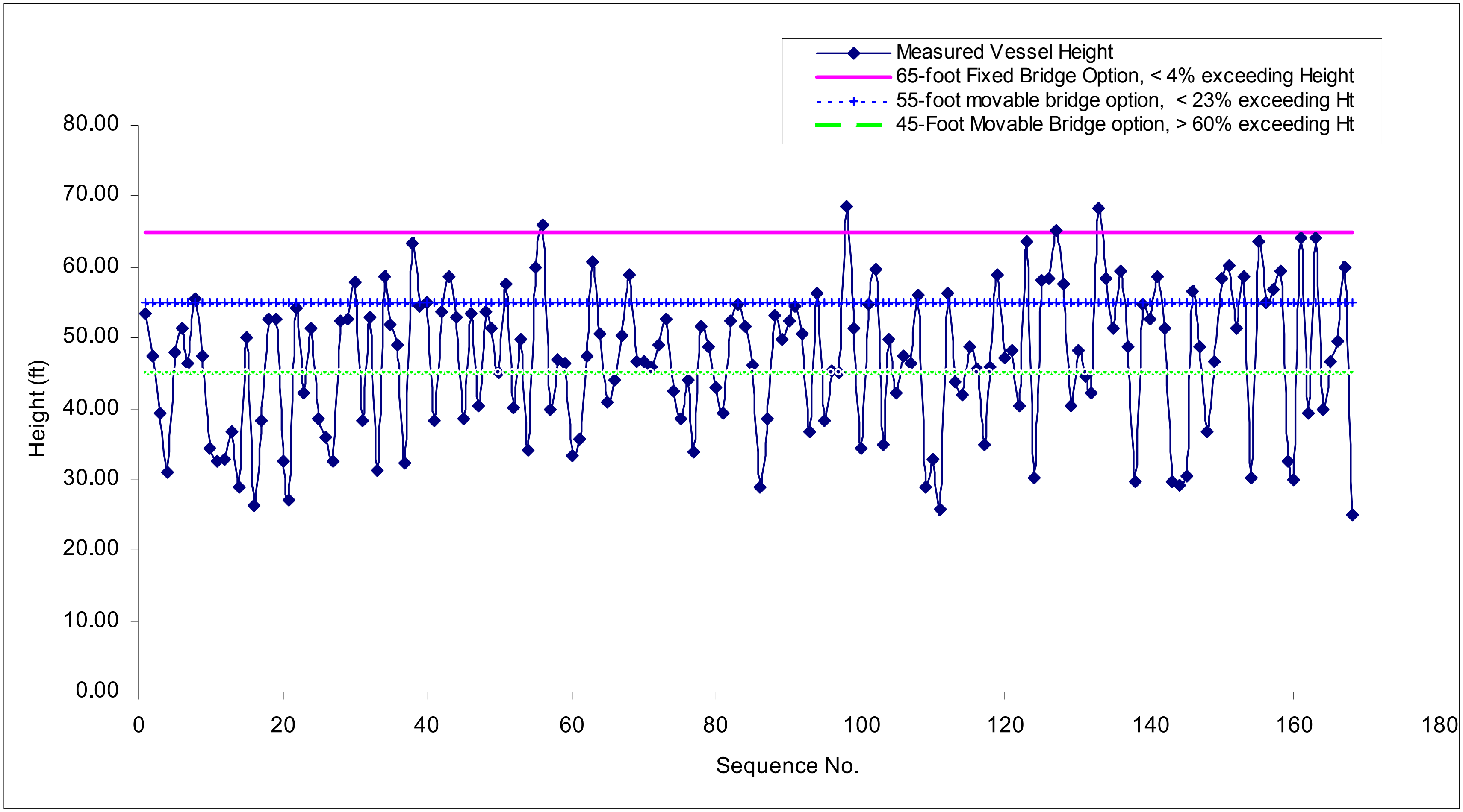


Figure 3.17 Bridge ID 150050 – Survey Results and Bridge Replacement options

Sample photographs from the data collection exercise are presented in the following pages.

Figure 3.18 Vessel Passage and Height Measurement at Bridge 860060

Figure 3.19 Vessel Passage (with Construction Equipment) at Bridge 860060

Figure 3.20 Vessel Passage at Bridge 860060

Figure 3.21 Vessel Passage at Bridge 930004

Figure 3.22 Initial Auto Queue at Bridge 930004

Figure 3.23 Initial Vessel Queue at Bridge 930004


Figure 3.24 Auto Queue at Bridge 780074

Figure 3.25 Auto Queue at Bridge 860060

## 3.4. User Delay Analyses

Vehicular and Vessel queues were analyzed under the existing and proposed replacement options for the current year and the projected year 2020. Queue analyses were carried out for only the moveable bridge options since the fixed bridge options would experience no queues that can be attributed to bridge openings.

### 3.4.1. Vessel Queue Analyses

Analysis of vessel queue was carried out under the existing bridge opening schemes for both weekdays and weekend days at the selected bridges. A simple queuing analysis procedure was used to analyze both vessel and vehicular queues based on a variety of factors such as bridge operating characteristics and vessel and vehicular traffic. The methodology and results of the queuing analysis are discussed below.

Vessel queues were calculated based on present bridge opening schemes for the existing and proposed replacement moveable bridges. It was assumed that all boat or vehicular queues dissipate during every bridge opening and closure. The existing operating scheme is such that the bridge stays open until all boats wishing to pass through do so. Vessel delays were calculated based on the existing opening bridge cycles, the vessel queue, and the service flow rate of vessels.

The average time for servicing of vessels during bridge openings, which is defined in minutes per vessel crossing, was estimated from data obtained during the bridge survey. The average of each day's data was used in the estimation of the service time and is given by the following formula:

$$\textit{Vessel service flow rate} = \frac{\textit{(sum of duration of all bridge openings in a day)}}{\textit{(Number of vessels observed passing per day)}} \qquad (3.6)$$

The duration of each bridge opening is mainly the vehicular traffic "red" signal time, comprised of three elements: the time taken for the mechanical opening and closing of the movable parts of the bridge; the time taken for vessels in the holding area to pass through the bridge when it is opened; and the extra amount of time the roadway is blocked by traffic control devices, traffic signals and drop gates. The total of these time elements is the service time for all vessels passing through the bridge at the opening. The average service time varied for each day and for each bridge site; the values obtained for weekend days were typically lower than for weekdays. These different service times probably result from the lower vessel volumes on weekdays. The same amount of time is needed to raise and lower the bridge regardless of the number of vessels passing underneath. The lower number of weekday vessels allocates this time to fewer vessels thus increasing the average service times.

The lowest value of vessel service rate during the entire study, 1.42 minutes for the weekend and 2.21 minutes for the weekday, were obtained from data collected at Bridge 860060. This can be attributed to the higher volume of vessel traffic and the longer survey period over which the data was collected during the week compared to the weekend period. Since this was from a larger data size, the average service time of 1.42 minutes was therefore selected for use in the vessel delay analyses.

One of the key factors in determining delay to roadway vehicles at a movable bridge is the number of vessels demanding passage through the bridge at each opening: the greater the number of vessels the greater the vehicular delay, all other factors being equal. Data collected during the reconnaissance and actual boat height survey indicated that vessel traffic had a pattern of gradually increasing from morning to a peak at mid-afternoon and early evening and dissipating thereafter. During the survey there were openings for up to 10 queued vessels. However most of the openings were for between 1 to 5 vessels as shown in the graph below. Number of vessels at peak hour openings used in the developed queue model, was estimated from the weekend data as these gave the worst kind of delay scenario resulting from heavier vessel traffic.

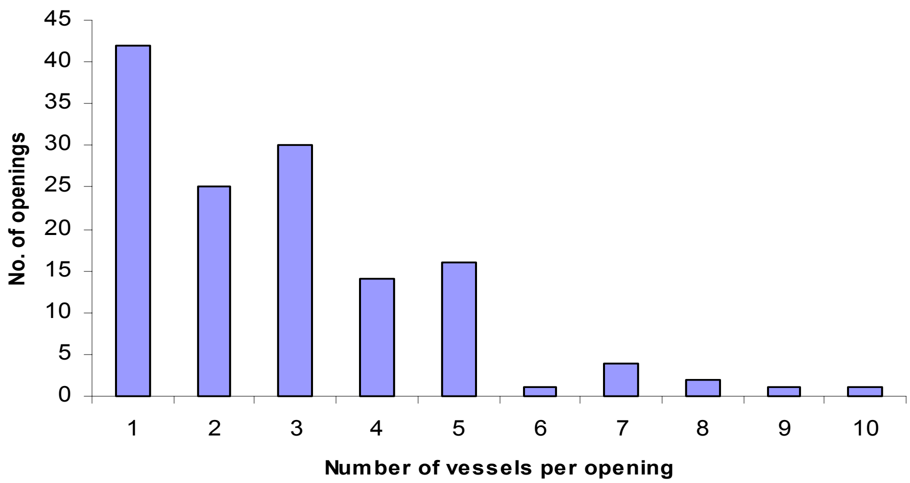


Figure 3.26 Distribution of vessel count per bridge opening at Bridge ID 860060

Analysis of the data obtained from the survey indicates that the duration of bridge opening on most occasions, including other bridge sites, was 6 minutes or less, with 1 to 5 vessels passing through the opening; this is true for about 80 percent of openings for the entire period of the survey. This means that during times of low vessel traffic, such as weekday morning periods, the duration of each bridge opening will be the same for both the existing bridge and any feasible higher moveable bridge replacement option, regardless of the vessel heights. The frequency of bridge openings would however be reduced in the case of the higher-level moveable option, due to reduction in vessels that have to queue.

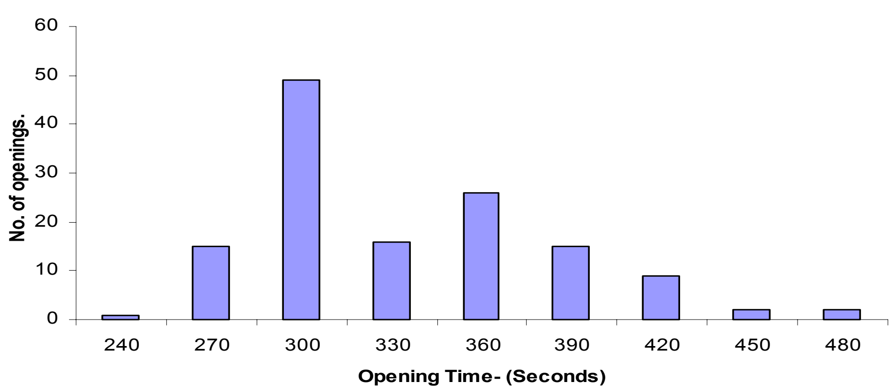


Figure 3.27 Distribution of bridge opening duration at Bridge ID 860060

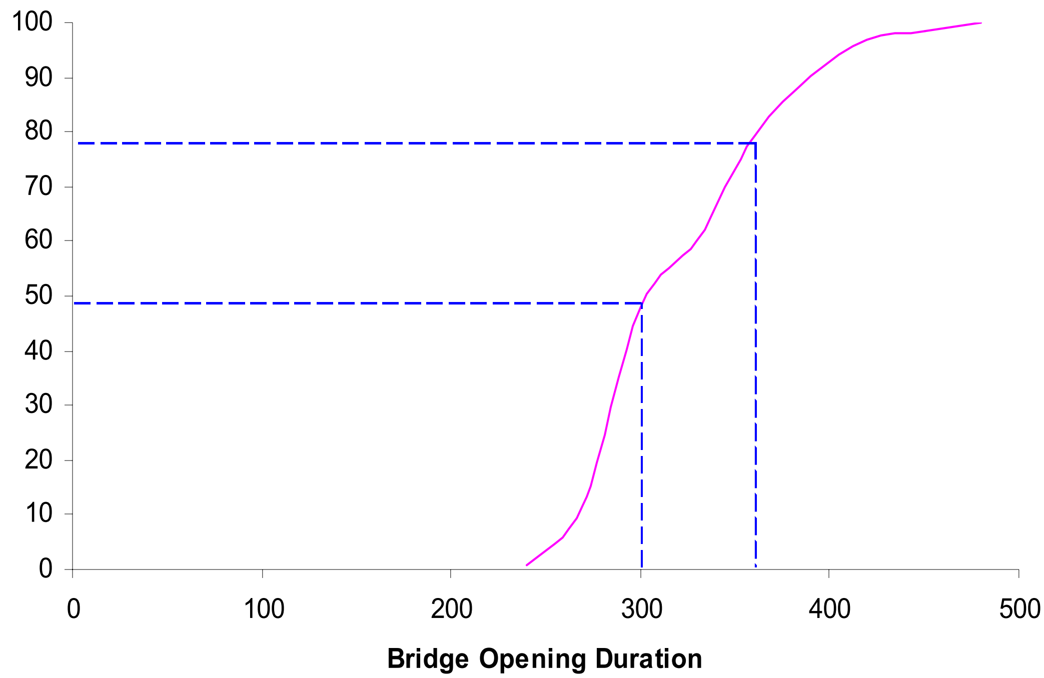


Figure 3.28 Cumulative distribution of bridge opening duration at Bridge ID 860060

The total duration at each bridge opening was formulated in three different ways:

1. Use of only the estimated average service time - Total duration of bridge opening was simply estimated as the product of the number of vessels in queue and an average service time of 1.42 minutes.
2. Use of a combination of the estimated average service time and the default five (5) minutes of minimum bridge opening duration.
3. Use of a formulated power function - This estimates total bridge opening duration based on the number of vessels in queue. The power function was formulated from a distribution of total opening durations obtained during the survey.

The use of only the average service time, for most queue vessels, gave estimated bridge opening durations that did not reflect the actual operating characteristics observed during the survey. This is attributable to the high average service time obtained for the survey period, which is in turn also attributable to the low total number of vessels and openings recorded during the survey period. A previous related study conducted in 1991 gave an average service time of 0.98 minutes from a higher vessel volume and more frequent openings.

One result of using a minimum opening time was that vehicle queues per bridge openings for the existing low-level movable bridge and any proposed high-level movable bridge during any off-peak hour opening were identical even though vessel queues were not. The actual daily total number of bridge openings will however be less for the higher replacement movable bridge. Bridge opening durations obtained from using the power function gave comparatively more realistic values than the other two service time formulations.

Table 3.8 Vessel Service Times– Power Function Model ($y = 265.79x^{-08281}$)

| No. of vessels | Average Vessel Service Times (min) for Bridge IDs | | | | | |
|---|---|---|---|---|---|---|
| | **150027** | **150050** | **860060** | **930004** | **Average** | **Predicted** |
| 1 | 273.290 | 210.600 | 289.320 | 298.430 | 267.910 | 267.780 |
| 2 | 153.478 | 124.152 | 160.266 | 166.254 | 151.038 | 151.063 |
| 3 | 109.514 | 91.138 | 113.442 | 118.073 | 108.042 | 108.075 |
| 4 | 86.192 | 73.189 | 88.778 | 92.620 | 85.195 | 85.219 |
| 5 | 71.582 | 61.740 | 44.481 | 76.721 | 63.631 | 70.876 |
| 6 | 61.502 | 53.727 | 62.840 | 65.778 | 60.962 | 60.968 |
| 7 | 54.096 | 47.770 | 55.104 | 57.754 | 53.681 | 53.680 |
| 8 | 48.405 | 43.146 | 49.177 | 51.598 | 48.082 | 48.075 |
| 9 | 43.885 | 39.441 | 44.481 | 46.716 | 43.630 | 43.618 |
| 10 | 40.200 | 36.396 | 40.661 | 42.741 | 40.000 | 39.983 |

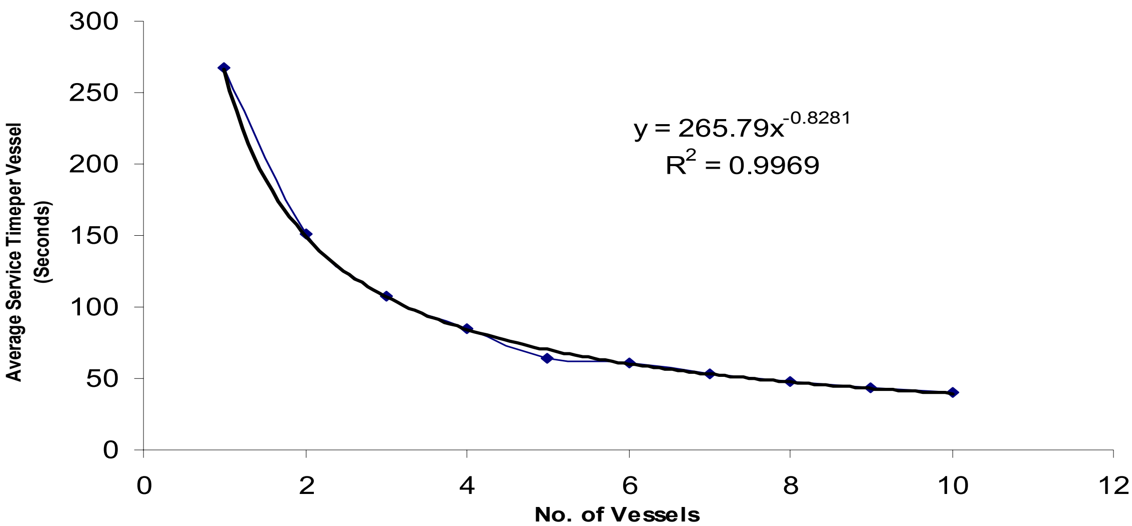


Figure 3. 29 Predicted average service time (Power Function)

Vessel delay is a function of time waiting for the bridge to open and the time spent clearing the queue. Assuming vessels arrive randomly at the bridge, the average vessel delay as a result of the bridge closure would be ½ of the bridge opening cycle length; or 15 minutes in the case of a 30-minute bridge opening cycle. The average vessel delay resulting from the vessel queue clearance for each vessel would be ½ of the duration of the bridge opening minus the time it would take for it to pass under the bridge, which is the estimated average service time. Total vessel delay per bridge opening would therefore be calculated as the sum of the delays from (1) the bridge opening cycle and (2) the queue clearance multiplied by the number of the vessels in the queue.

***Vessel delay per bridge opening cycle =***
***[(½ bridge opening cycle) + (queue clearance delay)] x (number of boats per cycle)*** (3.7)

Where the queue clearance delay = (½ bridge opening time – average service time)

It occurred infrequently that two vessels moving in opposite directions were serviced in about the same period. For simplicity in the analysis however, directional factor was not considered to estimate vessel delays. The service time for the total number of vessels held up on both sides of the bridge is assumed to be the same as would have been for the same total number of vessels held up on only side of the bridge.

There is also the need to project vessel traffic for the future. An analysis of existing data on bridge opening logs, which included data on the number of vessels serviced for each month, was made to determine the growth rate of vessels serviced by various movable bridges. The results of the analysis for the 20- year period, between 1981 and 2001, did not indicate a uniform growth pattern; with some bridges having missing data for some years. A further analysis was therefore carried out by eliminating bridges with incomplete data for any of the years over the narrowed down period of 1991-2001. This resulted in the elimination of all but twenty (20) bridges, which had complete data for the period. All of the 20 bridges were noted to be in district four (4), which had the highest number of movable bridges in the state. Analysis was then made based on the projected linear growth rate formula:

$$Y_f = Y_p (1+r)^n \qquad (3.8)$$

Where:
$Y_f$ is the forecasted number of vessels (after n years)
$Y_p$ is the present number of vessels
r is the growth rate expressed as a fraction of 100.
n is the number of years of for which growth is projected

The above formula can be expressed as a linear equation by the introduction of logarithmic functions:

$$\text{Log}(Y_f) = \text{Log}(Y_p) + n * \text{Log}(1+r) \qquad (3.9)$$

A plot of Log (Number of Vessels) versus year yields a straight-line graph with intercept Log ($Y_p$) and gradient Log (1+r). An antilog of the gradient minus (one) 1, gives the estimated growth rate, expressed in percent. The use of this procedure on the 20 bridges with complete data gave a range of growth rates between –2.5% and +4 %. For the purpose of the study however a positive growth rate would be assumed for all bridge sites. A previously related study (Delghani et al. 1993) carried out in 1991, also in FDOT District 4, reported estimated vessel growth rates of a low 1% and high 3%, based, among other things, on the feedback obtained from the interaction with professionals in the Marine industry. From the existing data and from observations made during data collection survey for this study the range 1% - 3%, was estimated to be representative of the growth of vessel traffic.

### 3.4.2. Vehicular Queue Analyses

Microsoft Excel spreadsheet models were developed to conduct a delay and queue analysis for the bascule bridge openings. The model is based on the bottleneck concept and the key input factors are the average annual daily Traffic (AADT), the Directional Design Hourly Volume (DDHV) and the average service time per vessel.

The Directional Design Hourly Volume (DDHV) used for the analysis was estimated from the peak hour factor, K and directional distribution factor, D, factors given in the FTI annual average daily traffic (AADT) database. The values used for the bridge sites selected for data collection are given in the table below.

Table 3.9 Vehicular Traffic Characteristics for AADT

| Bridge No. | Peak Hour factor, K | Directional Distribution Factor, D |
|---|---|---|
| 150027 | 9.88% | 59.18% |
| 150076 | 9.88% | 59.18% |
| 150050 | 9.88% | 59.18% |
| 780074 | 9.35% | 58.25% |
| 860060 | 9.39% | 56.32% |
| 930004 | 10.19% | 58.4% |

The future year of 2020 was used as the year of planning for the analysis. This was to enable forecasts to be comparable to those in FDOT's Year 2020 Transportation Plans. The annual vehicular traffic growth rate applied to the roadway carried by each bridge was developed from the existing AADT and the 2020 projected volume in the National Bridge Inventory (NBI) database using the projected growth rate formula below:

$$V_f = V_p (1+r)^n \tag{3.10}$$

where:
$V_f$ is the forecasted traffic volume (after n years)
$V_p$ is the present number of vessels
r is the growth rate expressed as a decimal fraction.
n is the number of years of for which growth is projected

Table 3.10 Projected Vehicular Traffic 2002 – 2020

| Bridge No. | Existing Year -2002 | | Projected Year - 2020 | |
|---|---|---|---|---|
| | Average Annual Daily Traffic (AADT) | Directional Design Hourly Volume (DDHV) | Average Annual Daily Traffic (AADT) | Directional Design Hourly Volume (DDHV) |
| 150027 | 21000 | 1228 | 32984 | 1929 |
| 150076 | 21000 | 1228 | 32984 | 1929 |
| 150050 | 15800 | 924 | 24817 | 1452 |
| 780074 | 22000 | 1199 | 34555 | 1882 |
| 860060 | 15100 | 799 | 23717 | 1253 |
| 930004 | 25000 | 1488 | 39267 | 2337 |

The typical spreadsheet model was developed to first estimate the bridge opening period based on the number of vessels passing through the bridge at each opening. Vehicle queues are then calculated based on the estimated length of bridge opening.

### 3.4.3. Estimating Vehicular Delays per Bridge Opening

Average vehicle delays were estimated per bridge opening, along with vehicle delay for the bridge opening taking place during the peak hour for vehicular traffic, for the different bridge options, based on the length of the bridge opening and the time it takes for the vehicle queue to dissipate. Total daily vehicle delays were calculated using the total number of openings, and also using the peak hour factor. The former was labeled the "Average Method" while the latter is referred to as the "Peak Hour Method." The portion of vehicle delay associated with the vehicle queue clearance was calculated based on the length of the vehicle queue and the capacity of the roadway using the Adolf May bottleneck model described in section 3.2.1.

The total number of vehicles affected by the opening includes vehicles that arrive when the bridge is open and are therefore forced to queue on the approach roads to the bridge, and vehicles that arrive when the bridge is closed but are forced to reduce their speed due to the clearance of the initial built up queue. The period of delay for each vehicle varies from a minimum of zero, for the last vehicle to arrive before normalization of flow, to a maximum of the total duration of the queue, for the first vehicle to be stopped when the traffic control gates drop. Due to the random arrival of vehicles, an average individual delay period for each affected vehicle is estimated be half of the bridge opening time, r.

The saturation flow rate s is estimated from the capacity of uninterrupted flow developed from tables for the Generalized Level of Service and the number of lanes in each direction of travel over the bridge.

A sample calculation to estimate vessel and vehicular queues and delays is presented as follows. The example shows the steps that were taken to calculate both vessel and vehicle queues for a 15-minute bridge opening cycle.

Vessel arrival rate is estimated from the total hourly volume. Vessels are assumed arrive at a uniform rate. For example, if vessel traffic during the peak hour was 45 vessels per hour, then the arrival rate will be given by 45/60 = 0.75 vessels per minute arriving at the bridge.

For a 15 minute cycle we have, 0.75 x 15mins = 11.25 ≈ 11 vessels in queue.

For illustration purposes, let us assume a vessel service flow rate = 0.98 minutes per vessel, as indicated in Delghani et al. (1993). For an 11 vessel queue therefore, the opening duration = 0.98 x 11 = 10.8 minutes. The arrival rate of vehicles is estimated from a traffic count conducted at the bridge site or from existing traffic count data obtained from the Florida Traffic Information. For a peak hour traffic count of 2190 vehicles per hour, the vehicles are assumed to also arrive at a uniform rate and the arrival rate is estimated as = 2190/60 or 36.5 vehicles per minute.

Therefore the vehicle queue length = 36.5 veh/min x 10.8 mins = 352 vehicles, or 176 vehicles per lane for a 2-lane roadway.
Vessel delay per bridge opening cycle = (bridge opening cycle) x (Number of boats per cycle) + (queue clearance delay) x (number of boats per cycle)=

= 7.5 min x 11 boats + [5.4 – 0.98] x 11 boats

= 131 boat minutes.

Note that (a) this is the average bridge opening cycle delay and (b) this is the average queue clearance delay.

From the Adolf May bottleneck model,

q, average arrival rate of vehicle traffic = 36.5 veh / min

s, saturation flow rate = 1850 x 2 lanes / 60
= 61.7 veh / min

$s_r$, flow rate at bottleneck during blockade = 0

r, duration of blockade = 11vessels x 0.98 vessels / min
= 10.8 min

$t_q$ = total elapsed time from when start of the blockade (bridge opening) until free flow resumes.
= 7.84 x (61.7 – 0) / (61.7 – 36.5)
= 19.2 mins

$t_o$ = time for the queue to dissipate after the blockade is removed in minutes

=19.2 - 7.84 = 11.36 minutes.

Average vehicle delay in minutes,

d = 7.84 / 2 = 3.92 minutes

Total number of vehicles affected,

N = 36.5 x 19.2 = 701 vehicles

Total vehicle delay,

D = 7.84 x 701 / 2 = 2748 vehicle minutes

### 3.4.4 Estimating Total Vehicular Average Daily Delay

As mentioned earlier, two methods were used to estimate the total daily vehicle delay: (1) average method and (2) peak hour method. For the peak hour method the vessel queue at the vehicular peak hour was determined and used in estimating peak hour delay on vehicles. The result is converted into an average daily delay using the roadway's vehicular traffic peak hour factor. The peak hour method estimates the worst delay scenario i.e. the delay at peak hour and converts it into total delay by the use of the peak hour factor.

In the average method, average hourly vessel queue is used to determine delay on the average hourly vehicular traffic volume. The total average daily delay is obtained as a product of the estimated average hourly delay and the average number of daily openings.

Most of the movable bridges are manned 24 hours daily but observations made during the survey period and from interaction with bridge tenders and locals indicated that most of the bridge openings, over 90%, occur during the daylight period of between 7am and 7pm. Vehicles arriving at the bridge during daylight will therefore be most affected by the bridge openings. An analysis effort was therefore undertaken to develop a diurnal distribution of roadway vehicles using existing data of hourly volumes from various traffic monitoring sites in the State of Florida.

The percentage of vehicular traffic volume between 7 a.m. to 7 p.m. was calculated for each of the selected sites. It must be noted that movable bridges did not necessarily carry the routes for which the traffic volumes were obtained for the analysis of the diurnal distribution. They were however randomly selected and in a way to reflect a fair geographical distribution for the state of Florida. The results indicated that a range of between 70 and 80 percent of the total daily vehicular traffic were recorded during the daylight period of 7 a.m. to 7 p.m.

For the model therefore an average value of 75% of the AADT was used as the volume of affected vehicles by the bridge openings. The average arrival rate of vehicles is estimated from the proportion of vehicular volume arriving at the bridge over the 12-hour daylight period and is formulated as follows:

$$q = AADT\ 0.75/(12*60) \qquad (3.11)$$

Typical results for two of the selected traffic monitoring sites are shown in the figures below.

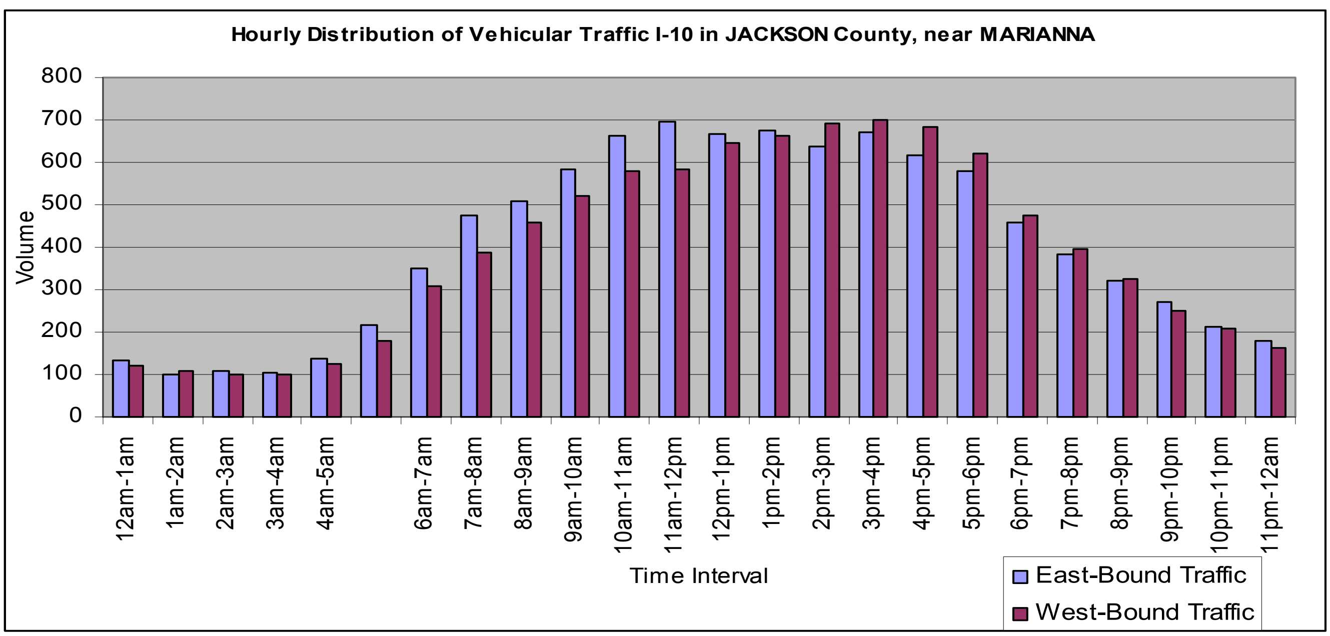


**Figure 3.30. Hourly Distribution of Vehicular Traffic on I-10 near Marianna, Florida**

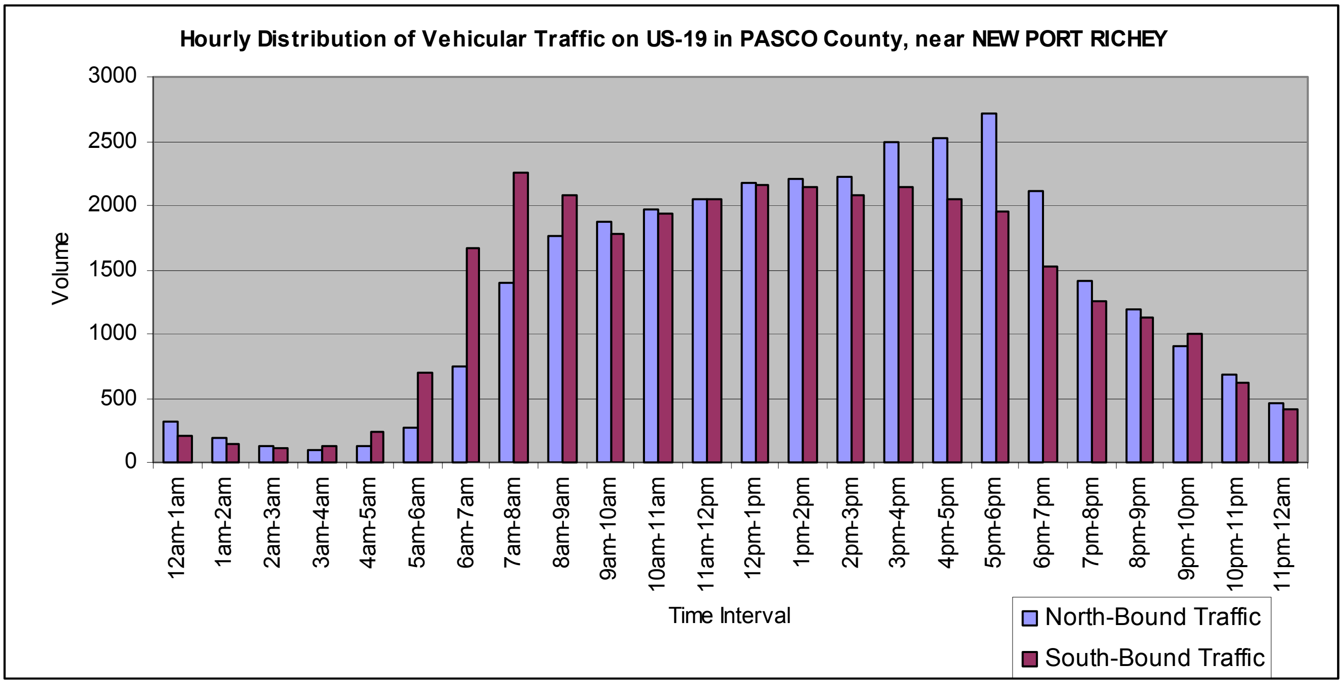


Figure 3.31 Hourly Distribution of Vehicular Traffic on US-19 near Newport Richey, Florida

Based on the methods described above, the results of the delay analyses at the study locations are presented in tables 3.11 to 3.14.

Table 3.11 Results of Vehicular Delay Analyses at Bridge ID 860060 (N.E. 14th Street Causeway)

N.E. 14TH STREET CAUSEWAY BRIDGE
VEHICLE DELAY ANALYSIS (Due to bridge Opening)

| YEAR | 30-minute Bridge Operating Scheme | Vessel Queue | Westbound Average vehicle Delay (minutes) | Eastbound Average vehicle Delay (minutes) | Westbound Total vehicle Delay per cycle (minutes) | Westbound Peak "Hour" delay (minutes) | Westbound Average Daily Delay (hours) | Eastbound Total vehicle Delay per cycle (minutes) | Eastbound Peak "Hour" delay (minutes) | Eastbound Average Daily Delay (hours) | Total Daily Delay (hours) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Existing 16-foot Bridge | | | | | | | | | | | |
| 2003 | Weekday | 5 | 2.5 | 2.5 | **184** | 368 | 56.0 | **136** | 272 | 41.4 | 97.4 |
| 2003 | Weekend | 9 | 3.5 | 3.5 | **416** | 832 | 119.7 | **304** | 608 | 87.5 | 207.2 |
| 2020 | Weekday | 9 | 3.5 | 3.5 | **739** | 1478 | 212.7 | **516** | 1032 | 148.5 | 361.2 |
| 2020 | Weekend | 15 | 5 | 5 | **1508** | 3016 | 401.5 | **1054** | 2108 | 280.6 | 682.1 |
| 55-foot Movable Option | | | | | | | | | | | |
| 2020 | Weekday | 3 | 2.5 | 2.5 | **377** | 754 | 114.7 | **263** | 526 | 80.0 | 194.7 |
| 2020 | Weekend | 5 | 2.5 | 2.5 | **377** | 754 | 114.7 | **263** | 526 | 80.0 | 194.7 |

# Table 3.12 Results of Vehicular Delay Analyses at Bridge ID 930004 (Parker)

## PARKER BRIDGE

VEHICLE DELAY ANAYSIS (Due to bridge Opening)

| YEAR | 30 / 20 -minute Bridge Operating Scheme | Vessel Queue | Northbound Average vehicle Delay (minutes) | Southbound Average vehicle Delay (minutes) | Northbound Total vehicle Delay per cycle (minutes) | Northbound Peak "Hour" delay (minutes) | Northbound Average Daily Delay (hours) | Southbound Total vehicle Delay per cycle (minutes) | Southbound Peak "Hour" delay (minutes) | Southbound Average Daily Delay (hours) | Total Daily Delay (hours) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Existing 16-foot Bridge | | | | | | | | | | | |
| 2003 | Weekday (30min) | 3 | 2.5 | 2.5 | **518** | 1036 | 145.2 | **309** | 618 | 86.6 | 231.9 |
| 2003 | Weekend (20 min) | 3 | 2.5 | 2.5 | **518** | 1554 | 203.3 | **309** | 927 | 121.3 | 324.6 |
| 2020 | Weekday (30min) | 5 | 2.5 | 2.5 | **1215** | 2430 | 340.7 | **595** | 1190 | 166.8 | 507.5 |
| 2020 | Weekend (20 min) | 5 | 2.5 | 2.5 | **1215** | 3645 | 476.9 | **595** | 1785 | 233.6 | 710.5 |
| 55-foot Movable Option | | | | | | | | | | | |
| 2020 | Weekday (30min) | 4 | 2.5 | 2.5 | **2546** | 5092 | 713.9 | **930** | 1860 | 260.8 | 974.6 |
| 2020 | Weekend (20 min) | 4 | 2.5 | 2.5 | **2546** | 7638 | 999.4 | **930** | 2790 | 365.1 | 1364.5 |

Table 3.13 Results of Vehicular Delay Analyses at Bridge IDs 150027 & 150076 (St. John's Pass)

JOHN'S PASS BRIDGE

VEHICLE DELAY ANAYSIS (Due to bridge Opening)

| YEAR | 20-minute(Weekday) 15-minute(Weekend) Bridge Operating Scheme | Vessel Queue | Northbound Average vehicle Delay (minutes) | Southbound Average vehicle Delay (minutes) | Northbound Total vehicle Delay per cycle (minutes) | Northbound Peak "Hour" delay (minutes) | Northbound Average Daily Delay (hours) | Southbound Total vehicle Delay per cycle (minutes) | Southbound Peak "Hour" delay (minutes) | Southbound Average Daily Delay (hours) | Total Daily Delay (hours) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Existing 21-foot Bridge | | | | | | | | | | | |
| 2003 | Weekday | 3 | 2.5 | 2.5 | **383** | 1149 | 155.1 | **229** | 687 | 92.7 | 247.8 |
| 2003 | Weekend | 5 | 2.5 | 2.5 | **383** | 1532 | 193.8 | **229** | 687 | 86.9 | 280.7 |
| 2020 | Weekday | 3 | 2.5 | 2.5 | **827** | 2481 | 334.8 | **428** | 1284 | 173.3 | 508.1 |
| 2020 | Weekend | 6 | 2.82 | 2.82 | **1047** | 4188 | 513.4 | **524** | 1572 | 192.7 | 706.1 |
| 55-foot Movable Option | | | | | | | | | | | |
| 2020 | Weekday | 2 | 2.5 | 2.5 | **827** | 2481 | 334.8 | **428** | 1284 | 173.3 | 508.1 |
| 2020 | Weekend | 2 | 2.5 | 2.5 | **827** | 3308 | 418.5 | **428** | 1284 | 162.4 | 581.0 |

Table 3.14 Results of Vehicular Delay Analyses at Bridge ID 150050 (Pinellas Bay Way)

PINELLAS BAYWAY BRIDGE

VEHICLE DELAY ANAYSIS (Due to bridge Opening)

| YEAR | 20-minute Bridge Operating Scheme | Vessel Queue | Westbound Average vehicle Delay (minutes) | Eastbound Average vehicle Delay (minutes) | Westbound Total vehicle Delay per cycle (minutes) | Westbound Peak "Hour" delay (minutes) | Westbound Average Daily Delay (hours) | Eastbound Total vehicle Delay per cycle (minutes) | Eastbound Peak "Hour" delay (minutes) | Eastbound Average Daily Delay (hours) | Total Daily Delay (hours) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Existing 24-foot Bridge | | | | | | | | | | | |
| 2003 | Weekday | 3 | 2.5 | 2.5 | **384** | 1152 | 175.3 | **203** | 609 | 92.7 | 267.9 |
| 2003 | Weekend | 6 | 2.75 | 2.75 | **465** | 1395 | 198.1 | **245** | 735 | 104.4 | 302.4 |
| 2020 | Weekday | 5 | 2.5 | 2.5 | **1219** | 3657 | 556.4 | **424** | 1272 | 193.5 | 749.9 |
| 2020 | Weekend | 10 | 3.75 | 3.75 | **2744** | 8232 | 1168.9 | **955** | 2865 | 406.8 | 1575.7 |
| 55-foot Movable Option | | | | | | | | | | | |
| 2020 | Weekday | 3 | 2.5 | 2.5 | **1219** | 3657 | 556.4 | **424** | 1272 | 193.5 | 749.9 |
| 2020 | Weekend | 5 | 2.5 | 2.5 | **1219** | 3657 | 556.4 | **424** | 1272 | 193.5 | 749.9 |

Analysis of the existing and forecast vehicle and vessel traffic was conducted under the existing movable bridge conditions and the higher movable replacement options. A statistical analysis of the bridge log (vessel) data and Delgani et al. (1993) suggested using the highgrowth rate of 3% to project vessel traffic to year 2020 and to conduct queuing and delay analysis on the waterway. Some comments are provided as follows.

**Bridge ID 860060 (N.E. 14th Street Causeway):** The tallest height recorded during the survey was just less than 90 feet. Ideally, a 90 ft fixed bridge would allow all vessels to pass. However, based on physical and geometric limitations the tallest possible replacement option for the N.E. $14^{th}$ Causeway Bridge would be a 65-foot fixed bridge; This option would theoretically accommodate more than 97.5% of the vessels passing under it (Figure 3.8), ignoring any buffer between vessel and bridge underside. A 55-foot moveable option would also accommodate 90% of the vessels.

Findings of the queuing analysis due to the bridge opening are summarized below for both the no-build option (i.e. the existing moveable bridge) and the build option (i.e. the 55-foot replacement moveable bridge). The fixed bridge option would require no queue analysis for the vehicular traffic.

<u>No-Build Condition</u> **-** Estimated total delay to vehicular traffic at each bridge opening are expected to increase by over 300 percent and 250 percent by the year 2020 for the weekday and weekend operations, respectively, if the existing bridge is not replaced. Both Weekday and weekend delays to vessels are also expected to increase by about 100 percent by 2020.

<u>Build Condition (55-foot moveable bridge)</u> **-**Estimated total delay to vehicular traffic at each bridge opening are expected to increase by over 100 percent by the year 2020, under the 55-foot replacement option moveable bridge, primarily due to the increase in total vehicular traffic demand on the roadway. Weekend vehicular traffic delay will however be expected to decrease by about 10 percent. The frequency of weekend bridge openings is also expected to reduce; hence the total weekend day delay will be greatly reduced.

**Bridge ID 930004 (Parker):** The tallest vessel height recorded during the survey was just less than 65 feet, indicating that bridge can accommodate 90% of al vessels. The bridge height of 65 feet was also estimated to be the geometrically tallest possible option considering the physical limitations beyond and around the bridge location. A fixed bridge of this height would completely eliminate all delays to both vehicular and vessel traffic.
Findings of the queuing analysis due to the bridge opening are summarized below for the no-build option (i.e. the existing moveable bridge) and a 55-foot replacement moveable bridge build option. The third option of a 65-foot fixed bridge would need no queuing analysis, as it would eliminate all queues.

<u>No-Build Condition</u> - Estimated total daily delays to vehicular traffic, due to the bridge openings, are expected to increase by between 200-300 percent and between 600-700 percent by the year 2020 for the weekday and weekend operations, respectively, if the existing bridge is not replaced. This will primarily be due to the increase in vehicular traffic demand on the roadway.

<u>Build Condition (55-foot moveable bridge)</u> **-** Estimated total daily delays to weekday vehicular traffic, due to the bridge openings, are expected to increase by over 200 percent by the year 2020, under the 55-foot replacement option moveable bridge, primarily due to the increase in vehicular traffic demand on the roadway. Weekend delays to vehicular traffic will however be expected to

increase by only about 20 percent. Weekday delays to vessels are expected to remain constant over time, in spite of the increased bridge under-clearance, primarily due to increased volume of vessel traffic. Weekend delays are however expected to decrease under the 55-foot replacement option.

**Bridge IDs150027 & 150076 (St. John's Pass):** The tallest vessel height recorded during the survey was just less than 60 feet. Providing a 5-foot buffer would make a 65-foot fixed bridge the most economically justified fixed bridge replacement option, based on the survey data obtained. A bridge height of over 65 feet is geometrically feasible but will require more right-of-way acquisition since the bridge is located in a heavily developed area.

Findings of the queuing analysis due to the bridge opening are summarized below for the no-build option (i.e. the existing moveable bridge) and a 55-foot replacement moveable bridge build option.

No-Build Condition **-** Estimated total delay to weekday vehicular traffic at each bridge opening is expected to increase by over 100 percent and about 200 percent by the year 2020 for the weekday and weekend operations, respectively, if the existing bridge is not replaced. This primarily will be due to the increase in vehicular traffic demand on the roadway and also due to the increase in vessel traffic on the waterway.

Build Condition (55-foot moveable bridge) **-**Estimated vehicular delay to weekday vehicular traffic at each bridge opening is expected to at worst remain the same by the year 2020, under the 55-foot replacement option moveable bridge. The reason for this is that even though the number of vessels requiring bridge opening will be greatly reduced under the higher replacement movable bridge, a minimum amount of time is required for the mechanical operation of the bridge irrespective of the number of vessels in the holding area. On the average however the total weekday delays would be greatly reduced due to the reduced frequency in bridge openings.

Estimated vehicular delay to weekend vehicular traffic at each bridge opening is expected to decrease by about 20 percent even with the projected increased vehicular traffic volume. The frequency of weekend bridge openings is also expected to reduce; hence the total weekend day delay will be greatly reduced.

**Bridge ID 150050 (Pinellas Bay Way):** The tallest vessel height recorded during the survey was just less than 70 feet.. A fixed bridge height of 65 feet would accommodate 96% of vessels passing under the bridge. The bridge height is also geometrically feasible considering the physical limitations beyond and around the bridge location. This option would completely eliminate all delays to vehicular and nearly vessel traffic. The number of lanes on the replacement bridge, either a movable or fixed bridge, should be increased to 4 lanes. This would provide a better level of service and would also adequately accommodate future traffic volumes.

Findings of the queuing analysis due to the bridge opening are summarized below for the no-build option (i.e. the existing moveable bridge) and a 55-foot replacement moveable bridge build option. The third option of a 75-foot fixed bridge would need no queuing analysis, as it would eliminate all queues.

No-Build Condition **-** Estimated total delay to weekday vehicular traffic at each bridge opening is expected to increase by over 200 percent and about 500 percent by the year 2020 for the weekday and weekend operations, respectively, if the existing bridge is not replaced. This primarily will be due to the increase in vehicular traffic demand on the roadway and also due to the increase in vessel traffic on the waterway.

Build Condition (55-foot moveable bridge) **-** Estimated vehicular delay to weekday vehicular traffic at each bridge opening is expected to at worst remain the same by the year 2020, under the 55-foot replacement option moveable bridge. The reason for this is that even though the number of vessels requiring bridge opening will be greatly reduced under the higher replacement movable bridge, a minimum amount of time is required for the mechanical operation of the bridge irrespective of the number of vessels in the holding area. On the average however the total weekday delays would be greatly reduced due to the reduced frequency in bridge openings.

Estimated vehicular delay to weekend vehicular traffic at each bridge opening is expected to decrease by about 100 percent even with the projected increased vehicular traffic volume. This will be due to the smaller number of vessels for which an opening will be required and therefore a shorter period of opening. The frequency of weekend bridge openings is also expected to reduce; hence the total weekend day delay will be greatly reduced.

## 3.5. User Cost Model Formulation

Benefits of functional improvements in Pontis are assessed in terms of user cost savings. A movable bridge when open for vessels whose heights exceed the navigable vertical clearance under the bridge subjects road users to travel time delay. To evaluate the functional replacement, which minimizes or corrects the deficiency of low navigable vertical clearance for vessels using the waterway, the user cost model estimates the cost of bridge openings for the existing and any proposed replacement option, the difference of which yields the user cost savings.

Annual User benefit $$B_m = (C_o - C_1) * 365 \tag{3.12}$$

Where: $C_o$ is the Daily Cost of Bridge Openings for Existing Bridge
$C_1$ is the Daily Cost of Bridge Openings for Replacement Bridge

### 3.5.1. Estimate of Individual Bridge User Costs

As mentioned earlier, Microsoft Excel spreadsheets were developed to estimate user costs. The following pages show the results of estimated user costs, using the three methodologies described, at each of the four moveable bridge locations used in the study.

Table 3.15 Peak-Hour Method Analyses for BRIDGE ID 860060 (N.E. 14th Street Causeway)

| ***TRAFFIC CHARACTERISTICS*** | YEAR 2002 | YEAR 2020 |
|---|---|---|
| Base year | 2002 | 2002 |
| Future year | 2002 | 2020 |
| Base yr AADT | 15100 | 15100 |
| Peak Hour Factor -k | 9.39 | 9.39 |
| Directional Factor - D | 56.32 | 56.32 |
| Traffic Growth rate | 0.025 | 0.025 |
| | | |
| ***BRIDGE OPENING INPUT*** | | |
| BRIDGE CYCLE LENGTH (minutes) | 30 | 30 |
| PEAK. HRLY VESSEL TRAFFIC (vess/hr) | 18 | 30 |
| AVG. SERVICE TIME (minutes) | 1.42 | 1.42 |
| | | |
| ***ROADWAY CHARACTERISTICS*** | | |
| NUMBER OF LANES ON ROADWAY (ONE DIRECTION) | 2 | 2 |
| SATURATION FLOW RATE (veh/hr/ln) | 1850 | 1850 |
| FLOW RATE AT BOTTLENECK (veh/min) | 0 | 0 |
| | | |
| ***COST INPUT*** | | |
| CPI - BASE YEAR - (1990) | 127.90 | 127.90 |
| CPI - CURRENT YEAR - (2002) | 173.30 | 173.30 |
| VALUE OF TRAVEL TIME IN BASE YEAR ($/Vehicle-Hour) | 9.75 | 9.75 |
| | | |
| **MODEL OUTPUT** | | |
| AADT | 15100 | 23551 |
| PEAK HRLY VEHICLE TRAFFIC (veh/hr)- Dir 1 | 799 | 1245 |
| PEAK HRLY VEHICLE TRAFFIC (veh/hr) -Dir 2 | 619 | 966 |
| VESSEL QUEUE - vessels | 9 | 15 |
| BRIDGE OPENING TIME[a] -minutes | 12.78 | 21.30 |
| VEHICLE QUEUE -Dir 1 - vehicles | 170 | 442 |
| VEHICLE QUEUE -Dir 2 - vehicles | 132 | 343 |
| VESSEL DELAY[b] -Boat minutes | 179.73 | 363.45 |
| Duration of vehicle queue -Dir 1 (minutes) | 16.30 | 32.11 |
| Number of vehicles affected | 217 | 666 |
| Duration of vehicle queue Dir 2 (minutes) | 15.35 | 28.83 |
| Number of vehicles affected | 158 | 464 |
| DELAY PER VEHICLE - Dir 1 per cycle (minutes) | 6.39 | 10.65 |
| DELAY PER VEHICLE - Dir 2 per cycle (minutes) | 6.39 | 10.65 |
| DELAY PER VESSEL (minutes) | 19.97 | 24.23 |
| VEHICLE DELAY -Dir 1 per cycle (vehicle minutes) | 1386 | 7098 |
| VEHICLE DELAY - Dir 2 per cycle (vehicle minutes) | 1012 | 4942 |
| "PEAK HOUR" DELAY-Dir 1 (Vehicle minutes) | 2772 | 14196 |
| "PEAK HOUR" DELAY-Dir 2 (Vehicle minutes) | 2025 | 9885 |
| Daily Average Delay[c] -Dir 1 (Vehicle Hours) | 345 | 1,474 |
| Daily Average Delay -Dir 2 (Vehicle Hours) | 252 | 1,026 |
| Total Daily Average User Delay Cost | $7,888 | $45,997 |
| Annual Average User Delay Cost | $ 2,879,062 | $16,788,988 |

[a]. *A minimum opening time of 5 minutes is set for when 5 or fewer vessels are in queue.*

[b]*Boat delay obtained as a sum of 1/2 bridge cycle period and 1/2 the bridge opening time MINUS the service flow rate per vessel, all multiplied by the number of vessels serviced during the opening.*

[c]*"peak hour" delay factored into daily average delay.*

Table 3.16 Peak-Hour Method Analyses for BRIDGE ID 930004 (Parker)

| ***TRAFFIC CHARACTERISTICS*** | YEAR 2002 | YEAR 2020 |
|---|---|---|
| Base year | 2002 | 2002 |
| Future year | 2012 | 2020 |
| Base yr AADT | 25000 | 25000 |
| Peak Hour Factor -k | 10.19 | 10.19 |
| Directional Factor - D | 58.4 | 58.4 |
| Traffic Growth rate | 0.025 | 0.025 |
| | | |
| ***BRIDGE OPENING INPUT*** | | |
| BRIDGE CYCLE LENGTH (minutes) | 20 | 20 |
| PEAK. HRLY VESSEL TRAFFIC (vess/hr) | 9 | 15 |
| AVG. SERVICE TIME (minutes) | 1.42 | 1.42 |
| | | |
| ***ROADWAY CHARACTERISTICS*** | | |
| NUMBER OF LANES ON ROADWAY (ONE DIRECTION) | 2 | 2 |
| SATURATION FLOW RATE (veh/hr/ln) | 1850 | 1850 |
| FLOW RATE AT BOTTLENECK (veh/min) | 0 | 0 |
| | | |
| | | |
| ***COST INPUT*** | | |
| CPI - BASE YEAR - (1990) | 127.90 | 127.90 |
| CPI - CURRENT YEAR - (2002) | 173.30 | 173.30 |
| VALUE OF TRAVEL TIME IN BASE YEAR ($/Vehicle-Hour) | 9.75 | 9.75 |
| | | |
| **MODEL OUTPUT** | | |
| AADT | 32002 | 38991 |
| PEAK HRLY VEHICLE TRAFFIC (veh/hr)- Dir 1 | 1904 | 2320 |
| PEAK HRLY VEHICLE TRAFFIC (veh/hr) -Dir 2 | 1357 | 1653 |
| VESSEL QUEUE - vessels | 3 | 5 |
| BRIDGE OPENING TIME[a] -minutes | 5.00 | 5.00 |
| VEHICLE QUEUE -Dir 1 - vehicles | 159 | 193 |
| VEHICLE QUEUE -Dir 2 - vehicles | 113 | 138 |
| VESSEL DELAY[b] -Boat minutes | 33.24 | 55.40 |
| Duration of vehicle queue -Dir 1 (minutes) | 10.30 | 13.41 |
| Number of vehicles affected | 327 | 519 |
| Duration of vehicle queue Dir 2 (minutes) | 7.89 | 9.04 |
| Number of vehicles affected | 178 | 249 |
| DELAY PER VEHICLE - Dir 1 per cycle (minutes) | 2.5 | 2.5 |
| DELAY PER VEHICLE - Dir 2 per cycle (minutes) | 2.5 | 2.5 |
| DELAY PER VESSEL (minutes) | 11.08 | 11.08 |
| VEHICLE DELAY -Dir 1 per cycle (vehicle minutes) | 818 | 1296 |
| VEHICLE DELAY - Dir 2 per cycle (vehicle minutes) | 446 | 622 |
| "PEAK HOUR" DELAY-Dir 1 (Vehicle minutes) | 2453 | 3889 |
| "PEAK HOUR" DELAY-Dir 2 (Vehicle minutes) | 1339 | 1867 |
| Daily Average Delay[c] -Dir 1 (Vehicle Hours) | 321 | 509 |
| Daily Average Delay -Dir 2 (Vehicle Hours) | 175 | 244 |
| Total Daily Average User Delay Cost | $7,985 | $13,861 |
| Annual Average User Delay Cost | $ 2,914,386 | $ 5,059,218 |

[a] *A minimum opening time of 5 minutes is set for when 5 or less vessels are in queue.*
[b]*Boat delay obtained as a sum of 1/2 bridge cycle period and 1/2 the bridge opening time MINUS the service flow rate per vessel, all multiplied by the number of vessels serviced during the opening.*
[c]*"peak hour" delay factored into daily average delay.*

Table 3.17 Peak-Hour Method Analyses for BRIDGE ID 150027 (St. John's Pass)

| ***TRAFFIC CHARACTERISTICS*** | YEAR 2002 | YEAR 2020 |
|---|---|---|
| Base year | 2002 | 2002 |
| Future year | 2002 | 2020 |
| Base yr AADT | 21000 | 21000 |
| Peak Hour Factor -k | 9.88 | 9.88 |
| Directional Factor - D | 59.18 | 59.18 |
| Traffic Growth rate | 0.025 | 0.025 |
| | | |
| ***BRIDGE OPENING INPUT*** | | |
| BRIDGE CYCLE LENGTH* (minutes) | 15 | 15 |
| PEAK. HRLY VESSEL TRAFFIC (vess/hr) | 15 | 25 |
| AVG. SERVICE TIME (minutes) | 1.42 | 1.42 |
| | | |
| ***ROADWAY CHARACTERISTICS*** | | |
| NUMBER OF LANES ON ROADWAY (ONE DIRECTION) | 2 | 2 |
| SATURATION FLOW RATE (veh/hr/ln) | 1850 | 1850 |
| FLOW RATE AT BOTTLENECK (veh/min) | 0 | 0 |
| | | |
| ***COST INPUT*** | | |
| CPI - BASE YEAR - (1990) | 127.90 | 127.90 |
| CPI - CURRENT YEAR - (2002) | 173.30 | 173.30 |
| VALUE OF TRAVEL TIME IN BASE YEAR ($/Vehicle-Hour) | 9.75 | 9.75 |
| | | |
| **MODEL OUTPUT** | | |
| AADT | 21000 | 32753 |
| PEAK HRLY VEHICLE TRAFFIC (veh/hr)- Dir 1 | 1228 | 1915 |
| PEAK HRLY VEHICLE TRAFFIC (veh/hr) -Dir 2 | 847 | 1321 |
| VESSEL QUEUE - vessels | 4 | 7 |
| BRIDGE OPENING TIME[a] -minutes | 5.00 | 9.94 |
| VEHICLE QUEUE -Dir 1 - vehicles | 102 | 317 |
| VEHICLE QUEUE -Dir 2 - vehicles | 71 | 219 |
| VESSEL DELAY[b] -Boat minutes | 34.32 | 77.35 |
| Duration of vehicle queue -Dir 1 (minutes) | 7.48 | 20.60 |
| Number of vehicles affected | 153 | 658 |
| Duration of vehicle queue Dir 2 (minutes) | 6.48 | 15.46 |
| Number of vehicles affected | 92 | 340 |
| DELAY PER VEHICLE - Dir 1 per cycle (minutes) | 2.5 | 4.97 |
| DELAY PER VEHICLE - Dir 2 per cycle (minutes) | 2.5 | 4.97 |
| DELAY PER VESSEL (minutes) | 8.58 | 11.05 |
| VEHICLE DELAY -Dir 1 per cycle (vehicle minutes) | 383 | 3269 |
| VEHICLE DELAY - Dir 2 per cycle (vehicle minutes) | 229 | 1691 |
| "PEAK HOUR" DELAY-Dir 1 (Vehicle minutes) | 1531 | 13074 |
| "PEAK HOUR" DELAY-Dir 2 (Vehicle minutes) | 915 | 6766 |
| Daily Average Delay[c] -Dir 1 (Vehicle Hours) | 194 | 1,326 |
| Daily Average Delay -Dir 2 (Vehicle Hours) | 116 | 686 |
| Total Daily Average User Delay Cost | $4,090 | $37,042 |
| Annual Average User Delay Cost | $ 1,492,671 | $ 13,520,414 |

**A 15-minute cycle has been assumed for the bridge opening to allow for the analysis. The bridge is regulated to open on demand and not according to a set cycle.*

[a]*Bridge Opening Duration obtained from power function derived from survey data.*

[b]*Boat delay obtained as a sum of 1/2 bridge cycle period and 1/2 the bridge opening time MINUS the service flow rate per vessel, all multiplied by the number of vessels serviced during the opening.*

[c]*"peak hour" delay factored into daily average delay.*

Table 3.18 Peak-Hour Method Analyses for BRIDGE ID 1500050 (Pinellas Bay Way)

| ***TRAFFIC CHARACTERISTICS*** | YEAR 2002 | YEAR 2020 |
|---|---|---|
| Base year | 2002 | 2002 |
| Future year | 2002 | 2020 |
| Base yr AADT | 15800 | 15800 |
| Peak Hour Factor -k | 9.88 | 9.88 |
| Directional Factor - D | 59.18 | 59.18 |
| Traffic Growth rate | 0.025 | 0.025 |
| | | |
| ***BRIDGE OPENING INPUT*** | | |
| BRIDGE CYCLE LENGTH (minutes) | 20 | 20 |
| PEAK. HRLY VESSEL TRAFFIC (vess/hr) | 9 | 15 |
| AVG. SERVICE TIME (minutes) | 1.42 | 1.42 |
| | | |
| ***ROADWAY CHARACTERISTICS*** | | |
| NUMBER OF LANES ON ROADWAY (ONE DIRECTION) | 2 | 2 |
| SATURATION FLOW RATE (veh/hr/ln) | 1850 | 1850 |
| FLOW RATE AT BOTTLENECK (veh/min) | 0 | 0 |
| | | |
| | | |
| ***COST INPUT*** | | |
| CPI - BASE YEAR - (1990) | 127.90 | 127.90 |
| CPI - CURRENT YEAR - (2002) | 173.30 | 173.30 |
| VALUE OF TRAVEL TIME IN BASE YEAR ($/Vehicle-Hour) | 9.75 | 9.75 |
| | | |
| **MODEL OUTPUT** | | |
| AADT | 15800 | 24643 |
| PEAK HRLY VEHICLE TRAFFIC (veh/hr)- Dir 1 | 924 | 1441 |
| PEAK HRLY VEHICLE TRAFFIC (veh/hr) -Dir 2 | 637 | 994 |
| VESSEL QUEUE - vessels | 3 | 5 |
| BRIDGE OPENING TIME[a] -minutes | 5.00 | 5.00 |
| VEHICLE QUEUE -Dir 1 - vehicles | 77 | 120 |
| VEHICLE QUEUE -Dir 2 - vehicles | 53 | 83 |
| VESSEL DELAY[b] -Boat minutes | 33.24 | 55.40 |
| Duration of vehicle queue -Dir 1 (minutes) | 6.66 | 8.19 |
| Number of vehicles affected | 103 | 197 |
| Duration of vehicle queue Dir 2 (minutes) | 6.04 | 6.84 |
| Number of vehicles affected | 64 | 113 |
| DELAY PER VEHICLE - Dir 1 per cycle (minutes) | 2.5 | 2.5 |
| DELAY PER VEHICLE - Dir 2 per cycle (minutes) | 2.5 | 2.5 |
| DELAY PER VESSEL (minutes) | 11.08 | 11.08 |
| VEHICLE DELAY -Dir 1 per cycle (vehicle minutes) | 257 | 492 |
| VEHICLE DELAY - Dir 2 per cycle (vehicle minutes) | 160 | 283 |
| "PEAK HOUR" DELAY-Dir 1 (Vehicle minutes) | 770 | 1475 |
| "PEAK HOUR" DELAY-Dir 2 (Vehicle minutes) | 481 | 849 |
| Daily Average Delay[c] -Dir 1 (Vehicle Hours) | 104 | 199 |
| Daily Average Delay -Dir 2 (Vehicle Hours) | 65 | 115 |
| Total Daily Average User Delay Cost | $2,230 | $5,772 |
| Annual Average User Delay Cost | $ 813,845 | $ 2,106,730 |

[a] *A minimum opening time of 5 minutes is set for when 5 or less vessels are in queue.*
[b] *Boat delay obtained as a sum of 1/2 bridge cycle period and 1/2 the bridge opening time MINUS the service flow rate per vessel, all multiplied by the number of vessels serviced during the opening.*
[c] *"peak hour" delay factored into daily average delay.*

Table 3.19 Average Hour Method Analyses for BRIDGE ID 860060 (N.E. 14th Street Causeway)

| ***TRAFFIC CHARACTERISTICS*** | YEAR 2002 | YEAR 2020 |
|---|---|---|
| Base year | 2002 | 2002 |
| Future year | 2002 | 2020 |
| Base yr AADT | 15100 | 15100 |
| Directional Split - (Peak direction) | 0.5632 | 0.5632 |
| Traffic Growth rate | 0.025 | 0.025 |
| Fraction of Daylight Traffic[a] | 0.75 | 0.75 |
| | | |
| ***BRIDGE OPENING INPUT*** | | |
| BRIDGE CYCLE LENGTH (minutes) | 30 | 30 |
| AVG. HRLY VESSEL TRAFFIC (vess/hr) | 9 | 15 |
| AVG. SERVICE TIME (minutes) | 1.42 | 1.42 |
| SATURATION FLOW RATE (veh/hr) | 1850 | 1850 |
| FLOW RATE AT BOTTLENECK (veh/min) | 0 | 0 |
| DAILY No. of OPENINGS | 24 | 24 |
| | | |
| ***COST INPUT*** | | |
| CPI - BASE YEAR - (1990) | 127.90 | 127.90 |
| CPI - CURRENT YEAR - (2002) | 173.30 | 173.30 |
| VALUE OF TRAVEL TIME IN BASE YEAR ($/Vehicle-Hour) | 9.75 | 9.75 |
| | | |
| **MODEL OUTPUT** | | |
| AADT | 15100 | 23551 |
| AVG. HRLY Daylight VEHICLE TRAFFIC (veh/hr)- Dir 1 | 532 | 829 |
| AVG. HRLY *Daylight* VEHICLE TRAFFIC (veh/hr) -Dir 2 | 412 | 643 |
| VESSEL QUEUE - vessels | 5 | 8 |
| BRIDGE OPENING TIME -minutes | 7.10 | 11.36 |
| VEHICLE QUEUE -Dir 1 - vehicles | 63 | 157 |
| VEHICLE QUEUE -Dir 1 - vehicles | 49 | 122 |
| VESSEL DELAY[b] -Boat minutes | 85.65 | 154.08 |
| Duration of vehicle queue -Dir 1 (minutes) | 8.29 | 14.64 |
| Number of vehicles affected | 73 | 202 |
| Duration of vehicle queue Dir 2 (minutes) | 7.99 | 13.75 |
| Number of vehicles affected | 55 | 147 |
| VEHICLE DELAY -Dir 1 (vehicle minutes) | 261 | 1149 |
| VEHICLE DELAY - Dir 2 (vehicle minutes) | 195 | 837 |
| DELAY PER VEHICLE - Dir 1(minutes) | 3.55 | 5.68 |
| DELAY PER VEHICLE - Dir 2(minutes) | 3.55 | 5.68 |
| DELAY PER VESSEL (minutes) | 17.13 | 19.26 |
| | | |
| Daily Average User Delay Cost | $ 2,408 | $ 14,617 |
| | | |
| Annual Average User Delay Cost | $ 878,801 | $ 5,335,214 |

[a] *Fraction of daylight vehicular traffic mostly affected by bridge openings.*
[b]*Boat delay obtained as a sum of 1/2 bridge cycle period and 1/2 the bridge opening time MINUS the service flow rate per vessel, all multiplied by the number of vessels serviced during the opening.*

Table 3.20 Average Hour Method Analyses for BRIDGE ID 930004 (Parker)

| ***TRAFFIC CHARACTERISTICS*** | YEAR 2002 | YEAR 2020 |
|---|---|---|
| Base year | 2002 | 2002 |
| Future year | 2002 | 2020 |
| Base yr AADT | 25000 | 25000 |
| Directional Split - (Peak direction) | 0.584 | 0.584 |
| Traffic Growth rate | 0.025 | 0.025 |
| Fraction of Daylight Traffic[a] | 0.75 | 0.75 |
| | | |
| ***BRIDGE OPENING INPUT*** | | |
| BRIDGE CYCLE LENGTH (minutes) | 20 | 20 |
| AVG. HRLY VESSEL TRAFFIC (vess/hr) | 5 | 8 |
| AVG. SERVICE TIME (minutes) | 1.42 | 1.42 |
| SATURATION FLOW RATE (veh/hr) | 1850 | 1850 |
| FLOW RATE AT BOTTLENECK (veh/min) | 0 | 0 |
| DAILY No. of OPENINGS | 24 | 24 |
| | | |
| ***COST INPUT*** | | |
| CPI - BASE YEAR - (1990) | 127.90 | 127.90 |
| CPI - CURRENT YEAR - (2002) | 173.30 | 173.30 |
| VALUE OF TRAVEL TIME IN BASE YEAR ($/Vehicle-Hour) | 9.75 | 9.75 |
| | | |
| **MODEL OUTPUT** | | |
| AADT | 25000 | 38991 |
| AVG. HRLY Daylight VEHICLE TRAFFIC (veh/hr)- Dir 1 | 913 | 1423 |
| AVG. HRLY *Daylight* VEHICLE TRAFFIC (veh/hr) -Dir 2 | 650 | 1014 |
| VESSEL QUEUE - vessels | 2 | 3 |
| BRIDGE OPENING TIME -minutes | 2.84 | 4.26 |
| VEHICLE QUEUE -Dir 1 - vehicles | 43 | 101 |
| VEHICLE QUEUE -Dir 1 - vehicles | 31 | 72 |
| VESSEL DELAY[b] -Boat minutes | 20.00 | 32.13 |
| Duration of vehicle queue -Dir 1 (minutes) | 3.77 | 6.92 |
| Number of vehicles affected | 57 | 164 |
| Duration of vehicle queue Dir 2 (minutes) | 3.45 | 5.87 |
| Number of vehicles affected | 37 | 99 |
| VEHICLE DELAY -Dir 1 (vehicle minutes) | 81 | 350 |
| VEHICLE DELAY - Dir 2 (vehicle minutes) | 53 | 211 |
| DELAY PER VEHICLE - Dir 1(minutes) | 1.42 | 2.13 |
| DELAY PER VEHICLE - Dir 2(minutes) | 1.42 | 2.13 |
| DELAY PER VESSEL (minutes) | 10.00 | 10.71 |
| | | |
| Daily Average User Delay Cost | $ 710 | $ 4,129 |
| | | |
| Annual Average User Delay Cost | $ 259,247 | $ 1,507,090 |

[a] *Fraction of daylight vehicular traffic mostly affected by bridge openings.*

[b]*Boat delay obtained as a sum of 1/2 bridge cycle period and 1/2 the bridge opening time MINUS the service flow rate per vessel, all multiplied by the number of vessels serviced during the opening.*

Table 3.21 Average Hour Method Analyses for BRIDGE ID 150027 (St. John's Pass)

| ***TRAFFIC CHARACTERISTICS*** | YEAR 2002 | YEAR 2020 |
|---|---|---|
| Base year | 2002 | 2002 |
| Future year | 2002 | 2020 |
| Base yr AADT | 21000 | 21000 |
| Directional Split - (Peak direction) | 0.5918 | 0.5918 |
| Traffic Growth rate | 0.025 | 0.025 |
| Fraction of Daylight Traffic[a] | 0.75 | 0.75 |
| | | |
| ***BRIDGE OPENING INPUT*** | | |
| BRIDGE CYCLE LENGTH (minutes) | 30 | 30 |
| AVG. HRLY VESSEL TRAFFIC (vess/hr) | 7 | 12 |
| AVG. SERVICE TIME (minutes) | 1.42 | 1.42 |
| SATURATION FLOW RATE (veh/hr) | 1850 | 1850 |
| FLOW RATE AT BOTTLENECK (veh/min) | 0 | 0 |
| DAILY No. of OPENINGS | 24 | 24 |
| | | |
| ***COST INPUT*** | | |
| CPI - BASE YEAR - (1990) | 127.90 | 127.90 |
| CPI - CURRENT YEAR - (2002) | 173.30 | 173.30 |
| VALUE OF TRAVEL TIME IN BASE YEAR ($/Vehicle-Hour) | 9.75 | 9.75 |
| | | |
| **MODEL OUTPUT** | | |
| AADT | 21000 | 32753 |
| AVG. HRLY Daylight VEHICLE TRAFFIC (veh/hr)- Dir 1 | 777 | 1211 |
| AVG. HRLY *Daylight* VEHICLE TRAFFIC (veh/hr) -Dir 2 | 536 | 836 |
| VESSEL QUEUE - vessels | 4 | 6 |
| BRIDGE OPENING TIME -minutes | 5.68 | 8.52 |
| VEHICLE QUEUE -Dir 1 - vehicles | 74 | 172 |
| VEHICLE QUEUE -Dir 1 - vehicles | 51 | 119 |
| VESSEL DELAY[b] -Boat minutes | 65.68 | 107.04 |
| Duration of vehicle queue -Dir 1 (minutes) | 7.19 | 12.67 |
| Number of vehicles affected | 93 | 256 |
| Duration of vehicle queue Dir 2 (minutes) | 6.64 | 11.01 |
| Number of vehicles affected | 59 | 153 |
| VEHICLE DELAY -Dir 1 (vehicle minutes) | 264 | 1090 |
| VEHICLE DELAY - Dir 2 (vehicle minutes) | 168 | 653 |
| DELAY PER VEHICLE - Dir 1(minutes) | 2.84 | 4.26 |
| DELAY PER VEHICLE - Dir 2(minutes) | 2.84 | 4.26 |
| DELAY PER VESSEL (minutes) | 16.42 | 17.84 |
| | | |
| Daily Average User Delay Cost | $ 2,287 | $ 12,826 |
| | | |
| Annual Average User Delay Cost | $ 834,678 | $ 4,681,646 |

[a] *Fraction of daylight vehicular traffic mostly affected by bridge openings.*

[b]*Boat delay obtained as a sum of 1/2 bridge cycle period and 1/2 the bridge opening time MINUS the service flow rate per vessel, all multiplied by the number of vessels serviced during the opening.*

Table 3.22 Average Hour Method Analyses for BRIDGE ID 1500050 (Pinellas Bay Way)

| ***TRAFFIC CHARACTERISTICS*** | YEAR 2002 | YEAR 2020 |
|---|---|---|
| Base year | 2002 | 2002 |
| Future year | 2002 | 2020 |
| Base yr AADT | 15800 | 15800 |
| Directional Split - (Peak direction) | 0.5918 | 0.5918 |
| Traffic Growth rate | 0.025 | 0.025 |
| Fraction of Daylight Traffic[a] | 0.75 | 0.75 |
| | | |
| ***BRIDGE OPENING INPUT*** | | |
| BRIDGE CYCLE LENGTH (minutes) | 20 | 20 |
| AVG. HRLY VESSEL TRAFFIC (vess/hr) | 7 | 11 |
| AVG. SERVICE TIME (minutes) | 1.42 | 1.42 |
| SATURATION FLOW RATE (veh/hr) | 1850 | 1850 |
| FLOW RATE AT BOTTLENECK (veh/min) | 0 | 0 |
| DAILY No. of OPENINGS | 24 | 24 |
| | | |
| ***COST INPUT*** | | |
| CPI - BASE YEAR - (1990) | 127.90 | 127.90 |
| CPI - CURRENT YEAR - (2002) | 173.30 | 173.30 |
| VALUE OF TRAVEL TIME IN BASE YEAR ($/Vehicle-Hour) | 9.75 | 9.75 |
| | | |
| **MODEL OUTPUT** | | |
| AADT | 15800 | 24643 |
| AVG. HRLY Daylight VEHICLE TRAFFIC (veh/hr)- Dir 1 | 584 | 911 |
| AVG. HRLY *Daylight* VEHICLE TRAFFIC (veh/hr) -Dir 2 | 403 | 629 |
| VESSEL QUEUE - vessels | 3 | 4 |
| BRIDGE OPENING TIME -minutes | 4.26 | 5.68 |
| VEHICLE QUEUE -Dir 1 - vehicles | 41 | 86 |
| VEHICLE QUEUE -Dir 1 - vehicles | 29 | 60 |
| VESSEL DELAY[b] -Boat minutes | 32.13 | 45.68 |
| Duration of vehicle queue -Dir 1 (minutes) | 5.06 | 7.54 |
| Number of vehicles affected | 49 | 114 |
| Duration of vehicle queue Dir 2 (minutes) | 4.78 | 6.84 |
| Number of vehicles affected | 32 | 72 |
| VEHICLE DELAY -Dir 1 (vehicle minutes) | 105 | 325 |
| VEHICLE DELAY - Dir 2 (vehicle minutes) | 68 | 204 |
| DELAY PER VEHICLE - Dir 1(minutes) | 2.13 | 2.84 |
| DELAY PER VEHICLE - Dir 2(minutes) | 2.13 | 2.84 |
| DELAY PER VESSEL (minutes) | 10.71 | 11.42 |
| | | |
| Daily Average User Delay Cost | $ 916 | $ 3,892 |
| | | |
| Annual Average User Delay Cost | $ 334,396 | $ 1,420,679 |

[a] *Fraction of daylight vehicular traffic mostly affected by bridge openings.*

[b] *Boat delay obtained as a sum of 1/2 bridge cycle period and 1/2 the bridge opening time MINUS the service flow rate per vessel, all multiplied by the number of vessels serviced during the opening.*

Table 3.23. **Peak Hour (Power Function) Method Analyses for BRIDGE ID 860060 (N.E. 14th Street)**

| ***TRAFFIC CHARACTERISTICS*** | YEAR 2002 | YEAR 2020 |
|---|---|---|
| Base year | 2002 | 2002 |
| Future year | 2002 | 2020 |
| Base yr AADT | 15100 | 15100 |
| Peak Hour Factor -k | 9.39 | 9.39 |
| Directional Factor - D | 56.32 | 56.32 |
| Traffic Growth rate | 0.025 | 0.025 |
| | | |
| ***BRIDGE OPENING INPUT*** | | |
| BRIDGE CYCLE LENGTH (minutes) | 30 | 30 |
| PEAK. HRLY VESSEL TRAFFIC (vess/hr) | 18 | 30 |
| | | |
| ***ROADWAY CHARACTERISTICS*** | | |
| NUMBER OF LANES ON ROADWAY (ONE DIRECTION) | 2 | 2 |
| SATURATION FLOW RATE (veh/hr/ln) | 1850 | 1850 |
| FLOW RATE AT BOTTLENECK (veh/min) | 0 | 0 |
| | | |
| ***COST INPUT*** | | |
| CPI - BASE YEAR - (1990) | 127.90 | 127.90 |
| CPI - CURRENT YEAR - (2002) | 173.30 | 173.30 |
| VALUE OF TRAVEL TIME IN BASE YEAR ($/Vehicle-Hour) | 9.75 | 9.75 |
| | | |
| **MODEL OUTPUT** | | |
| AADT | 15100 | 23551 |
| PEAK HRLY VEHICLE TRAFFIC (veh/hr)- Dir 1 | 799 | 1245 |
| PEAK HRLY VEHICLE TRAFFIC (veh/hr) -Dir 2 | 619 | 966 |
| VESSEL QUEUE - vessels | 9 | 15 |
| BRIDGE OPENING TIME[a] -minutes | 6.67 | 7.20 |
| VEHICLE QUEUE -Dir 1 - vehicles | 89 | 149 |
| VEHICLE QUEUE -Dir 2 - vehicles | 69 | 116 |
| VESSEL DELAY[b] -Boat minutes | 165.02 | 278.97 |
| Duration of vehicle queue -Dir 1 (minutes) | 8.51 | 10.85 |
| Number of vehicles affected | 113 | 225 |
| Duration of vehicle queue Dir 2 (minutes) | 8.01 | 9.74 |
| Number of vehicles affected | 83 | 157 |
| DELAY PER VEHICLE - Dir 1 per cycle (minutes) | 3.34 | 3.597688007 |
| DELAY PER VEHICLE - Dir 2 per cycle (minutes) | 3.34 | 3.597688007 |
| DELAY PER VESSEL (minutes) | 18.34 | 18.59768801 |
| VEHICLE DELAY -Dir 1 per cycle (vehicle minutes) | 378 | 810 |
| VEHICLE DELAY - Dir 2 per cycle (vehicle minutes) | 276 | 564 |
| "PEAK HOUR" DELAY-Dir 1 (Vehicle minutes) | 756 | 1620 |
| "PEAK HOUR" DELAY-Dir 2 (Vehicle minutes) | 552 | 1128 |
| Daily Average Delay[c] -Dir 1 (Vehicle Hours) | 110 | 232 |
| Daily Average Delay -Dir 2 (Vehicle Hours) | 80 | 161 |
| Total Daily Average User Delay Cost | $2,508 | $7,239 |
| Annual Average User Delay Cost | $ 915,424 | $ 2,642,413 |

[a]*Bridge Opening Duration obtained from power function derived from survey data.*
[b]*Boat delay obtained as a sum of 1/2 bridge cycle period and 1/2 the bridge opening time MINUS the service flow rate per vessel, all multiplied by the number of vessels serviced during the opening.*
[c]*"peak hour" delay factored into daily average delay.*

Table 3.24 Peak Hour (Power Function) Method Analyses for BRIDGE ID 930004 (Parker)

| ***TRAFFIC CHARACTERISTICS*** | YEAR 2002 | YEAR 2020 |
|---|---|---|
| Base year | 2002 | 2002 |
| Future year | 2002 | 2020 |
| Base yr AADT | 25000 | 25000 |
| Peak Hour Factor -k | 10.19 | 10.19 |
| Directional Factor - D | 58.4 | 58.4 |
| Traffic Growth rate | 0.025 | 0.025 |
| | | |
| ***BRIDGE OPENING INPUT*** | | |
| BRIDGE CYCLE LENGTH (minutes) | 20 | 20 |
| PEAK. HRLY VESSEL TRAFFIC (vess/hr) | 9 | 15 |
| | | |
| ***ROADWAY CHARACTERISTICS*** | | |
| NUMBER OF LANES ON ROADWAY (ONE DIRECTION) | 2 | 2 |
| SATURATION FLOW RATE (veh/hr/ln) | 1850 | 1850 |
| FLOW RATE AT BOTTLENECK (veh/min) | 0 | 0 |
| | | |
| ***COST INPUT*** | | |
| CPI - BASE YEAR - (1990) | 127.90 | 127.90 |
| CPI - CURRENT YEAR - (2002) | 173.30 | 173.30 |
| VALUE OF TRAVEL TIME IN BASE YEAR ($/Vehicle-Hour) | 9.75 | 9.75 |
| | | |
| **MODEL OUTPUT** | | |
| AADT | 25000 | 38991 |
| PEAK HRLY VEHICLE TRAFFIC (veh/hr)- Dir 1 | 1488 | 2320 |
| PEAK HRLY VEHICLE TRAFFIC (veh/hr) -Dir 2 | 1060 | 1653 |
| VESSEL QUEUE - vessels | 3 | 5 |
| BRIDGE OPENING TIME[a] -minutes | 5.90 | 6.39 |
| VEHICLE QUEUE -Dir 1 - vehicles | 146 | 247 |
| VEHICLE QUEUE -Dir 2 - vehicles | 104 | 176 |
| VESSEL DELAY[b] -Boat minutes | 38.86 | 65.98 |
| Duration of vehicle queue -Dir 1 (minutes) | 9.87 | 17.15 |
| Number of vehicles affected | 245 | 663 |
| Duration of vehicle queue Dir 2 (minutes) | 8.27 | 11.56 |
| Number of vehicles affected | 146 | 318 |
| DELAY PER VEHICLE - Dir 1 per cycle (minutes) | 2.95 | 3.20 |
| DELAY PER VEHICLE - Dir 2 per cycle (minutes) | 2.95 | 3.20 |
| DELAY PER VESSEL (minutes) | 12.95 | 13.20 |
| VEHICLE DELAY -Dir 1 per cycle (vehicle minutes) | 723 | 2120 |
| VEHICLE DELAY - Dir 2 per cycle (vehicle minutes) | 431 | 1018 |
| "PEAK HOUR" DELAY-Dir 1 (Vehicle minutes) | 2168 | 6359 |
| "PEAK HOUR" DELAY-Dir 2 (Vehicle minutes) | 1294 | 3053 |
| Daily Average Delay[c] -Dir 1 (Vehicle Hours) | 274 | 788 |
| Daily Average Delay -Dir 2 (Vehicle Hours) | 163 | 378 |
| Total Daily Average User Delay Cost | $5,776 | $21,466 |
| Annual Average User Delay Cost | $ 2,108,205 | $ 7,835,189 |

[a] *Bridge Opening Duration obtained from power function derived from survey data.*
[b] *Boat delay obtained as a sum of 1/2 bridge cycle period and 1/2 the bridge opening time MINUS the service flow rate per vessel, all multiplied by the number of vessels serviced during the opening.*
[c] *"peak hour" delay factored into daily average delay.*

Table 3.25 Peak Hour (Power Function) Method Analyses for BRIDGE ID 150027 (St. John's Pass)

| ***TRAFFIC CHARACTERISTICS*** | YEAR 2002 | YEAR 2020 |
|---|---|---|
| Base year | 2002 | 2002 |
| Future year | 2002 | 2020 |
| Base yr AADT | 21000 | 21000 |
| Peak Hour Factor -k | 9.88 | 9.88 |
| Directional Factor - D | 59.18 | 59.18 |
| Traffic Growth rate | 0.025 | 0.025 |
| | | |
| ***BRIDGE OPENING INPUT*** | | |
| BRIDGE CYCLE LENGTH* (minutes) | 15 | 15 |
| PEAK. HRLY VESSEL TRAFFIC (vess/hr) | 15 | 25 |
| | | |
| ***ROADWAY CHARACTERISTICS*** | | |
| NUMBER OF LANES ON ROADWAY (ONE DIRECTION) | 2 | 2 |
| SATURATION FLOW RATE (veh/hr/ln) | 1850 | 1850 |
| FLOW RATE AT BOTTLENECK (veh/min) | 0 | 0 |
| | | |
| ***COST INPUT*** | | |
| CPI - BASE YEAR - (1990) | 127.90 | 127.90 |
| CPI - CURRENT YEAR - (2002) | 173.30 | 173.30 |
| VALUE OF TRAVEL TIME IN BASE YEAR ($/Vehicle-Hour) | 9.75 | 9.75 |
| | | |
| **MODEL OUTPUT** | | |
| AADT | 21000 | 32753 |
| PEAK HRLY VEHICLE TRAFFIC (veh/hr)- Dir 1 | 1228 | 1915 |
| PEAK HRLY VEHICLE TRAFFIC (veh/hr) -Dir 2 | 847 | 1321 |
| VESSEL QUEUE - vessels | 4 | 7 |
| BRIDGE OPENING TIME[a] -minutes | 5.75 | 6.31 |
| VEHICLE QUEUE -Dir 1 - vehicles | 118 | 201 |
| VEHICLE QUEUE -Dir 2 - vehicles | 81 | 139 |
| VESSEL DELAY[b] -Boat minutes | 41.49 | 74.59 |
| Duration of vehicle queue -Dir 1 (minutes) | 8.60 | 13.08 |
| Number of vehicles affected | 176 | 418 |
| Duration of vehicle queue Dir 2 (minutes) | 7.45 | 9.82 |
| Number of vehicles affected | 105 | 216 |
| DELAY PER VEHICLE - Dir 1 per cycle (minutes) | 2.87 | 3.16 |
| DELAY PER VEHICLE - Dir 2 per cycle (minutes) | 2.87 | 3.16 |
| DELAY PER VESSEL (minutes) | 10.37 | 10.66 |
| VEHICLE DELAY -Dir 1 per cycle (vehicle minutes) | 506 | 1318 |
| VEHICLE DELAY - Dir 2 per cycle (vehicle minutes) | 302 | 682 |
| "PEAK HOUR" DELAY-Dir 1 (Vehicle minutes) | 2023 | 5271 |
| "PEAK HOUR" DELAY-Dir 2 (Vehicle minutes) | 1209 | 2728 |
| Daily Average Delay[c] -Dir 1 (Vehicle Hours) | 247 | 626 |
| Daily Average Delay -Dir 2 (Vehicle Hours) | 147 | 324 |
| Total Daily Average User Delay Cost | $5,207 | $17,476 |
| Annual Average User Delay Cost | $ 1,900,518 | $ 6,378,637 |

**A 15-minute cycle has been assumed for the bridge opening to allow for the analysis. The bridge is regulated to open on demand and not according to a set cycle.*

[a]*Bridge Opening Duration obtained from power function derived from survey data.*

[b]*Boat delay obtained as a sum of 1/2 bridge cycle period and 1/2 the bridge opening time MINUS the service flow rate per vessel, all multiplied by the number of vessels serviced during the opening.*

[c]*"peak hour" delay factored into daily average delay.*

Table 3.26 Peak Hour (Power Function) Method Analyses for BRIDGE ID 1500050 (Pinellas Bay Way)

| ***TRAFFIC CHARACTERISTICS*** | YEAR 2002 | YEAR 2020 |
|---|---|---|
| Base year | 2002 | 2002 |
| Future year | 2002 | 2020 |
| Base yr AADT | 15800 | 15800 |
| Peak Hour Factor -k | 9.88 | 9.88 |
| Directional Factor - D | 59.18 | 59.18 |
| Traffic Growth rate | 0.025 | 0.025 |
| | | |
| ***BRIDGE OPENING INPUT*** | | |
| BRIDGE CYCLE LENGTH (minutes) | 20 | 20 |
| PEAK. HRLY VESSEL TRAFFIC (vess/hr) | 9 | 15 |
| | | |
| | | |
| ***ROADWAY CHARACTERISTICS*** | | |
| NUMBER OF LANES ON ROADWAY (ONE DIRECTION) | 2 | 2 |
| SATURATION FLOW RATE (veh/hr/ln) | 1850 | 1850 |
| FLOW RATE AT BOTTLENECK (veh/min) | 0 | 0 |
| | | |
| | | |
| ***COST INPUT*** | | |
| CPI - BASE YEAR - (1990) | 127.90 | 127.90 |
| CPI - CURRENT YEAR - (2002) | 173.30 | 173.30 |
| VALUE OF TRAVEL TIME IN BASE YEAR ($/Vehicle-Hour) | 9.75 | 9.75 |
| | | |
| **MODEL OUTPUT** | | |
| AADT | 15800 | 24643 |
| PEAK HRLY VEHICLE TRAFFIC (veh/hr)- Dir 1 | 924 | 1441 |
| PEAK HRLY VEHICLE TRAFFIC (veh/hr) -Dir 2 | 637 | 994 |
| VESSEL QUEUE - vessels | 3 | 5 |
| BRIDGE OPENING TIME[a] -minutes | 4.56 | 5.14 |
| VEHICLE QUEUE -Dir 1 - vehicles | 70 | 124 |
| VEHICLE QUEUE -Dir 2 - vehicles | 48 | 85 |
| VESSEL DELAY[b] -Boat minutes | 36.84 | 62.86 |
| Duration of vehicle queue -Dir 1 (minutes) | 6.07 | 8.43 |
| Number of vehicles affected | 94 | 202 |
| Duration of vehicle queue Dir 2 (minutes) | 5.50 | 7.03 |
| Number of vehicles affected | 58 | 117 |
| DELAY PER VEHICLE - Dir 1 per cycle (minutes) | 2.28 | 2.57 |
| DELAY PER VEHICLE - Dir 2 per cycle (minutes) | 2.28 | 2.57 |
| DELAY PER VESSEL (minutes) | 12.28 | 12.57 |
| VEHICLE DELAY -Dir 1 per cycle (vehicle minutes) | 213 | 521 |
| VEHICLE DELAY - Dir 2 per cycle (vehicle minutes) | 133 | 300 |
| "PEAK HOUR" DELAY-Dir 1 (Vehicle minutes) | 639 | 1562 |
| "PEAK HOUR" DELAY-Dir 2 (Vehicle minutes) | 400 | 899 |
| Daily Average Delay[c] -Dir 1 (Vehicle Hours) | 88 | 210 |
| Daily Average Delay -Dir 2 (Vehicle Hours) | 55 | 121 |
| Total Daily Average User Delay Cost | $1,885 | $6,076 |
| Annual Average User Delay Cost | $ 688,193 | $ 2,217,801 |

[a]*Bridge Opening Duration obtained from power function derived from survey data.*
[b]*Boat delay obtained as a sum of 1/2 bridge cycle period and 1/2 the bridge opening time MINUS the service flow rate per vessel, all multiplied by the number of vessels serviced during the opening.*
[c]*"peak hour" delay factored into daily average delay*

## 3.6. Bridge Replacement Analyses

The following alternative improvements were considered for the existing moveable bridges with the aim of either reducing or eliminating user costs incurred by users of the facility:

- Mid-Level moveable bridge with maximum of 45 feet of vertical clearance
- High-Level moveable bridge with maximum of 55 feet of vertical clearance
- Fixed-span bridge with a minimum of 65 of vertical clearance. Based on FDOT design practice and U. S. Coast Guard requirements, only the 65 ft. option should be considered.

### 3.6.1. Methodology and Procedure

Each of the proposed replacement options is intended to reduce delays to both vehicular and vessel traffic; the taller the replacement option the greater the extent of delay reduction.

A higher moveable bridge option will accommodate relatively more vessels in the closed position and therefore reduce the number of openings and the duration of openings. The fixed bridge options however will completely eliminate delays to both vehicles and vessels but may also permanently limit the height of vessels accommodated on the waterway.

Table 3.27 Attributes of Bridge Replacement Options

| REPLACEMENT OPTION | BENEFIT | | DISADVANTAGE |
|---|---|---|---|
| | VEHICLES | VESSELS | |
| MOVEABLE BRIDGE OPTION | REDUCED DELAY DUE TO FEWER NUMBER OF OPENINGS AND FEWER VESSELS SERVICED DURING BRIDGE OPENINGS | 1. REDUCED DELAY DUE TO FEWER VESSELS IN WAITING AREA<br><br>2. ALL VESSELS CAN BE ACCOMMODATED | DELAY TO VESSELS AND VEHICLES DURING BRIDGE CLOSURE AND OPENINGS |
| FIXED BRIDGE OPTION | NO DELAY | NO DELAY | NOT ALL VESSELS MAY BE ACCOMMODATED |

An analysis of possible replacement options for the existing movable bridges was conducted based on the existing physical conditions near and at the existing bridge sites. Data on the bridges was obtained from the National Bridge Inventory (NBI) database for all 153 movable bridges in the State of Florida. For some of the bridge sites, as-built plans were obtained from the appropriate FDOT district office. Other sources of information used were:

1. Florida Traffic Information (FTI)
2. Geographic Information System (GIS) maps of the State of Florida Highway Network
3. Roadway Maps and Aerial Views, sourced from Mapquest.com

Existing conditions at and near each bridge including bridge height, bridge span and the approach roadway were noted. Lengths of roadway from each end of the existing bridge over which the bridge span could be extended without a realignment of the existing roadway were estimated. These were termed as “Available lengths”. The touchdown points for the replacement alternatives were limited to the nearest signalized intersections on either side of the existing bridge and that defined the “available lengths’ assumed for the analysis.

The approach roads to about twenty-three percent (23%) of the 150 bridges analyzed had existing driveways located between the end of the bridge span and the nearest signalized intersection which, may no longer be used as access points to and from properties fronting the existing approach roadway. Some of the replacement alternatives may be geometrically designed to overpass some driveways. A few of the bridges have their existing touchdown points to coastal roads and therefore no extension beyond the estimated available lengths will be geometrically feasible. Some of the existing bridge touchdown points also had close proximity to major roadways, so any higher replacement alternative would require the design of an overpass over these intersecting major roadways.

Additional Bridge spans required for each of the various replacement alternatives were estimated assuming a gradient of between 5% and 6% for each bridge approach span. The tallest replacement option under existing right of way and geometric conditions was selected for each bridge. A sample of the results is shown in the table 3.26.

Table 3.28 Geometric Requirements for Movable Bridge Replacement Options

| Bridge No | Available Length (m) | | Additional Required Length (m) | | | Tallest Bridge Replacement Option |
|---|---|---|---|---|---|---|
| | Direction 1 | Direction 2 | 45-ft Moveable Option | 55-ft Moveable Option | 65-ft Fixed Option | |
| 150044 | 50 | 380 | 107 | 159 | 211 | 0 |
| 790172 | 120 | 800 | 117 | 169 | 221 | 45 |
| 930453 | 110 | 250 | 55 | 107 | 159 | 55 |
| 170036 | 270 | 500 | 107 | 159 | 211 | 65 |
| 124043 | 450 | 500 | 115 | 167 | 219 | 65 |
| **150027**[1] | 1000 | 1000 | 134 | 186 | 238 | 65 |
| **150076**[1] | 1000 | 1000 | 134 | 186 | 238 | 65 |
| **150050**[1] | 700 | 600 | 79 | 131 | 183 | 65 |
| **780074**[1] | 750 | 80 | 107 | 159 | 211 | 0 |
| **860060**[1] | 300 | 930 | 157 | 209 | 261 | 65 |
| **930004**[1] | 500 | 230 | 107 | 159 | 211 | 65 |

[1] Bridges at which data were collected for the study.

In analyzing the available lengths for each of the bridges, other design considerations include the assumption that the proposed replacement alternative would have the same roadway and bridge alignment as the existing. Many possible alternatives of reasonable lengths may be eliminated due to the physical constraints along the existing alignment, including, for instance, crossing streets.. It must however be noted that higher-clearance replacement alternatives may be possible if the replacement bridge and approach roadway could be realigned. This of course would require the procurement of an extended or new right-of –way for the replacement alternative. As shown in the following pages, the geometric layouts of bridge locations were also reviewed to identify possible physical impacts of replacement alternatives on nearest intersections and crossing streets.

There are several other factors that will have to be considered towards the final design of a replacement option, including the following:

- Right-of way acquisition
- Closure or over-passing of existing driveways and the redesign of access to properties

- Relocation of businesses and residences.

The replacement options recommended were made solely from a traffic operations viewpoint. There however will be socio-economic issues such as availability of project funds and community reception, and environmental issues to be addressed before any final design or implementation.

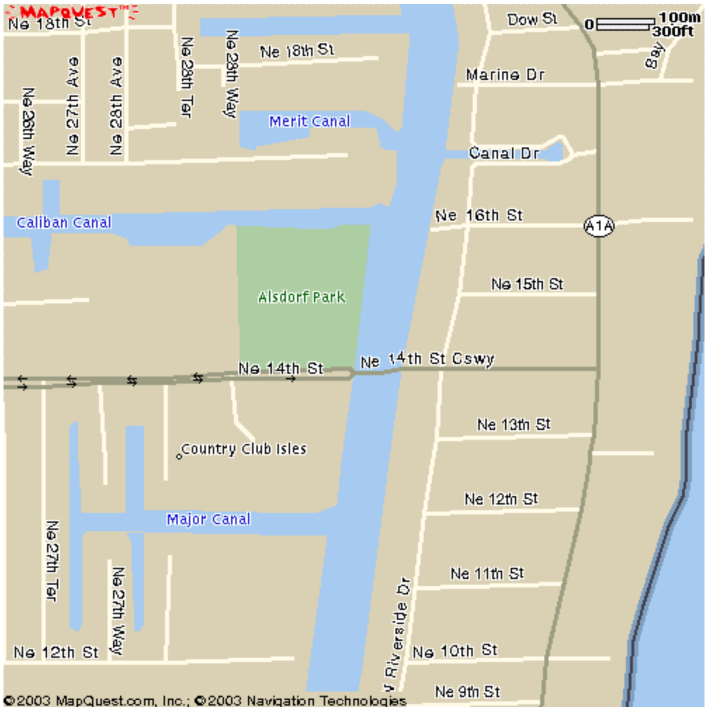

Figure 3.32 Map Layout for Bridge 860060

Figure 3.33 Aerial Photo Map Layout for Bridge 860060

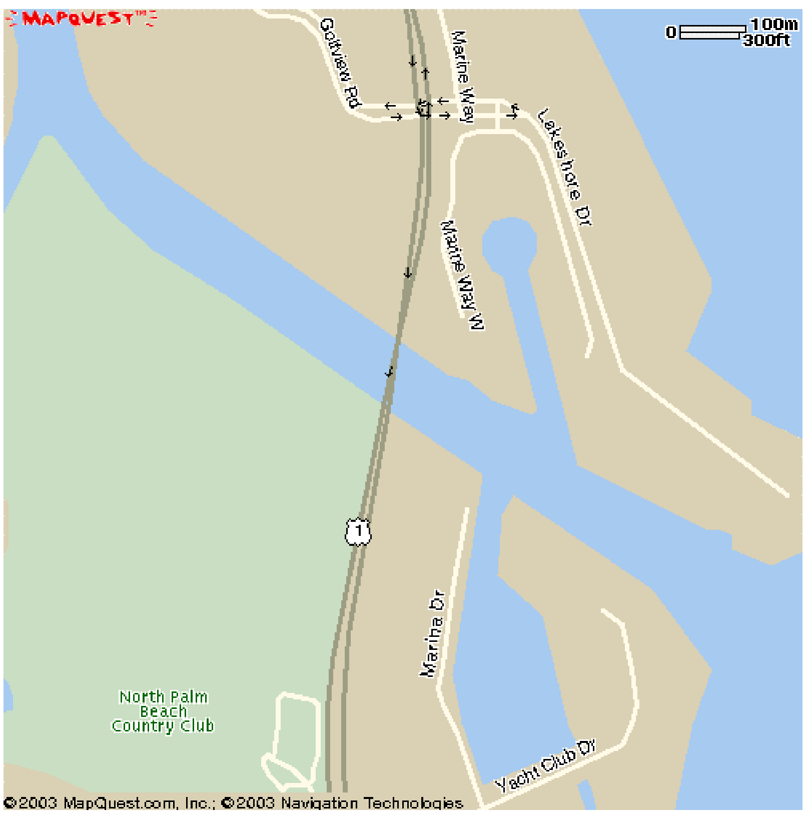

Figure 3.34 Map Layout at Bridge 930040

Figure 3.35 Aerial Photo Map Layout for Bridge 930040

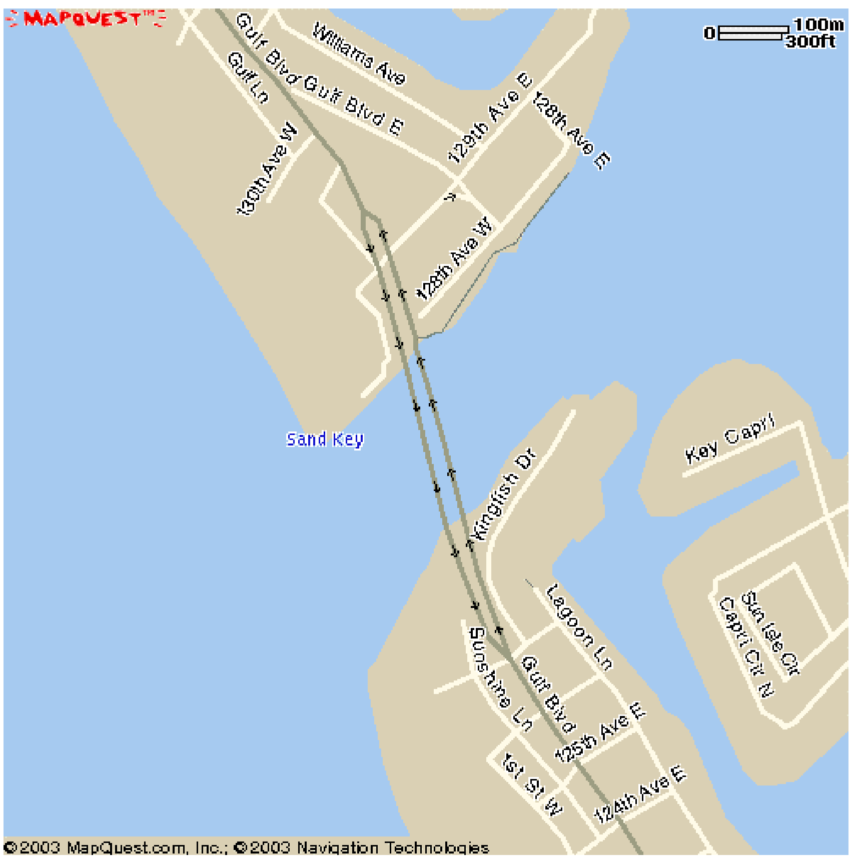

Figure 3.36 Map Layout for Bridge 150027/150076


Figure 3.37 Aerial Photo Map Layout for Bridges 150027/150076


Figure 3.38 Map Layout for Bridge 150050


Figure 3.39 Aerial Photo Map Layout for Bridge 150050

### 3.6.2. Bridge Replacement Evaluation Matrix

An evaluation matrix was developed to evaluate and compare the costs and impacts of selected bridge replacement alternatives. The focus of this evaluation was to determine the minimum level of vertical bridge clearance for which there would be a substantial decrease in vehicular user delay due to openings of the Bridge. Components of the matrix are the user delay costs and the initial costs of each replacement alternative, which include the cost of construction, design, right-of-way and Construction and Engineering Inspection (CEI).

FDOT in a year 2002 Bridge Development Report on Cost Estimating outlined a three-step concept for estimating the cost of planned or proposed bridges. The first step is to utilize the average unit material costs provided to develop a cost estimate based on the completed preliminary design. The second step is to adjust the total bridge cost for the unique site conditions by use of given site adjustment factors. The third and final step is to review the computed total bridge cost on a cost per square foot basis and compare this cost against the historical cost range for similar structure types.

This three-step process should produce a reasonably accurate cost estimate for structure type selection. However, if a site has a set of odd circumstances, which will affect the bridge cost, further adjustments are made to account for these unique site conditions in the estimate. For the evaluation matrix however the developed average unit cost of construction will be used for uniformity of estimates.

Table 3.29 Cost Estimate for Bridge Construction

| Bridge Type | Average Cost Per Square foot of Deck area |
|---|---|
| Fixed | $ 53 |
| Movable | $935 |
| Approach Spans to Movable | $ 53 |

This process will develop costs for the bridge superstructure and substructure from beginning to end bridge. Costs for all other items including but not limited to the following are excluded from the costs estimated with the above values: mobilization, operation costs for existing bridge(s); removal of existing bridge or bridge fenders; lighting; walls; deck drainage systems; embankment; fenders; approach slabs; maintenance of traffic; load tests; bank stabilization.

A linear equation with the bridge height, width and gradient as the variables was derived to estimate the average construction cost of each movable bridge alternative. A fixed cost estimate of $19 million, obtained from analysis of cost estimates presented in the St. John's Pass P.D &E report, is added to represent the cost of design, construction and engineering inspection and other items such as mobilization, operation costs for existing bridge(s); removal of existing bridge or bridge fenders; lighting; walls; deck drainage systems; embankment; fenders; approach slabs; maintenance of traffic; load tests; bank stabilization.

The derived equation is as follows:

$$\boldsymbol{Cm} = 935 \times 10^{-6}(\boldsymbol{X} \times \boldsymbol{W})/\boldsymbol{S} + 19 \qquad (3.13)$$

Where ***Cm*** is the average cost of construction, design and CEI for a replacement movable bridge, (Millions of Dollars); ***X*** is the bridge height (feet); ***W*** is the total bridge width comprising of the roadway pedestrian walkway and median (feet); and ***S*** is the gradient of the bridge expressed as a fraction (e.g. **.05** for 5% slope).

A similar equation was derived for the fixed bridge replacement options. An estimated fixed cost estimate of $28 Million, also obtained from analysis of cost estimates presented in the St. John's Pass P.D &E report, is added to represent the cost of design, construction and engineering inspection and other items such as mobilization, operation costs for existing bridge(s); removal of existing bridge or bridge fenders; lighting; walls; deck drainage systems; embankment; fenders; approach slabs; maintenance of traffic; load tests; bank stabilization.

$$Cf = 53 \cdot 10^{-6} (X \cdot W)/S + 28 \qquad (3.14)$$

Where ***Cf*** is the average cost of construction, design and CEI for a replacement fixed bridge, (millions of dollars); ***X*** is the bridge height (feet); ***W*** is the total bridge width. (feet); and ***S*** is the gradient of the bridge expressed as a fraction of 100 (eg. **.05** for 5% slope)

The costs from the formulated equations were validated against the estimates from the St. John's Pass report. The results show that the formulated equation can be effectively used as a cost model in estimating bridge construction costs, assuming normal site conditions.

Table 3.30 Validation of cost model

| Bridge Height | Cost of Construction ($Million) | |
|---|---|---|
| | Model | St. John's Report |
| 21 | 50.416 | 50.8 |
| 26.5 | 58.644 | 56.8 |
| 29 | 62.384 | 59.6 |
| 32 | 66.872 | 65.6 |
| 32.5 | 67.62 | 66.6 |
| 35 | 71.36 | 78.6 |
| 39 | 77.344 | 76.8 |
| 45 | 86.32 | 90.1 |
| 50 | 93.8 | 89.8 |
| 60 | 108.76 | 107.2 |
| 65 | 33.512 | 34.3 |
| 74 | 34.2752 | 34.5 |

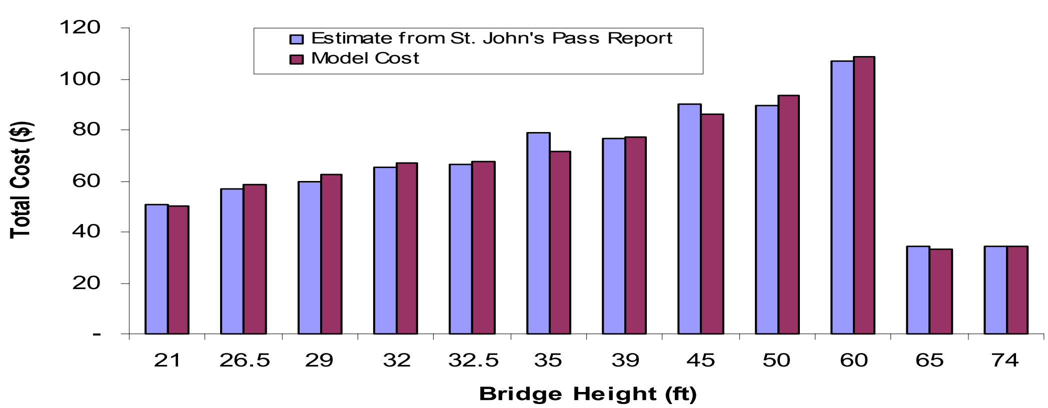


Figure 3.40 Bridge Construction Costs –Model Costs vs. Estimates from St. John's Pass' Project

The additional right-of –way cost estimate required for each replacement alternative is based on the additional length of bridge structure, which is estimated from the bridge height and gradient. The cost involved is usually for the value of property to be purchased to make way for replacement structure. In cases where no additional land is required the right-of-way cost is zero. A linear equation derived from the estimates given in the St. John's Pass PDE report is as follows:

$$Ro = 1.3465 * Xd \qquad (3.15)$$

$$Xd = X - Xo \qquad (3.16)$$

Where ***Ro*** is the average cost estimate of the required right-of-way;
***X*** is the height replacement bridge;
***Xo*** is the height of the existing bridge; and
***Xd*** is the increase in bridge height.

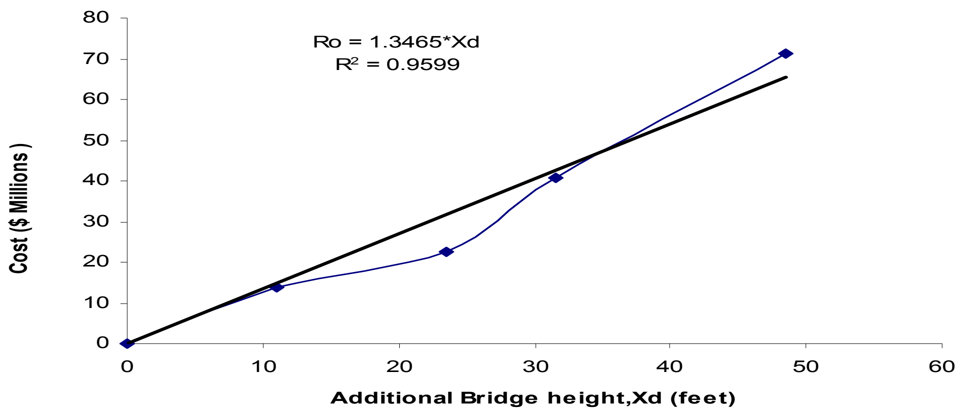


Figure 3.41 Formulation of Equation for Estimation of Right-of-Way Cost

User delay costs were estimated by the Peak Hour and Average methods. The user delay cost for each alternative was estimated based on the number of vessels estimated to be held up until the bridge opening during the peak vehicular traffic hour. The number of vessels held at peak hour was obtained by relating the vessel height distribution obtained from the survey to each replacement bridge's vertical clearance. The percentage of vessels that may be held up was then estimated. For simplicity of analysis the number of vessels at peak hour opening, and therefore the duration of peak hour opening, was assumed to remain the same throughout the life of each movable bridge alternative.

The average method involved estimating the average delay for each opening using the average duration of each opening for the bridge. The total daily delay was then obtained as a product of the average delay per opening and the average number of daily openings.

The user delay cost for each year was estimated, with the annual average vehicular traffic and the corresponding unit cost of travel time as changing variables.
A fixed bridge alternative is assumed to hold up no vessel and will therefore have zero user delay cost.

An economic assessment to estimate the equivalent present worth of user delay cost over the life of the bridge was made by discounting all projected annual user delay costs to the present year. The annual average delay costs were formulated into a series, comprising of two components: (1) A uniform series with value equal to the delay cost at the end of year one; and (2) an arithmetic gradient uniform series representing the annual increase (Figure 3.42) lay at the end of the first yea. The total present worth will therefore be equal to the sum of the discounted uniform series and the discounted arithmetic gradient series over the total number of years for which economic assessment is being made.

The variables needed for the formulation of the user delay cost component of the matrix will be:

1. Total annual average delay (in vehicle-hours)
2. Total annual average user delay cost at end of year one (Dollars)
3. Discount rate
4. Number of years over which the economic assessment of the replacement alternative is to be made, which usually is over the useful life of the bridge.

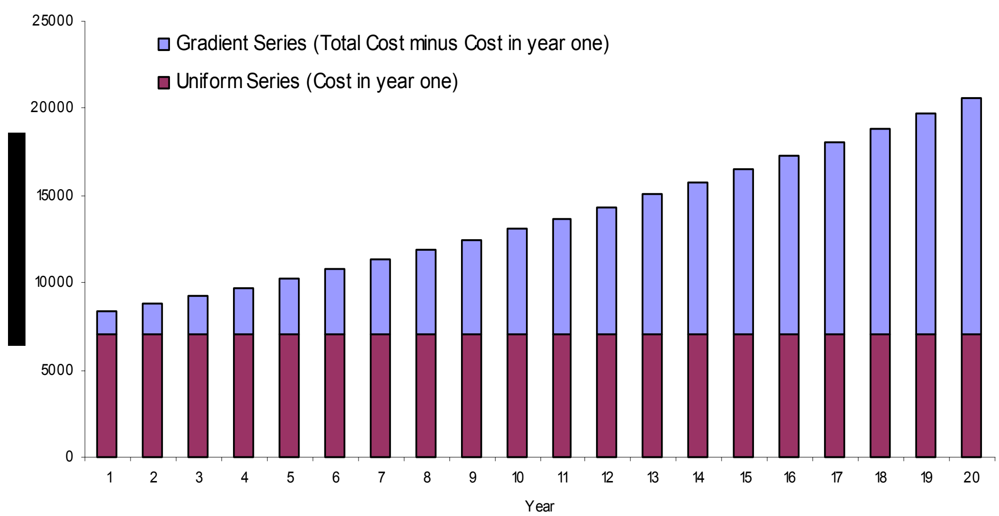


Figure 3.42 Formulation of Cost Series from 20-year Average Daily User Delay Cost (*Peak Hour Method)*

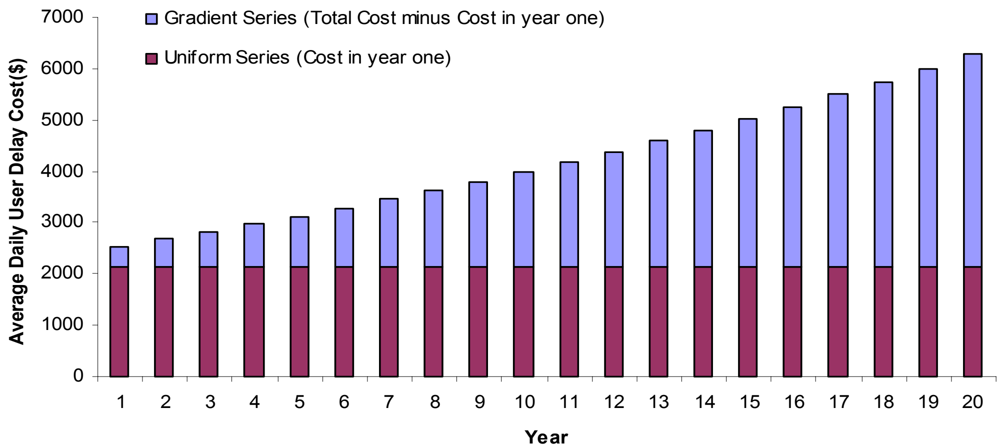


Figure 3.43 Formulation of Cost Series from 20-year Average Daily User Delay Cost *(Average Method)*

Table 3.31 Bridge Replacement Evaluation Matrix - Bridge 860060 (Peak Hour Method)

| Bridge Type | Bridge Geometrics | | | Bridge Openings | | | | | | Initial Costs | | | User Delay Cost | | | Life Cycle Cost | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Height (ft) | Width (ft) | Grade (%) | Estimated Percentage Reduction | Predicted Average Weekday Openings | Predicted Average Weekend Openings | Predicted Average Peak Hour Openings | Annual Delay in Base Year (Vehicle-hrs) | Annual Cost of Delay in Base Year ( $) | Construction/ Design ( in MIL $) | Right-of Way ( in MIL $) | Total ( in MIL $) | User Delay ( in MIL $) | User Delay Savings ( in MIL $) | Benefit/Cost ratio | Total ( in MIL $) | Benefit/Cost ratio |
| Movable | 15 | 80 | 5 | N/A | 20 | 24 | 2 | 217931 | 2879120 | 41.44 | 0.00 | 41.44 | 113.13 | 0.00 | 0.00 | 154.57 | N/A |
| Movable | 45 | 80 | 5 | 60 | 8 | 10 | 2 | 40880 | 538740 | 86.32 | 40.40 | 126.72 | 8.48 | 104.65 | 0.83 | 135.19 | 1.23 |
| Movable | 55 | 80 | 5 | 75 | 5 | 5 | 1 | 40880 | 538740 | 101.28 | 53.86 | 155.14 | 5.30 | 107.83 | 0.70 | 160.44 | 0.95 |
| Fixed | 65 | 80 | 5 | 100 | 0 | 0 | 0 | 0 | 0 | 33.51 | 67.33 | 100.84 | 0.00 | 113.13 | 1.12 | 100.84 | 1.90 |

Table 3.32 Bridge Replacement Evaluation Matrix – Bridge 930004 (Peak Hour Method)

| Bridge Type | Bridge Geometrics | | | Bridge Openings | | | | | | Initial Costs | | | User Delay Cost | | | Life Cycle Cost | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Height (ft) | Width (ft) | Grade (%) | Estimated Percentage Reduction | Predicted Average Weekday Openings | Predicted Average Weekend Openings | Predicted Average Peak Hour Openings | Annual Delay in Base Year (Vehicle-hrs) | Annual Cost of Delay in Base Year ( $) | Construction/ Design ( in MIL $) | Right-of Way ( in MIL $) | Total ( in MIL $) | User Delay ( in MIL $) | User Delay Savings ( in MIL $) | Benefit/Cost ratio | Total ( in MIL $) | Benefit/Cost ratio |
| Movable | 25 | 80 | 5 | N/A | 16 | 30 | 3 | 118603 | 1566859 | 56.40 | 0.00 | 56.40 | 61.57 | 0.00 | 0.00 | 117.97 | N/A |
| Movable | 45 | 80 | 5 | 50 | 8 | 15 | 3 | 118603 | 1566859 | 86.32 | 26.93 | 113.25 | 30.78 | 30.78 | 0.27 | 144.03 | 0.54 |
| Movable | 55 | 80 | 5 | 65 | 5 | 10 | 2 | 84717 | 1119185 | 101.28 | 40.40 | 141.68 | 15.39 | 46.17 | 0.33 | 157.07 | 0.54 |
| Fixed | 65 | 80 | 5 | 100 | 0 | 0 | 0 | 0 | 0.00 | 33.51 | 53.86 | 87.37 | 0.00 | 61.57 | 0.70 | 87.37 | 1.99 |

Table 3.33 Bridge Replacement Evaluation Matrix - Bridge 150027 (Peak Hour Method)

| Bridge Type | Bridge Geometrics | | | Bridge Openings | | | | | | Initial Costs | | | User Delay Cost | | | Life Cycle Cost | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Height (ft) | Width (ft) | Grade (%) | Estimated Percentage Reduction | Predicted Average Weekday Openings | Predicted Average Weekend Openings | Predicted Average Peak Hour Openings | Annual Delay in Base Year (Vehicle-hrs) | Annual Cost of Delay in Base Year ( $) | Construction/ Design ( in MIL $) | Right-of Way ( in MIL $) | Total ( in MIL $) | User Delay ( in MIL $) | User Delay Savings ( in MIL $) | Benefit/Cost ratio | Total ( in MIL $) | Benefit/Cost ratio |
| Movable | 21 | 80 | 5 | N/A | 24 | 36 | 5 | 112988 | 1492671 | 50.42 | 0.00 | 50.42 | 58.65 | 0.00 | 0.00 | 109.07 | N/A |
| Movable | 45 | 80 | 5 | 40 | 8 | 10 | 3 | 90390 | 1194137 | 86.32 | 32.32 | 118.64 | 28.15 | 30.50 | 0.26 | 146.79 | 0.45 |
| Movable | 55 | 80 | 5 | 70 | 5 | 5 | 1 | 64564 | 852955 | 101.28 | 45.78 | 147.06 | 10.05 | 48.60 | 0.33 | 157.12 | 0.50 |
| Fixed | 65 | 80 | 5 | 100 | 0 | 0 | 0 | 0 | 0 | 33.51 | 59.25 | 92.76 | 0.00 | 58.65 | 0.63 | 92.76 | 1.39 |

Table 3.34 Bridge Replacement Evaluation Matrix – Bridge 150050 (Peak Hour Method)

| Bridge Type | Bridge Geometrics | | | Bridge Openings | | | | | | Initial Costs | | | User Delay Cost | | | Life Cycle Cost | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Height (ft) | Width (ft) | Grade (%) | Estimated Percentage Reduction | Predicted Average Weekday Openings | Predicted Average Weekend Openings | Predicted Average Peak Hour Openings | Annual Delay in Base Year (Vehicle-hrs) | Annual Cost of Delay in Base Year ( $) | Construction/ Design ( in MIL $) | Right-of Way ( in MIL $) | Total ( in MIL $) | User Delay ( in MIL $) | User Delay Savings ( in MIL $) | Benefit/Cost ratio | Total ( in MIL $) | Benefit/Cost ratio |
| Movable | 23 | 80 | 5 | N/A | 24 | 30 | 3 | 61604 | 813845 | 53.41 | 0.00 | 53.41 | 31.98 | 0.00 | 0.00 | 85.39 | N/A |
| Movable | 45 | 80 | 5 | 30 | 16 | 20 | 3 | 61604 | 813845 | 86.32 | 29.62 | 115.94 | 22.38 | 9.59 | 0.08 | 138.33 | 0.15 |
| Movable | 55 | 80 | 5 | 60 | 8 | 12 | 2 | 44003 | 538740 | 101.28 | 43.09 | 144.37 | 8.83 | 23.15 | 0.16 | 153.19 | 0.25 |
| Fixed | 65 | 80 | 5 | 100 | 0 | 0 | 0 | 0 | 0 | 33.51 | 56.55 | 90.07 | 0.00 | 31.98 | 0.36 | 90.07 | 0.87 |

Table 3.35 Bridge Replacement Evaluation Matrix – Bridge 860060 (Average Method)

| Bridge Type | Bridge Geometrics | | | Bridge Openings | | | | | | | Initial Costs | | | User Delay Costs | | | Life Cycle Cost | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Height (ft) | Width (ft) | Grade (%) | Estimated Percentage Reduction | Predicted Average Weekday Openings | Predicted Average Weekend Openings | Predicted Average Peak Hour Openings | Average Delay per opening in Base Year (Vehicle-hrs) | Annual Delay in Base Year (Vehicle-hrs) | Annual Cost of Delay in Base Year ( $) | Construction/ Design ( in MIL $) | R.O.W ( in MIL $) | Total ( in MIL $) | User Delay ( in MIL $) | Savings ( in MIL $) | Benefit/ Cost ratio | Total | Benefit/ Cost ratio |
| Movable | 15 | 80 | 5 | N/A | 20 | 24 | 2 | 3.77 | 29014 | 383274 | 41.44 | 0.00 | 41.44 | 15.06 | 0.00 | 0.00 | 56.50 | N/A |
| Movable | 45 | 80 | 5 | 60 | 8 | 10 | 2 | 3.77 | 11762 | 155381 | 86.32 | 40.40 | 126.72 | 6.11 | 8.95 | 0.07 | 132.82 | 0.11 |
| Movable | 55 | 80 | 5 | 75 | 5 | 5 | 1 | 3.77 | 6861 | 90639 | 101.28 | 53.86 | 155.14 | 3.56 | 11.50 | 0.07 | 158.70 | 0.10 |
| Fixed | 65 | 80 | 5 | 100 | 0 | 0 | 0 | 0 | 0 | 0 | 33.51 | 67.33 | 100.84 | 0.00 | 15.06 | 0.15 | 100.84 | 0.25 |

Table 3.36 Bridge Replacement Evaluation Matrix - Bridge 93004 (Average Method)

| Bridge Type | Bridge Geometrics | | | Bridge Openings | | | | | | | Initial Costs | | | User Delay Costs | | | Life Cycle Cost | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Height (ft) | Width (ft) | Grade (%) | Estimated Percentage Reduction | Predicted Average Weekday Openings | Predicted Average Weekend Openings | Predicted Average Peak Hour Openings | Average Delay per opening in Base Year (Vehicle-hrs) | Annual Delay in Base Year (Vehicle-hrs) | Annual Cost of Delay in Base Year ( $) | Construction/ Design ( in MIL $) | R.O.W ( in MIL $) | Total ( in MIL $) | User Delay ( in MIL $) | Savings ( in MIL $) | Benefit/ Cost ratio | Total | Benefit/ Cost ratio |
| Movable | 25 | 80 | 5 | N/A | 16 | 30 | 3 | 6.94 | 50523 | 667411 | 56.40 | 0.00 | 56.40 | 26.23 | 0.00 | 0.00 | 82.63 | N/A |
| Movable | 45 | 80 | 5 | 50 | 8 | 15 | 3 | 6.94 | 25262 | 333706 | 86.32 | 26.93 | 113.25 | 13.11 | 13.11 | 0.12 | 126.36 | 0.23 |
| Movable | 55 | 80 | 5 | 65 | 5 | 10 | 2 | 6.94 | 16240 | 214525 | 101.28 | 40.40 | 141.68 | 8.43 | 17.80 | 0.13 | 150.10 | 0.21 |
| Fixed | 65 | 80 | 5 | 100 | 0 | 0 | 0 | 0 | 0 | 0 | 33.51 | 53.86 | 87.37 | 0.00 | 26.23 | 0.30 | 87.37 | 0.85 |

Table 3.37 Bridge Replacement Evaluation Matrix - Bridge 150027 (Average Method)

| Bridge Type | Bridge Geometrics | | | Bridge Openings | | | | | | | Initial Costs | | | User Delay Costs | | | Life Cycle Cost | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Height (ft) | Width (ft) | Grade (%) | Estimated Percentage Reduction | Predicted Average Weekday Openings | Predicted Average Weekend Openings | Predicted Average Peak Hour Openings | Average Delay per opening in Base Year (Vehicle-hrs) | Annual Delay in Base Year (Vehicle-hrs) | Annual Cost of Delay in Base Year ( $) | Construction/ Design ( in MIL $) | R.O.W ( in MIL $) | Total ( in MIL $) | User Delay ( in MIL $) | Savings ( in MIL $) | Benefit/ Cost ratio | Total | Benefit/ Cost ratio |
| Movable | 21 | 80 | 5 | N/A | 24 | 36 | 5 | 5.59 | 55811 | 737257 | 50.42 | 0.00 | 50.42 | 28.97 | 0.00 | 0.00 | 79.39 | N/A |
| Movable | 45 | 80 | 5 | 40 | 8 | 10 | 3 | 5.59 | 17441 | 230393 | 86.32 | 32.32 | 118.64 | 9.05 | 19.92 | 0.17 | 127.69 | 0.29 |
| Movable | 55 | 80 | 5 | 70 | 5 | 5 | 1 | 5.59 | 10174 | 134396 | 101.28 | 45.78 | 147.06 | 5.28 | 23.69 | 0.16 | 152.34 | 0.25 |
| Fixed | 65 | 80 | 5 | 100 | 0 | 0 | 0 | 0 | 0 | 0 | 33.51 | 59.25 | 92.76 | 0.00 | 28.97 | 0.31 | 92.76 | 0.68 |

Table 3.38 Bridge Replacement Evaluation Matrix - Bridge 150050 (Average Method)

| Bridge Type | Bridge Geometrics | | | Bridge Openings | | | | | | | Initial Costs | | | User Delay Costs | | | Life Cycle Cost | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Height (ft) | Width (ft) | Grade (%) | Estimated Percentage Reduction | Predicted Average Weekday Openings | Predicted Average Weekend Openings | Predicted Average Peak Hour Openings | Average Delay per opening in Base Year (Vehicle-hrs) | Annual Delay in Base Year (Vehicle-hrs) | Annual Cost of Delay in Base Year ( $) | Construction/ Design ( in MIL $) | R.O.W ( in MIL $) | Total ( in MIL $) | User Delay ( in MIL $) | Savings ( in MIL $) | Benefit/ Cost ratio | Total | Benefit/ Cost ratio |
| Movable | 23 | 80 | 5 | N/A | 24 | 30 | 3 | 3.98 | 37253 | 492109 | 53.41 | 0.00 | 53.41 | 19.34 | 0.00 | 0.00 | 72.75 | N/A |
| Movable | 45 | 80 | 5 | 30 | 16 | 20 | 3 | 3.98 | 24835 | 328073 | 86.32 | 29.62 | 115.94 | 12.89 | 6.45 | 0.06 | 128.83 | 0.10 |
| Movable | 55 | 80 | 5 | 60 | 8 | 12 | 2 | 3.98 | 13245 | 174972 | 101.28 | 43.09 | 144.37 | 6.88 | 12.46 | 0.09 | 151.24 | 0.14 |
| Fixed | 65 | 80 | 5 | 100 | 0 | 0 | 0 | 0 | 0 | 0 | 33.51 | 56.55 | 90.07 | 0.00 | 19.34 | 0.21 | 90.07 | 0.53 |

## 3.7. Recommendations for Pontis Implementation

Pontis simulates the functional improvement needs of bridges from considerations of functional standards and improvement feasibility. The following types of Improvement models in Pontis are used in establishing the rules that are used in these considerations: Widening; Raising; Strengthening; and Replacement.

When a bridge has low or insufficient load capacity or under clearance certain trucks traveling on or under the bridge are forced to detour and thus incurring additional operating and travel time costs. For the benefit of strengthening Pontis calculates the savings in terms of vehicle operating and travel time costs if a functional improvement action is taken to rectify the deficiency of insufficient bridge load capacity. For the benefit of raising Pontis calculates the savings in terms of vehicle operating and travel time costs if a functional improvement is taken to rectify the deficiency of inadequate or low vertical bride under clearance. Road users are theoretically subject to higher accident risks on narrower travel ways. To evaluate a functional improvement or replacement that corrects the deficiency of narrow travel lanes, the user cost model estimates the benefit of the improvement in terms of savings in accidents or accident risks.

Movable bridges like all other standard bridges can be analyzed for the benefits of strengthening and widening. However, application of the existing Pontis model to movable bridges will yield zero or no benefit for any bridge raising, considering vehicular traffic on the routes under the bridge. The main condition for bridge raising is based on vertical deficiency on the bridges' under routes and the resulting effect on trucks. Movable bridge openings to vessels using the waterway, on the other hand, affect all vehicles traveling on the route over them, irrespective of their weights. The openings are, due to the low or insufficient under-clearance available to vessels traveling on the waterways over which the bridges span. Vehicles are forced to queue up during bridge openings and these results in extra travel times. The benefit of raising or replacing a movable bridge, improves the deficiency of the low navigable vertical clearance. This benefit can therefore be estimated in terms of the user cost savings that may result from reduced delays to vehicles passing over the bridge.

Analysis of improvements to bridges can generally described as the following sequence of operations: Modeling of the roadway; Evaluation of the width, strength, and under clearance deficiencies; Evaluation of the feasibility of the required improvements; Calculation of the improvement cost; and Calculation of the improvement benefit.

Based on the unique functional attributes of movable bridges, the following recommendations are made for the Pontis simulation of the functional needs:

### 3.7.1 Roadway Performance

Roadway performance modeling for movable bridges should follow the existing logic in Pontis. For the traffic volume estimation for a given analysis year, Pontis uses a non-linear function fit between two ADT points: observed and future.

$$ADT\ (Y) = ADT\ (Y_0) *[ADT\ (Y_1) /ADT\ (Y_0)]/\ ^{\{(Y-Y_0)/(Y_1-Y_0)\}} \qquad (3.17)$$

Where: ADT ($Y_0$) is the most recent actual traffic volume estimate (NBI item 29)
$Y_0$ is the year of the most recent traffic volume estimate (NBI item 30)

ADT ($Y_1$) is the forecast future traffic volume (NBI item 114)
$Y_1$ is the year of forecast traffic volume (NBI item 115)
Y is the year of the simulation

### 3.7.2 Improvement Feasibility of Movable Bridges

A study of the current Pontis criteria for determining the feasibility of improvements to bridges revealed that the design codes for the three types of movable bridges; 15 (Lift), 16 (Bascule) and 17 (Swing) are not eligible for bridge raising or widening improvements. Feasibility of raising a bridge is based on the design code of its main span (NBI Item 43B) being less than 11 and also not being equal to 7. This criterion may need to be modified, because analyses can now be done using the results of this study, to evaluate feasible raising alternative projects on movable bridges, assuming the approach spans would also be raised appropriately.

### 3.7.3 Widening Improvement

Thompson et al. (1999) developed a user cost model for Florida, addressing widening improvements.

### 3.7.4 Raising Improvement

The existing Pontis model considers the raising of a bridge based on the following two conditions:

- The service type on the bridge belongs to the list for which raising can be an option; and
- There is a vertical clearance deficiency on at least one under-roadway of the bridge.

The functional classes for which the first condition makes bridges not eligible for consideration are classes 9 and 19. An analysis of the distribution of roadway functional classes for movable bridges showed that most of Florida's movable bridge would satisfy this condition, with only 5% in functional classes 9 and 19. The second condition would however not be directly applicable to movable bridges because the routes under the bridge are waterways used by vessels rather than vehicles. Vertical clearance deficiency needs to be redefined for movable bridges, using two attributes: vessel height distributions; and required number of daily bridge openings.

However, raising a movable bridge may be impractical. Given the typical expenses expected both in agency costs and user costs to raise a movable bridge, it may not be a feasible economic option. Moreover, the bridge would have to be significantly raised to reduce the number of bridge openings to vessels.

### 3.7.5 Strengthening Improvement

The parameters for strengthening improvement considerations for movable bridges can also be the same as in the existing Pontis models. Pontis considers strengthening of a bridge if the following conditions are met:

- The service type on the bridge belongs to the list for which strengthening can be an option; and
- The load rating on at least one of the on-roadway of the bridge is below the legal standard

Pontis assumes that user benefits of strengthening incur through the reduction of the number of vehicles that have to bypass the bridge. The fraction of detouring trucks is estimated from a default

Pontis model for all roadway functional classes. An improvement to this truck weight model based on the roadway functional class has been proposed in chapter two of this report has been.

### 3.7.6 Bridge Replacement

The replacement alternatives will include both moveable bridges and fixed bridges. The benefit of replacing a movable bridge, with a higher movable bridge, as discussed earlier, is assumed to incur through the reduction of delays to road users during bridge openings to vessels. In addition, there may be a need to strengthen the bridge; Pontis already has the capability to handle this. A user delay cost model is proposed for implementation into Pontis, described as follows.

Estimation of average daily vessel traffic (ADVT) on the waterway for any year of analysis is modeled by the non-linear function below. A growth rate range of 1-3% is recommended for the analysis.

$$ADVT\ (Y) = ADVT\ (Y_0)\ (1+ r)^{(Y-Y_0)} \qquad (3.18)$$

Where: ADVT ($Y_0$) is the most recent actual traffic volume estimate
$Y_0$ is the year of the most recent traffic volume estimate
r is the growth rate expressed as a fraction of 100.

Y is the year of the simulation

User delay parameters in terms of roadway and vehicular traffic parameters required for the estimation of delay include:

- The Average daily traffic (ADT) on the bridge for the year of analysis, which is estimated under the roadway performance
- The number of lanes on the roadway (NBI Item 028A)
- Saturation flow rate on roadway, published per lane, available from the FDOT Generalized Level of Service (LOS) tables.

The following vessel traffic parameters need to be recorded and updated in the Pontis database for the estimation of user delay:

- Average daily vessel traffic (ADVT)
- Vessel height distribution (maximum and average)
- Average daily bridge openings (ADBO)

Given the bridge vertical under clearance of a proposed improvement or replacement, the prediction of the expected openings can be done using a probability model developed from the distribution of vessel height data collected during the survey. As shown in equation 3.19 below and demonstrated in Figure 3.44, the predicted proportion is applied to the most recent average daily bridge openings (ADBO) value available in the database to estimate the expected number of openings for the replacement bridge.

$$B_o = [ADBO]*\ [1.8457\ + -\ 0.0669x + 0.0008x^2 - 3*10^{-6}\, x^3] \qquad (3.19)$$

Where:
$B_o$ is the predicted number of bridge openings
*x* is the proposed bridge's under clearance (feet)

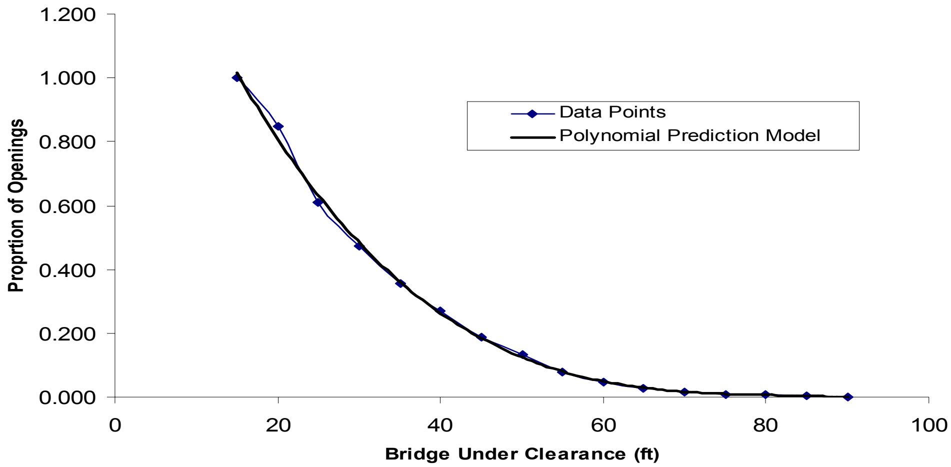


Figure 3.44 Prediction Model for Reduction in Bridge Openings

The total duration of roadway blockage, *t*, and the average number of vessels at each bridge opening can be calculated as follows:

$$N_{avg} = ADVT\ (Y) / B_o \qquad (3.20)$$

$$t = N_{avg} * S \qquad (3.21)$$

Where:
ADVT (Y) = projected vessel traffic volume for year Y
$B_o$ = the predicted number of bridge openings
$N_{avg}$ is the average number of vessels at each bridge opening
*S* is the average service time per vessel,

The recommended value for *S* based on the data obtained from the recent the vessel survey is 1.42 minutes or 85 seconds. It must be noted here that the study reported two other methods of estimating the total roadway blockage time.

1. One approach was to set default roadway blockage duration time *t* at 5 minutes for $N_{avg} \leq 5$; and the use of equation 5 based on the *S* default value when $N_{avg} > 5$.
2. Another approach was the use of a developed power function model to predict the total roadway blockage duration based on the number of vessels requiring passage.

For a given bridge the total delay to vehicles at each opening is calculated as:

$$D = 0.5 * q * t^2 * s / (s - q) \qquad (3.22)$$

Where:

q = average arrival rate of traffic (vehicles per minute) upstream of bottleneck
s = saturation flow rate or capacity of uninterrupted flow (vehicles per minute)
t = duration of bridge opening, in minutes

An analysis of existing vehicular traffic data and observations made at the bridge sites revealed that approximately about 75% of vehicular traffic occur during the daylight period of between 7 am to 7 pm. The hourly arrival rate of vehicular traffic at the bridge, Q, is therefore calculated as:

$$Q = 0.75 * ADT /12 \tag{3.23}$$

Analysis of existing data, data collected during the recent survey and observations made at the at the study sites and interaction with bridge operators also revealed that even though movable bridges were operated throughout each day, most of bridge openings, approximately 90% of the total openings for the day, occur during the daylight period of between 7 am to 7 pm.
It was therefore concluded that approximately 90% of the daily bridge openings affect approximately 75% of the vehicular traffic. The total daily delay at each bridge, $D_d$, is therefore calculated as:

$$D_d = 0.9 * B_o * D \tag{3.24}$$

The Pontis Cost matrix contains a unit cost parameter *HrDetourCost,* defined as the cost per hour of detour; it can be used in estimating the cost of delay. The total annual user cost due to vehicular delay can be calculated as follows:

$$U_{delay} = 365.25 * D_d * \text{HrDetour Cost} \tag{3.25}$$

The total annual user benefit of replacement of moveable bridges is therefore, basically the sum of the benefits from strengthening and reduction or elimination of costs due to vehicular delays during bridge openings to vessel traffic.

The cost of replacing a movable bridge can be calculated based on the deck area and unit replacement costs for a bridge of the same functional class. The existing Pontis Cost Matrix can be used with relevant estimates made for the bridges' movable span, i.e. if the replacement option is also a movable bridge. A complete bridge replacement cost should include but not limited to the cost of design, construction, additional right-of-way costs where applicable and Construction and Engineering Inspection costs. Life cycle cost calculations will be necessary, especially at project level analysis.

## 3.8. Estimate of Network User Delay Costs

Roadway vehicle delay costs were computed for each of the 147 movable bridges in the 2001 Florida Inventory data using the above-described methodology. The results of the analysis for year 2002 AADT and a 20-year projected AADT are shown in the figures below. The average method of user delay analysis was used; with default inputs of 5 minutes of bridge opening duration and 24 daily openings for all the bridge sites. The 2002 annual cost of user delay per bridge site ranged from $377 at bridge no. 360800 with an AADT of 15 vehicles/day to about $1,628,000 at bridge no. 720022 with an AADT of 4,1500. The average per bridge site was $ 475,000. The range for the year 2020 was from $1,278 to over $5,000,000 and the expected average per bridge site is $1,068,000.The result of this statewide network analysis gives an estimated statewide cost of user delay for the present year and in the future, if no bridge is replaced.

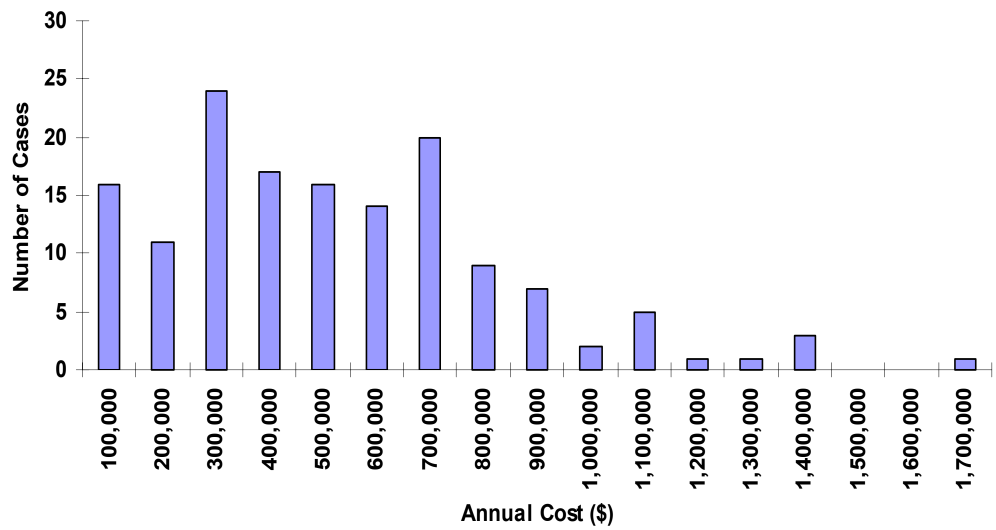


Figure 3.45 Estimated Statewide Annual User Delay Cost - 2002

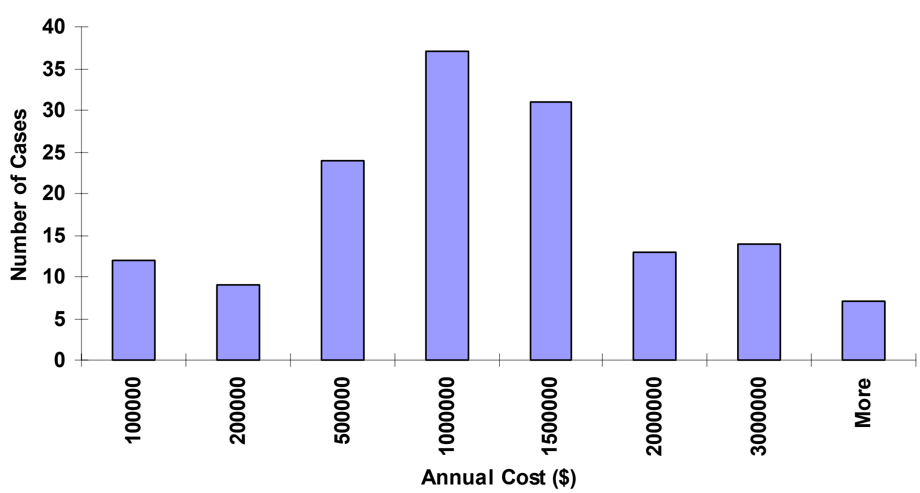


Figure 3.46 Estimated Statewide Annual User Delay Cost - 2020

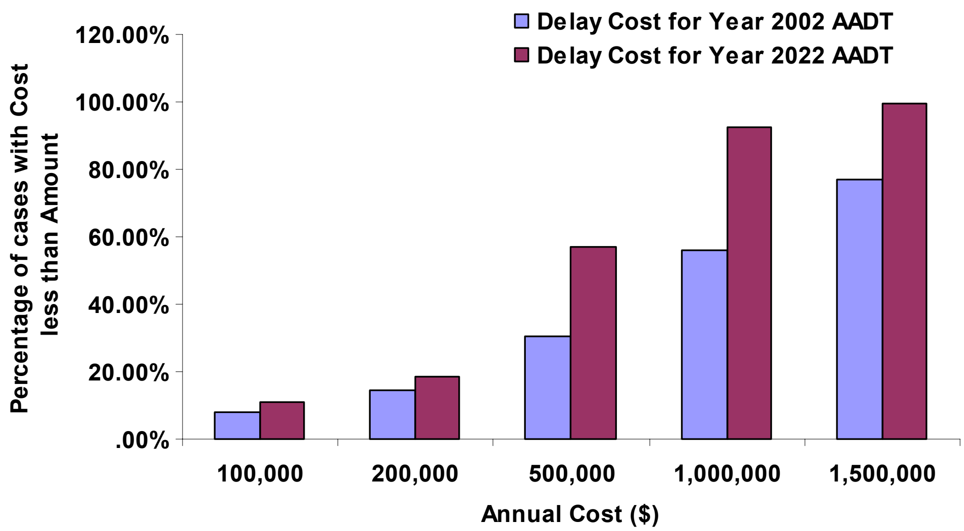


Figure 3.47 Comparison of Estimated Statewide User Delay Costs – Present and Future

## 3.9. Estimate of Network User Costs Including Strengthening

Bridges are generally replaced with the objective of correcting or eliminating strength, under clearance or width deficiencies. Movable bridges do not have under routes and therefore do not present an under clearance problem for the passage of trucks or other tall vehicles. However, the opening of the bridge's movable spans during the passage of tall vessels forces all vehicles, irrespective of weight, to experience extended delays. Movable bridges therefore have raising needs and in this case it is due to low navigable vertical under clearance for the passage of vessels when the bridge is in a closed position. A replacement model for a movable bridge can therefore be aimed at addressing both strengthening and raising needs; strengthening, to accommodate trucks which may be forced to detour onto alternate and usually longer routes; and raising, to accommodate all or at least a greater number of vessels when in the closed position, thereby reducing delay experienced by motorists.

The user benefit of replacement of a movable bridge will therefore be estimated as the sum of the benefits from reduced costs of detours incurred on the bridge's roadway due to insufficient load rating and the reduced delays due to the opening of the bridge for the passage of vessels. The fraction of truck traffic that has to make detours on the movable bridge can be estimated by comparing the adjusted values of bridge's operating rating (NBI Item 64) with developed truck weight distributions for the various functional classes. Details on estimating user benefits due to strengthening have been presented earlier in section 2 of this report. Pontis currently uses a default general piecewise truck weight distribution for all roadway functional classes. The resulting user cost benefit of strengthening the bridge can then be calculated using the following NBI items: bypass/detour length (NBI Item 19), the percentage of daily truck traffic (NBI Item 109) and the total average daily traffic (ADT) (NBI Item 29). Required user input variables are the detour speed on each functional class, the vehicle operating cost and the unit cost of truck travel time.

For the reduced delays an 'average" queue model, which reflects the average effect of bridge opening on the daily traffic is proposed for use. Existing NBI items used as variables for the model are the ADT (NBI Item 29) and number of bridge roadway lanes (NBI Item 28A). The fraction of ADT used in the queue model will be the difference of the total roadway ADT and the estimated number of detoured trucks. Required user inputs will be the estimated average number of daily openings of the bridge, the average duration of each opening and the unit average cost of vehicle travel time. The unit average cost of vehicle travel time can be obtained by weighting the cost of travel time for passenger-cars and the cost of travel time for trucks by their respective percentages in the traffic stream on the roadway. The model estimates the number of affected vehicles by each opening of the bridge and the total delay in vehicle-hours based on which the resulting user delay cost is calculated.

As described in section 2 "Truck Height and Weight Models " of this report, a model for bridge strengthening needs will require a sub-model for the distribution of truck weight in the truck traffic stream. This sub-model will help estimate the fraction of trucks that may have to detour due to bridge strength deficiencies. Two truck weight models: a 3-segment piecewise linear model (Pontis) and a piecewise curvilinear model, which were developed from truck weight data for the non-interstate roadways, were used in determining the fraction of detouring trucks on each bridge. Almost all moveable bridges in Florida are on non-interstate roadways. For comparison, the default Pontis piecewise linear truck weight model was also used.

These three models, the existing and developed, were used in determining the strengthening needs and the corresponding costs for Florida's movable bridges. The equations used for the truck weight models are summarized in following table and previously in section 2 of this report .

Table 3.39 Default Pontis Piecewise Linear Model

| Functional Class | Bridge Operating Rating Range (Lbs) | Fraction of Detouring Trucks (NB. X=Truck weight in Tons) |
|---|---|---|
| All Classes | <5065 | 1 |
| | 5065<x<39636 | 1-0.031576*(x-2.3) |
| | 39636<x<90282 | 0.50425-0.02192*(x-18) |
| | >90282 | 0 |

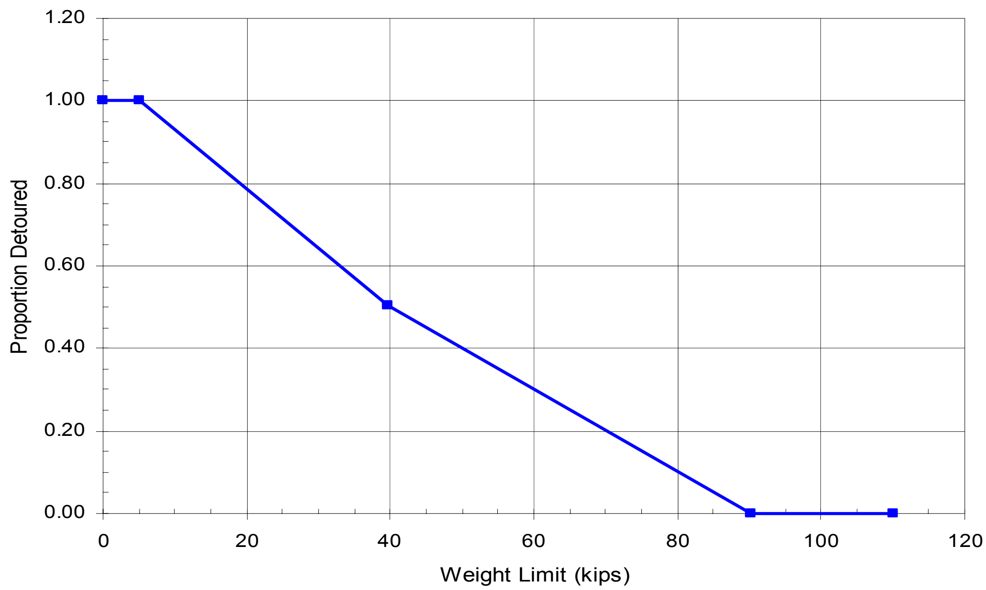


Figure 3.48 Default Pontis Piecewise Truck Weight Model

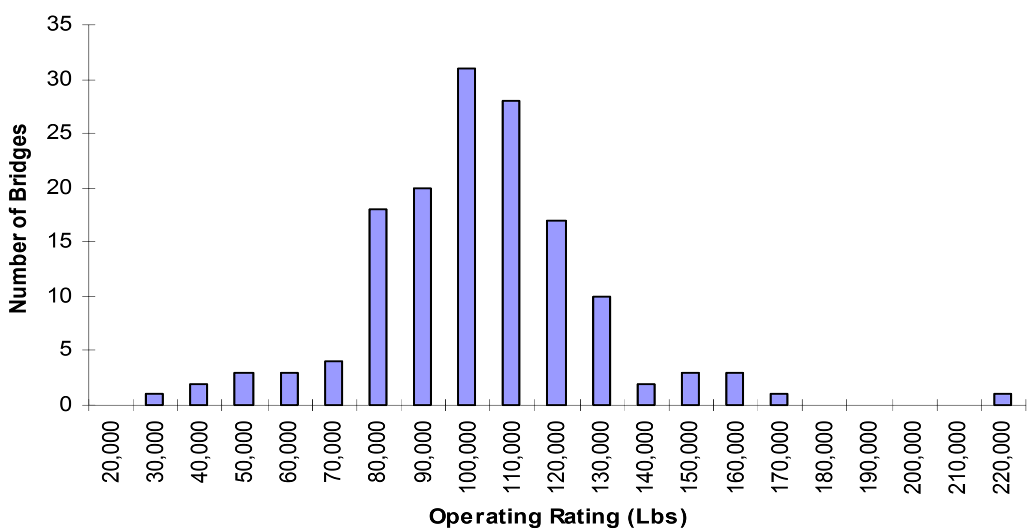


Figure 3.49 Distribution of Moveable Bridges by Operating ratings

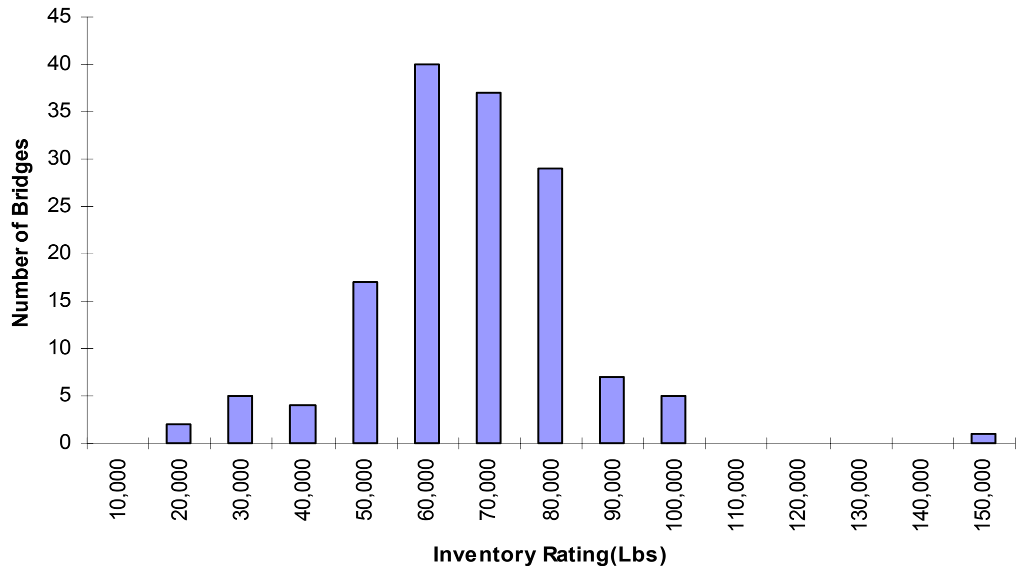


Figure 3.50 Distribution of Moveable Bridges by Inventory ratings

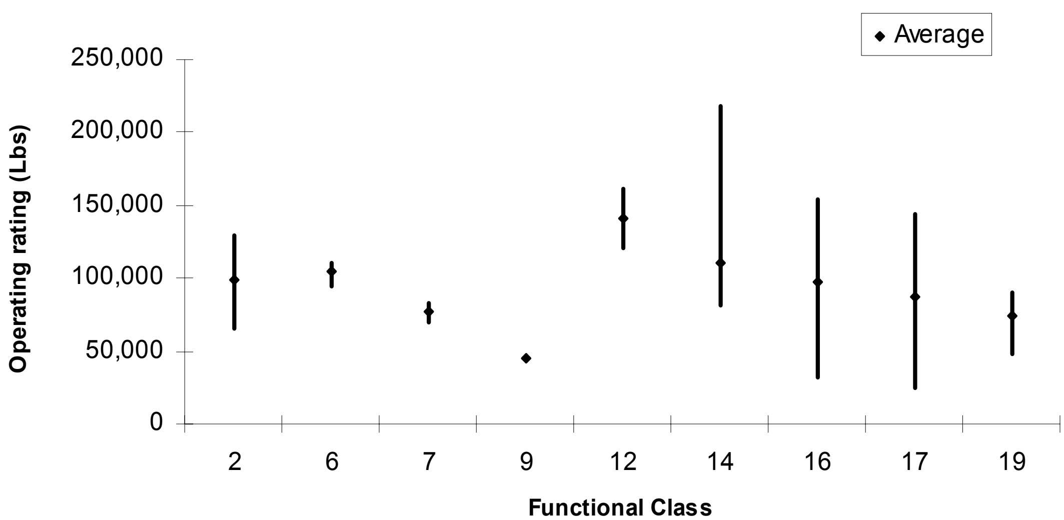


Figure 3.51 Variation in Operating Rating by Roadway Functional Class

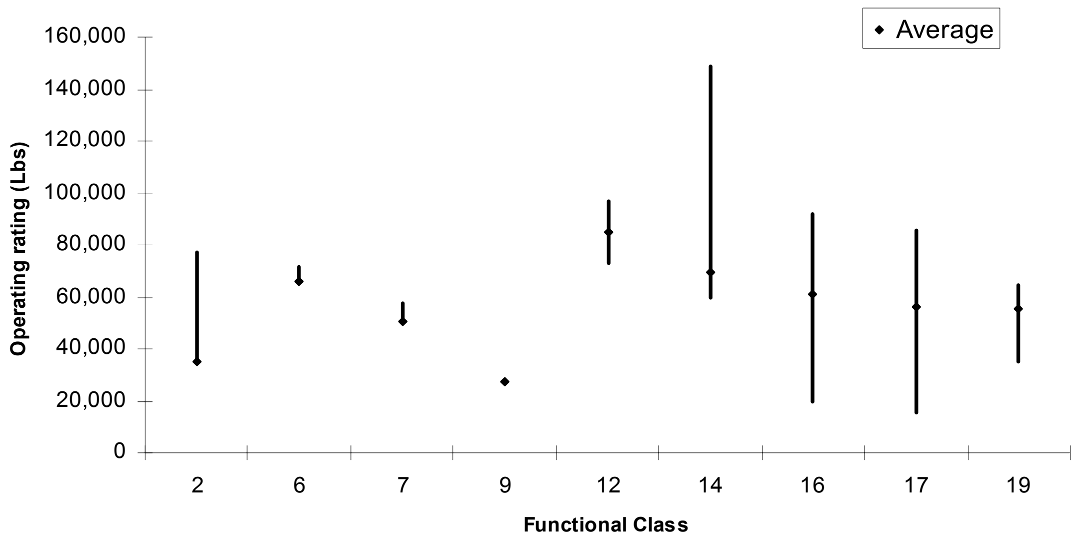


Figure 3.52 Variation in Inventory Rating by Roadway Functional Class

From the distribution of operating ratings of the State's movable bridges, approximately 20% of movable bridges will require strengthening to accommodate trucks with weights up to the Legal Gross Weight limit of 80,000 Lbs. The benefit of strengthening obtained for each bridge may have varied for each truck weight distribution model due to the different break points of the piecewise curves used in each of the models. The values obtained from the existing Pontis model was relatively high for all the functional classes because the same break points were used for all the classes, while they varied for the other two models. However, irrespective of the weight distribution model the estimated benefits were most significant for bridges on roadways of functional classes 16 and 17, which make up over 50% of the total number of movable bridges.

For the total benefit of replacement, the most significant amounts were for bridges on functional classes 14, 16 and 17, which together make up over 80% of the total number of movable bridges in the State of Florida.

It must be noted that for almost all the functional classes approximately 90% or more of the total benefit will be contributed by the savings from reduced or eliminated delays due to bridge openings.

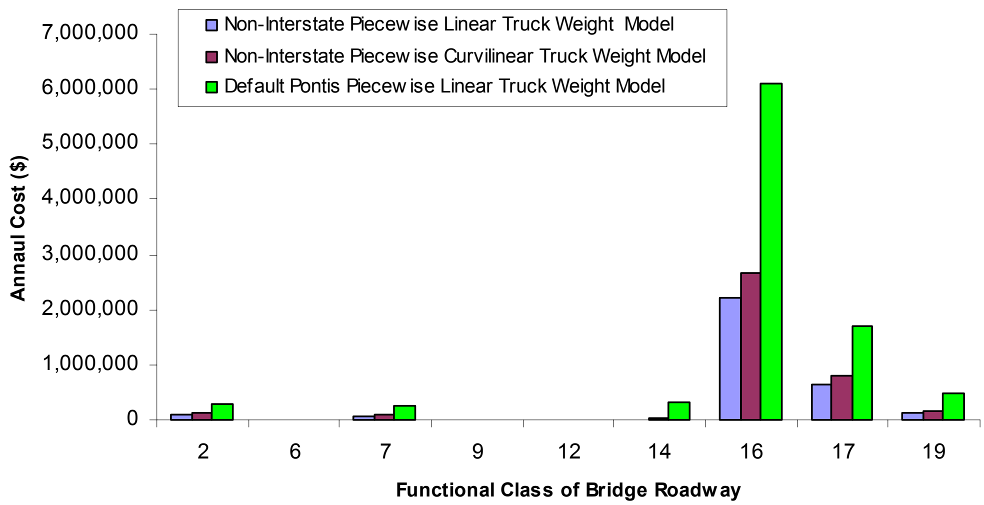


Figure 3.53 Estimate of Network User Cost of Strengthening by Roadway Functional Class

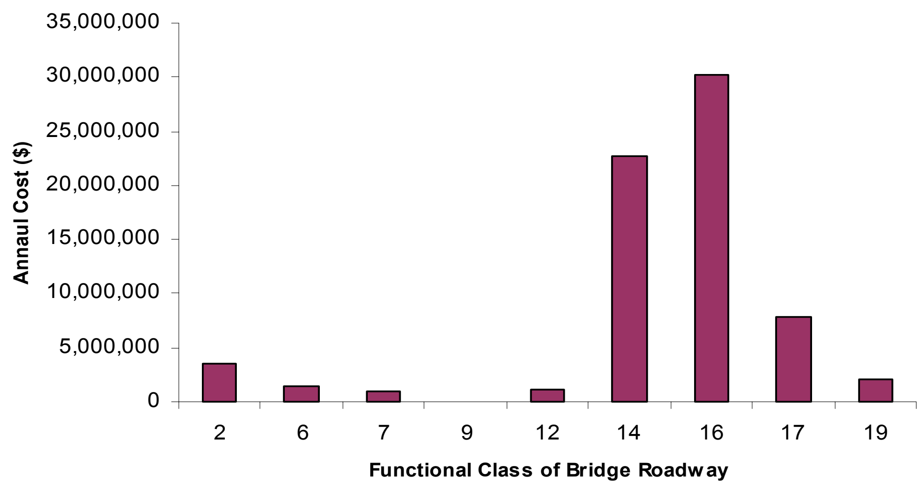


Figure 3.54 Estimate of Network User Delay Cost by Roadway Functional Class

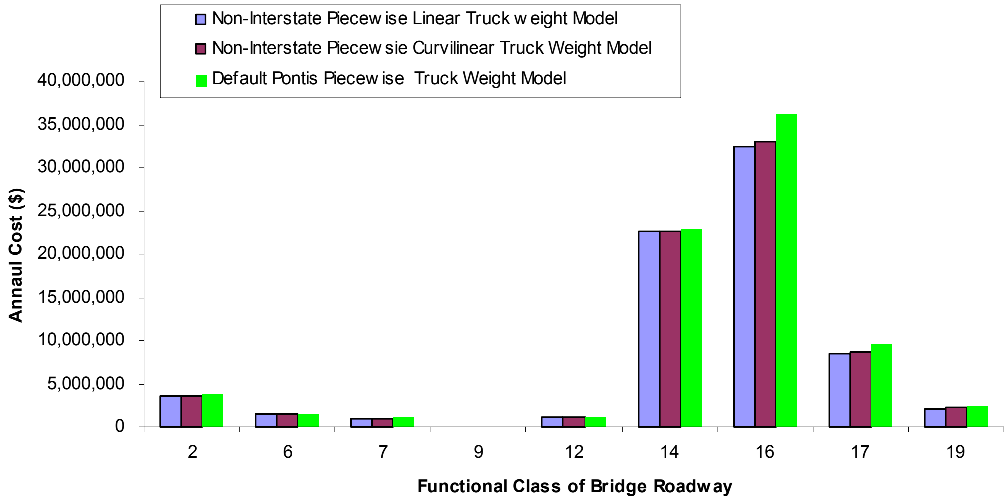


Figure 3.55 Comparison of Total Bridge Replacement Benefits

Table 3.40 Bridge Replacement Benefits

| Functional Class | Annual Benefit of Replacement Non-Interstate Linear | Non-Interstate Curvilinear | Default Pontis Linear | User Delay Cost | Benefit of Strengthening Non-Interstate Linear | Non-Interstate Curvilinear | Default Pontis Linear |
|---|---|---|---|---|---|---|---|
| 2 | 3,593,557 | 3,609,201 | 3,757,486 | 3,487,222 | 106,335 | 121,979 | 273,848 |
| 6 | 1,462,184 | 1,462,184 | 1,462,184 | 1,459,805 | 2,379 | 2,379 | 0 |
| 7 | 990,948 | 1,018,937 | 1,185,981 | 920,558 | 70,390 | 98,378 | 267,010 |
| 9 | 442 | 452 | 477 | 377 | 66 | 76 | 102 |
| 12 | 1,166,075 | 1,166,075 | 1,166,075 | 1,166,075 | 0 | 0 | 0 |
| 14 | 22,621,417 | 22,659,435 | 22,920,255 | 22,618,172 | 3,245 | 41,263 | 311,118 |
| 16 | 32,478,037 | 32,930,134 | 36,282,742 | 30,261,323 | 2,216,714 | 2,668,811 | 6,092,001 |
| 17 | 8,492,129 | 8,654,836 | 9,540,390 | 7,855,862 | 636,267 | 798,974 | 1,696,098 |
| 19 | 2,152,255 | 2,204,614 | 2,516,619 | 2,038,364 | 113,891 | 166,250 | 490,061 |

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# Appendix B: Calibration of Truck Height Measuring Equipment

Table B.1: Calibration of Impulse Laser Rangefinder

| **DATE:** 10/25/02 | | TEMP: 72 F | HUMIDITY: 93% | | |
|---|---|---|---|---|---|
| **Reading No.** | **Distance from Object (ft)** | **Fixed height (ft)** | **Measured Height (ft)** | **Squared Height ($ft^2$)** | **Deviation (ft.)** |
| 1 | 18.50 | 10.23 | 10.31 | 106.30 | 0.08 |
| 2 | 22.41 | 10.23 | 10.32 | 106.50 | 0.09 |
| 3 | 30.20 | 10.23 | 10.23 | 104.65 | 0.00 |
| 4 | 36.48 | 10.23 | 10.34 | 106.92 | 0.11 |
| 5 | 41.97 | 10.23 | 10.21 | 104.24 | -0.02 |
| 6 | 39.44 | 10.23 | 10.08 | 101.61 | -0.15 |
| 7 | 38.11 | 10.23 | 10.13 | 102.62 | -0.10 |
| 8 | 32.98 | 10.23 | 10.38 | 107.74 | 0.15 |
| 9 | 32.55 | 10.23 | 10.16 | 103.23 | -0.07 |
| 10 | 21.85 | 10.23 | 10.17 | 103.43 | -0.06 |
| 11 | 17.39 | 10.23 | 10.37 | 107.54 | 0.14 |
| 12 | 26.61 | 10.23 | 10.24 | 104.86 | 0.01 |
| 13 | 39.14 | 10.23 | 10.32 | 106.50 | 0.09 |
| 14 | 49.31 | 10.23 | 10.39 | 107.95 | 0.16 |
| 15 | 50.92 | 10.23 | 10.40 | 108.16 | 0.17 |
| 16 | 29.39 | 10.23 | 10.36 | 107.33 | 0.13 |
| 17 | 34.08 | 10.23 | 10.23 | 104.65 | 0.00 |
| 18 | 35.61 | 10.23 | 10.25 | 105.06 | 0.02 |
| 19 | 38.19 | 10.23 | 10.20 | 104.04 | -0.03 |
| 20 | 39.70 | 10.23 | 10.40 | 108.16 | 0.17 |
| 21 | 41.48 | 10.23 | 10.07 | 101.40 | -0.16 |
| 22 | 72.01 | 10.23 | 10.24 | 104.86 | 0.01 |
| 23 | 77.27 | 10.23 | 10.34 | 106.92 | 0.11 |
| 24 | 90.19 | 10.23 | 10.25 | 105.06 | 0.02 |
| 25 | 93.76 | 10.23 | 10.47 | 109.62 | 0.24 |
| 26 | 96.23 | 10.23 | 10.33 | 106.71 | 0.10 |
| 27 | 89.92 | 10.23 | 10.30 | 106.09 | 0.07 |
| 28 | 30.29 | 10.23 | 10.52 | 110.67 | 0.29 |
| 29 | 49.37 | 10.23 | 10.24 | 104.86 | 0.01 |
| 30 | 48.98 | 10.23 | 10.40 | 108.16 | 0.17 |
| 31 | 44.97 | 10.23 | 10.21 | 104.24 | -0.02 |
| 32 | 37.44 | 10.23 | 10.34 | 106.92 | 0.11 |
| 33 | 31.97 | 10.23 | 10.31 | 106.30 | 0.08 |
| 34 | 29.02 | 10.23 | 10.30 | 106.09 | 0.07 |
| 35 | 28.02 | 10.23 | 10.30 | 106.09 | 0.07 |
| 36 | 20.65 | 10.23 | 10.23 | 104.65 | 0.00 |
| 37 | 26.15 | 10.23 | 10.34 | 106.92 | 0.11 |
| 38 | 24.85 | 10.23 | 10.44 | 108.99 | 0.21 |
| 39 | 38.10 | 10.23 | 10.46 | 109.41 | 0.23 |

Table B.1: Calibration of Impulse Laser Rangefinder **(Cont'd)**

| Reading No. | Distance from Object (ft) | Fixed height (ft) | Measured Height (ft) | Squared Height ($ft^2$) | Deviation (ft.) |
|---|---|---|---|---|---|
| 40 | 40.45 | 10.23 | 10.34 | 106.92 | 0.11 |
| 41 | 42.14 | 10.23 | 10.32 | 106.50 | 0.09 |
| 42 | 50.16 | 10.23 | 10.30 | 106.09 | 0.07 |
| 43 | 50.83 | 10.23 | 10.12 | 102.41 | -0.11 |
| 44 | 55.39 | 10.23 | 10.44 | 108.99 | 0.21 |
| 45 | 35.54 | 10.23 | 10.24 | 104.86 | 0.01 |
| 46 | 35.47 | 10.23 | 10.23 | 104.65 | 0.00 |
| 47 | 32.75 | 10.23 | 10.23 | 104.65 | 0.00 |
| 48 | 32.75 | 10.23 | 10.30 | 106.09 | 0.07 |
| 49 | 24.00 | 10.23 | 10.40 | 108.16 | 0.17 |
| 50 | 19.12 | 10.23 | 10.31 | 106.30 | 0.08 |

Height of Fixed Object = 10.23 ft.
Mean of Instrument Readings = 10.296 ft.
Root Mean Square (RMS) of Instrument Readings = 10.297 ft.
Standard Deviation of Instrument Readings = 0.099 ft.

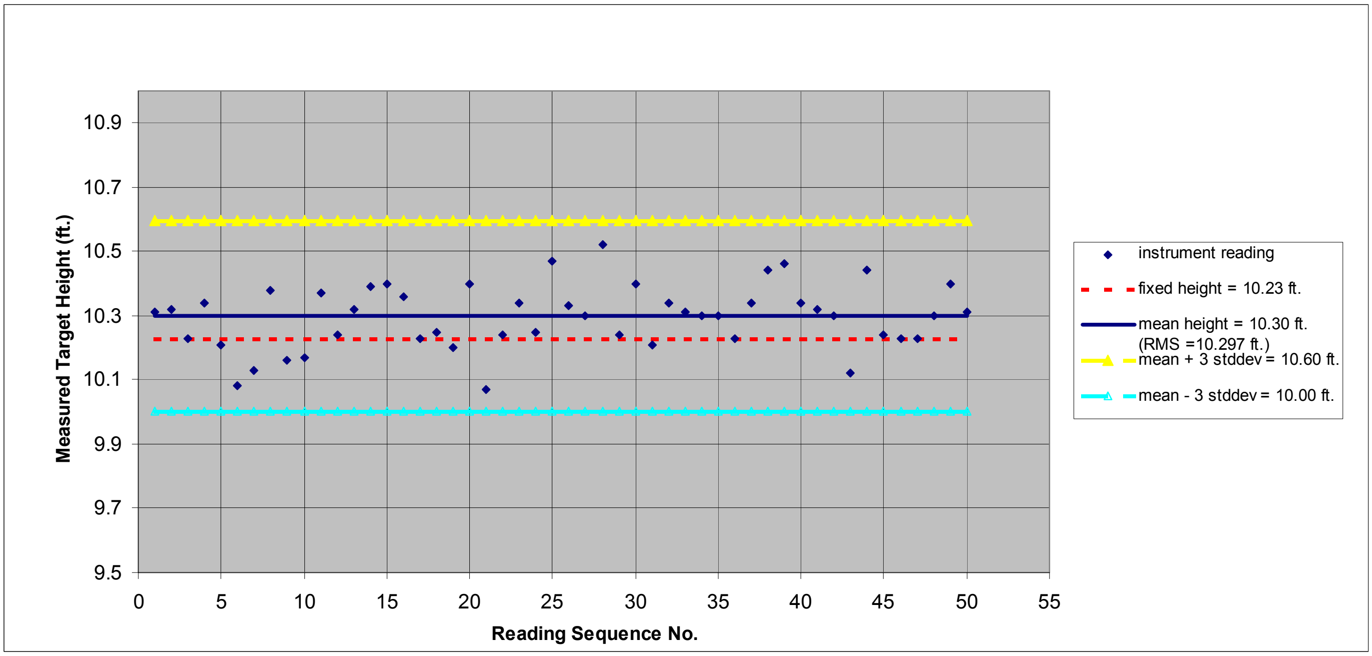



Figure B.1: Calibration of Impulse Laser Rangefinder Based on Reading Sequence

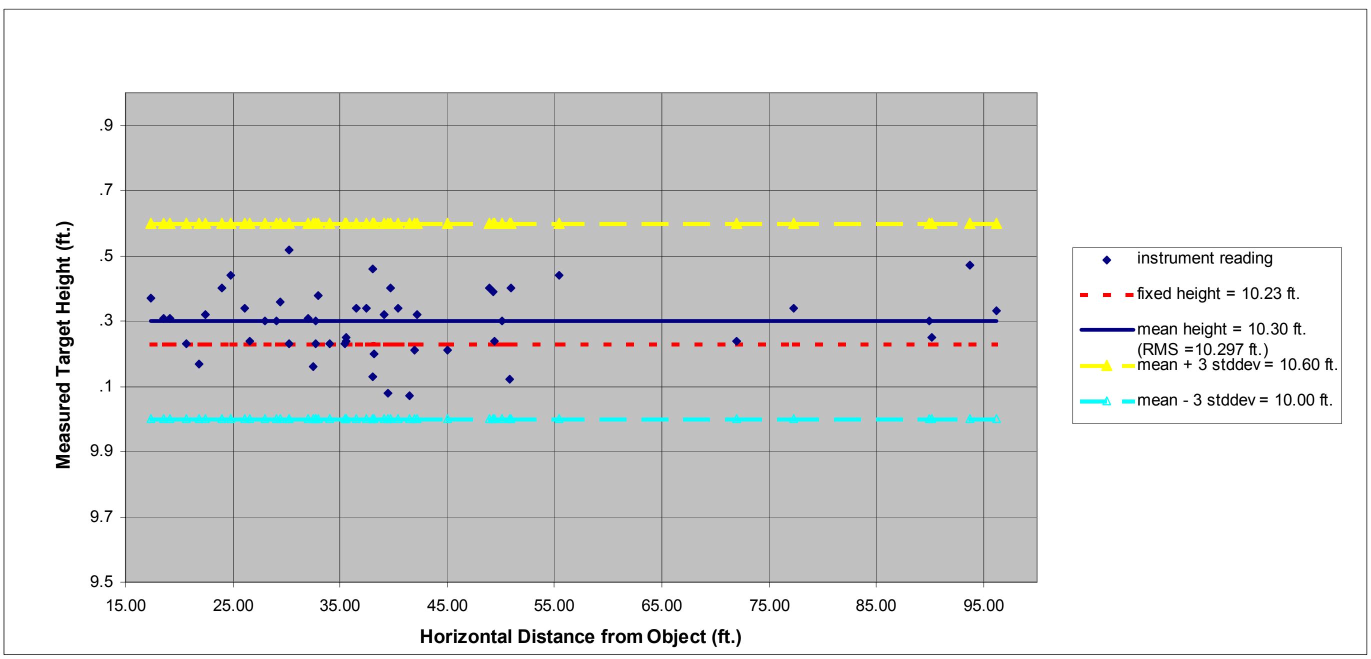



Figure B.2: Calibration of Impulse Laser Rangefinder Based on Horizontal Distance from Object

The following is a sample of what the sti_data.txt file will look like. Each line is terminated with a <CR><LF> (HEX 0D, and HEX 0A) or (decimal 13, and decimal 10).

The fields are separated by commas. The fields are as follows:

MM/DD/YYYY,hh:mm:ss,CC,HHHH,A,GGGGG,W1,W2…WN<CR><LF>

Where: MM – Month
DD – Day of the month
YYYY – Year
hh – Hour of the day (24 hour format)
mm – Minute
ss – Second
CC – Vehicle Class
HHHH – Vehicle height in tenths of inches. For example, 1620 would be 162.0 inches (or 13'6.0").
A – Number of Axles
GGGGG – Gross Weight
W1 – Weight of Axle 1
W2 – Weight of Axle 2
WN – Weight of Axle N (last axle)

```
11/04/2002,13:12:20,09,1620,5,53400,11900,10300,8100,10700,12400
11/04/2002,13:12:22,09,1620,5,53400,11900,10300,8100,10700,12400
11/04/2002,13:12:22,09,1680,5,66000,12100,17100,15000,10100,11700
11/04/2002,13:12:25,11,1200,5,74000,10900,15700,15500,15800,16100
11/04/2002,13:12:26,09,1700,5,112000,12100,19000,17000,32000,31900
11/04/2002,13:12:28,11,1640,5,67800,10200,14200,16400,13000,14000
11/04/2002,13:12:29,09,1700,5,112000,12100,19000,17000,32000,31900
11/04/2002,13:12:29,09,1620,5,53400,11900,10300,8100,10700,12400
11/04/2002,13:12:30,09,1680,5,66000,12100,17100,15000,10100,11700
11/04/2002,13:12:31,09,1620,5,53400,11900,10300,8100,10700,12400
11/04/2002,13:12:32,09,1620,5,53400,11900,10300,8100,10700,12400
11/04/2002,13:12:32,09,1620,5,53400,11900,10300,8100,10700,12400
```

Figure B3. Sample Data File and Dictionary from STI Vehicle Scanner as Programmed into the Mettler Toledo WIM System

Table B.2. File Dictionary for FDOT Truck Weight Historical Data

| | | | | |
|---|---|---|---|---|
| TAG | TRK | 1 | - | 3 |
| CO | County Number | 4 | - | 5 |
| STAT | Station Number | 6 | - | 9 |
| | Filler | 10 | - | 10 |
| L | Lane Number | 11 | - | 11 |
| | Filler | 12 | - | 12 |
| YY | Survey Year | 13 | - | 14 |
| MM | Survey Month | 15 | - | 16 |
| DD | Survey Day | 17 | - | 18 |
| HR | Survey Hour | 19 | - | 20 |
| MN | Survey Minute | 21 | - | 22 |
| SC | Survey Second | 23 | - | 24 |
| VEHNO | Vehicle Number | 25 | - | 29 |
| CL | Vehicle Class (Scheme "F") Code | 30 | - | 31 |
| VEHTYP | Vehicle Type (optional) | 32 | - | 37 |
| VIO | Violation Code (default is 000) | 38 | - | 40 |
| SPD | Speed of Vehicle (in MPH) | 41 | - | 43 |
| LENGTH | Length of Vehicle From Bumper to Bumper (format 99.99 decimal implied) | 44 | - | 48 |
| GROSWT | Gross Weight of Vehicle (in lbs) | 49 | - | 54 |
| | Filler | 55 | - | 55 |
| LAXL1 | Left Axle 1 Weight (in lbs) | 56 | - | 60 |
| RAXL1 | Right Axle 1 Weight (in lbs) | 61 | - | 65 |
| AXLE1 | Total Axle 1 Weight (in lbs) | 66 | - | 70 |
| LAXL2 | Left Axle 2 Weight (in lbs) | 71 | - | 75 |
| RAXL2 | Right Axle 2 Weight (in lbs) | 76 | - | 80 |
| AXLE2 | Total Axle 2 Weight (in lbs) | 81 | - | 85 |
| LAXL3 | Left Axle 3 Weight (in lbs) | 86 | - | 90 |
| RAXL3 | Right Axle 3 Weight (in lbs) | 91 | - | 95 |
| AXLE3 | Total Axle 3 Weight (in lbs) | 96 | - | 100 |
| …. | …. | | | |
| ….. | ….. | | | |
| LAXL9 | Left Axle 9 Weight (in lbs) | 176 | - | 180 |
| RAXL9 | Right Axle 9 Weight (in lbs) | 181 | - | 185 |
| AXLE9 | Total Axle 9 Weight (in lbs) | 186 | - | 190 |
| | Filler | 191 | - | 191 |
| NS | Number of Axle Spaces | 192 | - | 192 |
| NA | Number of Axles | 193 | - | 193 |
| | Filler | 194 | - | 194 |
| WHLB | Wheel base - distance from first to last axle (format 99.99 decimal implied) | 195 | - | 198 |
| ASP1 | Axle Space 1-2 (format 99.99 decimal implied) | 199 | - | 202 |
| ASP2 | Axle Space 2-3 (format 99.99 decimal implied) | 203 | - | 206 |
| ASP3 | Axle Space 3-4 (format 99.99 decimal implied) | 207 | - | 210 |
| ASP4 | Axle Space 4-5 (format 99.99 decimal implied) | 211 | - | 214 |
| ASP5 | Axle Space 5-6 (format 99.99 decimal implied) | 215 | - | 218 |
| ASP6 | Axle Space 6-7 (format 99.99 decimal implied) | 219 | - | 222 |
| ASP7 | Axle Space 7-8 (format 99.99 decimal implied) | 223 | - | 226 |
| ASP8 | Axle Space 8-9 (format 99.99 decimal implied) | 227 | - | 230 |

**Table B.3. Raw Data for Simultaneous Calibration of Impulse Laser Rangefinder and STI Scanner (White Springs Station)***

| DATA AT STATIC WEIGH SCALE | | | | | | STI SCANNER DATA | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Reading Sequence | Time Stamp | Veh. Class | Range Finder Reading (in) | Staff Reading (in) | Static Scale Weight (lb) | Time Stamp | Veh. Class | STI Reading (in) | No. Of Axles | Weight At Wim (lb) | COMMENTS |
| 1 | 12:32 | 3S2 | 164.40 | 154 | 76900 | 12:31:40 | 9 | 158.3 | 5 | 77900 | |
| 2 | N/A | 3S2 | 162.48 | 155 | | | | | | | No STI Hard copy; to match STI data. |
| 3 | 12:36 | 3S2 | 158.52 | 158.5 | 73700 | 12:35:12 | 9 | 160.5 | 5 | 73800 | |
| 4 | 12:41 | 3S2 | 154.56 | 155.5 | 76260 | 12:41:02 | 9 | 153.8 | 5 | 79600 | |
| 5 | 12:42 | 3S3 | 156.36 | 157 | 90780 | 12:41:16 | 10 | 154.5 | 6 | 90600 | |
| 6 | 12:44 | 3S2 | 153.72 | 157.5 | 78360 | 12:41:58 | 9 | 155.3 | 5 | 78900 | |
| 7 | 12:46 | 3S2 | 127.56 | 127.5 | 78060 | 12:45:05 | 9 | 138 | 5 | 75900 | Staff could not measure max. pt (Exhaust). |
| 8 | 12:47 | 3S2 | 160.92 | 159 | 79400 | 12:45:34 | 9 | 156.8 | 5 | 82000 | |
| 9 | 12:49 | 3S2 | 161.28 | 161.5 | 69320 | 12:48:04 | 9 | 160.5 | 5 | 66800 | |
| 10 | 12:50 | 3S2 | 157.80 | 160 | 74520 | 12:49:03 | 9 | 159 | 5 | 78800 | |
| 11 | 12:52 | 3S2 | 160.92 | 161 | 77280 | 12:50:50 | 9 | 158.3 | 5 | 78800 | |
| 12 | 12:52 | 3S2 | 157.32 | 158.5 | 73540 | 12:51:19 | 9 | 156.8 | 5 | 75400 | |
| 13 | 12:54 | 3S2 | 160.56 | 158.5 | 78600 | 12:53:22 | 9 | 157.5 | 5 | 79100 | |
| 14 | 12:55 | 3S2 | 158.88 | 160 | 77600 | 12:54:13 | 9 | 157.5 | 5 | 78200 | |
| 15 | N/A | 3S2 | 134.76 | 134 | | 12:55:35 | 9 | 132 | 5 | 72500 | No STI Hard copy-( top of Exhaust ) |
| 16 | 13:01 | 3S2 | 159.12 | 160 | 70120 | 12:59:59 | 9 | 158.3 | 5 | 70800 | |
| 17 | 13:03 | 3S2 | 127.32 | 126 | 79820 | 13:02:11 | 9 | 133.5 | 5 | 67700 | Staff could not measure max. pt (Exhaust). |
| 18 | N/A | 2S3 | 162.00 | 161 | | 13:09:32 | 9 | 158.3 | 5 | 78000 | |
| 19 | N/A | 3S2 | 159.84 | 158 | | 13:09:46 | 9 | 157.5 | 5 | 75000 | |
| 20 | N/A | 3S3 | 127.56 | 0 | | 13:11:14 | 10 | 128.3 | 6 | 46200 | No Staff readings (see pics) |
| 21 | 13:13 | 2S2 | 166.08 | 0 | 35940 | 13:11:18 | 8 | 170.3 | 4 | 38400 | Staff reading impossible for max. pt. |
| 22 | 13:16 | 3S2 | 160.44 | 160.5 | 78140 | 13:15:17 | 9 | 158.3 | 5 | 80000 | |
| 23 | 13:16 | 2S2-1 | 157.80 | 158 | 62280 | 13:15:52 | 11 | 157.5 | 5 | 73000 | |
| 24 | 13:19 | 2S2-1 | 160.44 | 160 | 78020 | 13:16:09 | 11 | 159.8 | 5 | 68000 | |
| 25 | 13:23 | 3S2 | 158.88 | 159 | 77640 | 13:21:43 | 9 | 156.8 | 5 | 79300 | |
| 26 | 13:25 | 3S2 | 159.96 | 160.5 | 71440 | 13:23:48 | 9 | 158.3 | 5 | 73600 | |
| 27 | 13:25 | 3S2 | 159.12 | 158.5 | 76800 | 13:24:07 | 9 | 157.5 | 5 | 79400 | |
| 28 | 13:29 | 3S2 | 156.00 | 153.5 | 76300 | 13:28:13 | 9 | 144.8 | 5 | 75300 | |
| 29 | 13:31 | 3S2 | 156.72 | 157.5 | 43860 | 13:29:50 | 9 | 176.3 | 5 | 46900 | |
| 30 | N/A | 3S2 | 152.40 | 156.5 | | 13:33:37 | 9 | 153.8 | 5 | 70800 | |
| 31 | 13:34 | 3S2 | 156.24 | 0 | 69840 | 13:34:00 | 9 | 155.3 | 5 | 76200 | No staff reading. |
| 32 | 13:37 | 3S2 | 161.28 | 163 | 78700 | 13:35:49 | 9 | 160.5 | 5 | 76500 | |
| 33 | 13:39 | 3S2 | 161.40 | 161.5 | 63440 | 13:37:50 | 9 | 176.3 | 5 | 62700 | |
| 34 | 13:39 | 3S2 | 157.80 | 159 | 79360 | 13:38:36 | 9 | 156.8 | 5 | 78300 | |
| 35 | 13:49 | 2S1-2 | 156.72 | 156.5 | 64780 | 13:47:57 | 11 | 157.5 | 5 | 73800 | |

***Matching static scale printouts on axle spacing, time, etc. with STI computer-downloaded data; comments indicate bad or unavailable data.**

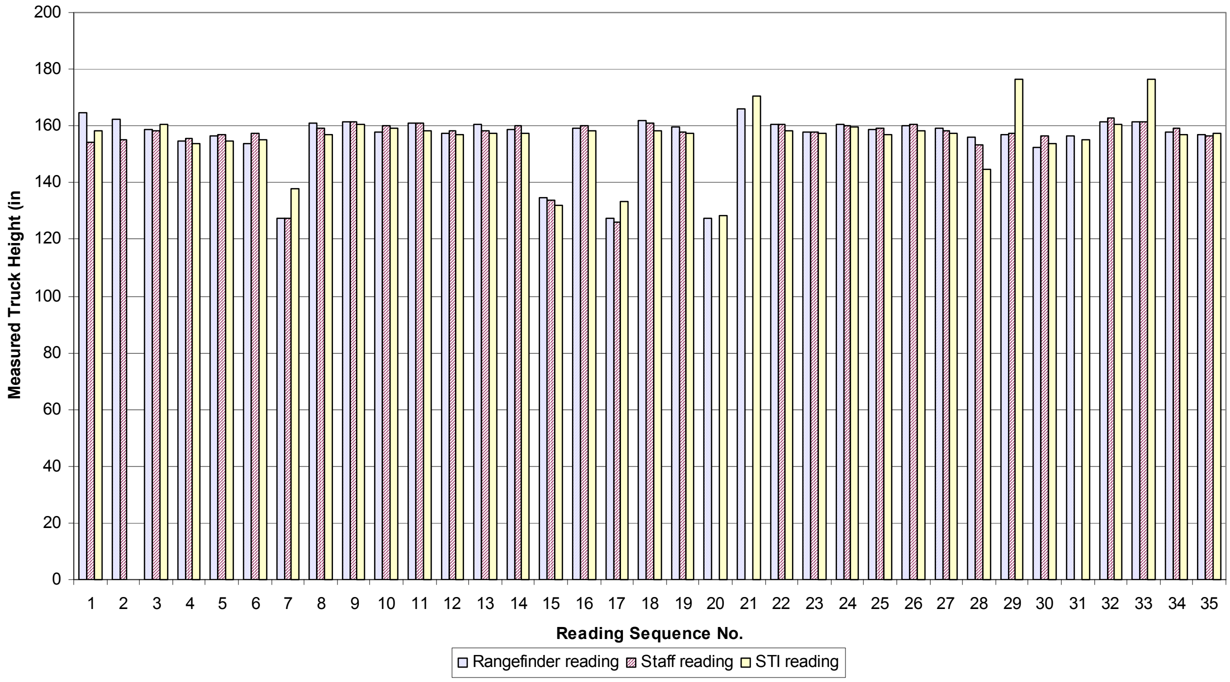



Figure B.5. Comparison of Raw Data Readings from STI Scanner, Rangefinder, and Measuring Staff

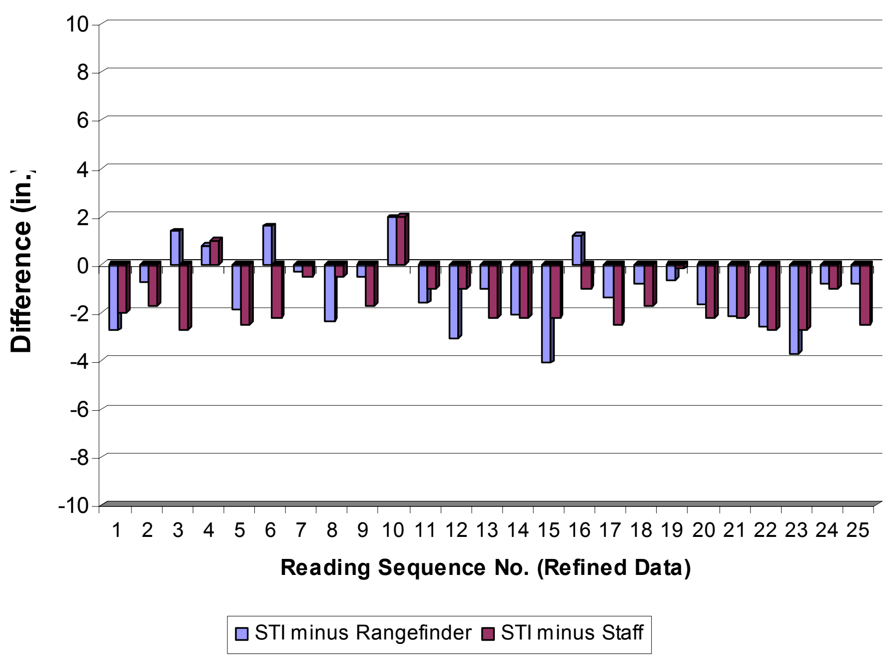



Figure B.6. Differences Between STI Scanner and Rangefinder/Staff Readings

Table B.4. ANOVA Test for Means of Readings (Refined Data) from STI Scanner, Rangefinder and Measuring Staff

SUMMARY

| *Groups* | *Count* | *Sum* | *Average* | *Variance* |
|---|---|---|---|---|
| rangefinder | 25 | 3941.4 | 157.656 | 28.6728 |
| staff | 25 | 3951.5 | 158.06 | 28.1525 |
| Sti scanner | 25 | 3913.4 | 156.536 | 29.4974 |

ANOVA

| *Source of Variation* | *SS* | *df* | *MS* | *F* | *P-value* | *F crit* |
|---|---|---|---|---|---|---|
| Between Groups | 31.16826667 | 2 | 15.58413333 | 0.541600297 | 0.584168002 | 3.123901138 |
| Within Groups | 2071.7448 | 72 | 28.77423333 | | | |
| | | | | | | |
| Total | 2102.913067 | 74 | | | | |

Recommendation: Accept Null Hypothesis, Same means

Figure B.7: Calibration of STI Vehicle Scanner – OMCC's Measuring Staff

Figure B.8: Calibration of STI Vehicle Scanner – Staff Measurement of Truck Height

# Appendix C: Truck Height Histograms

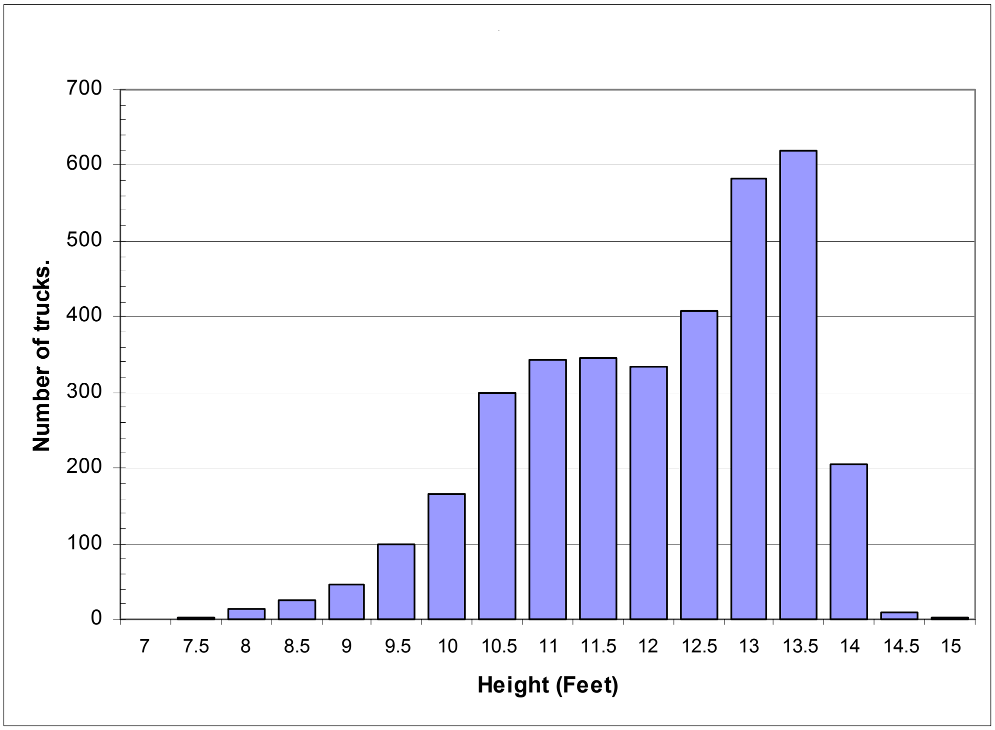



Figure C.1: Truck Height Histogram, OMCC Station #19, I-75, Punta Gorda, FL. Functional Class 01. August 12-August 16, 2002

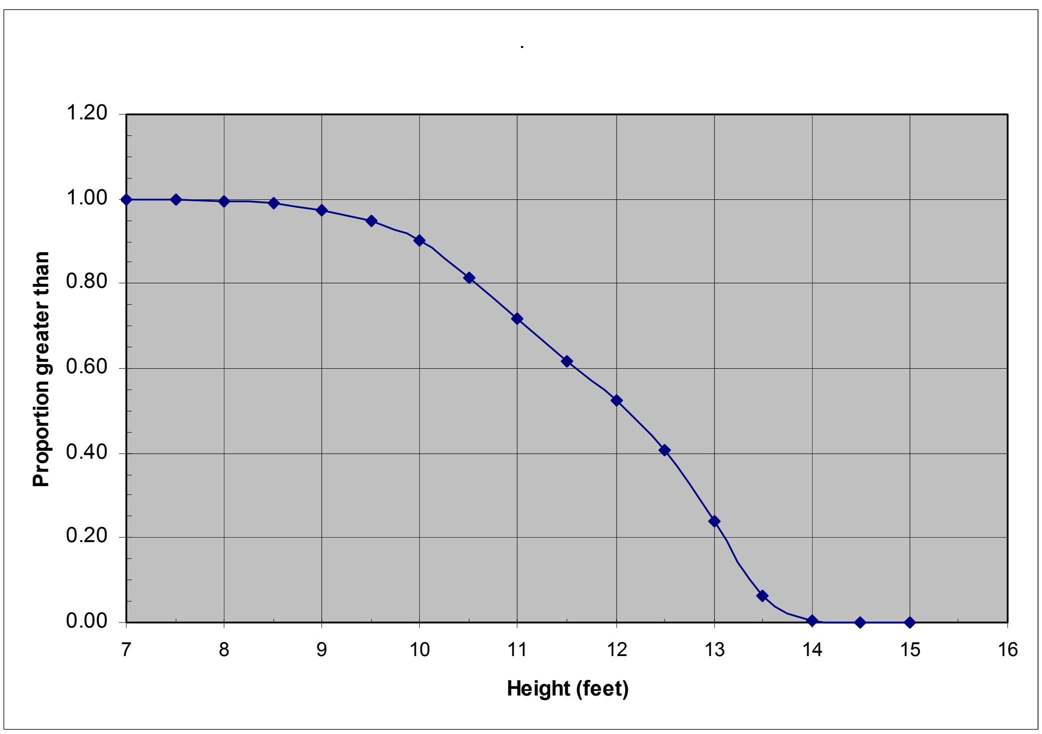



Figure C.2: Truck Height Reverse Cumulative Frequency Curve, OMCC Station #19, I-75, Punta Gorda, FL. Functional Class 01. August 12-August 16, 2002

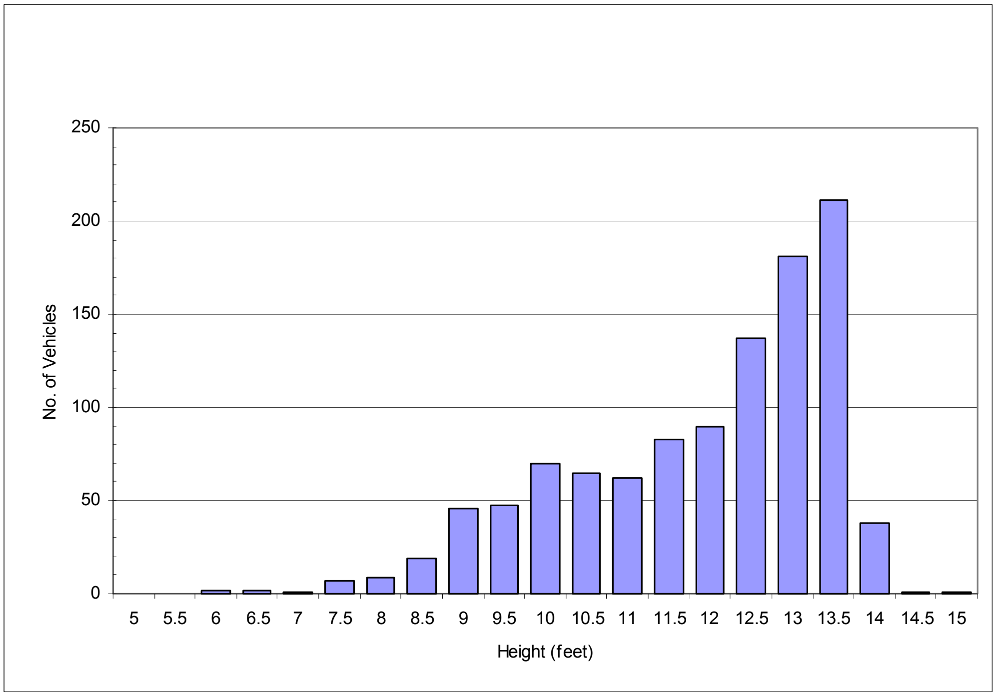
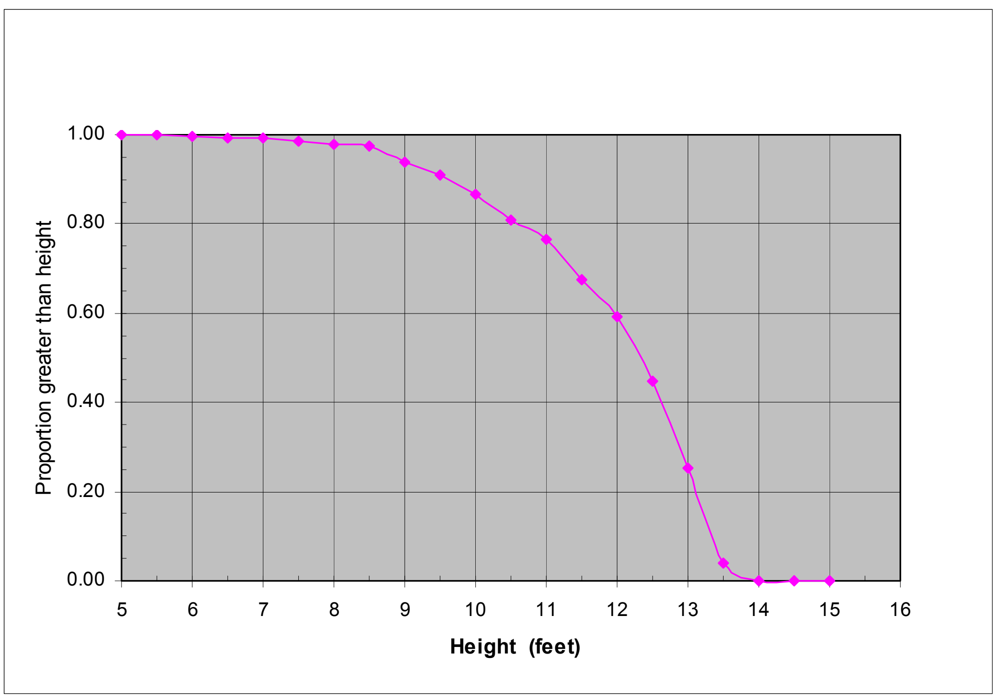



Figure C.3: Truck Height Histogram, OMCC Station #11, US-19 Old Town, FL. Functional Class 02. June 19- June 21, 2002

.




.

Figure C.4: Truck Height Cumulative Relative Frequency, OMCC Station #11, US-19 Old Town, FL. Functional Class 02. June 19- June 21, 2002

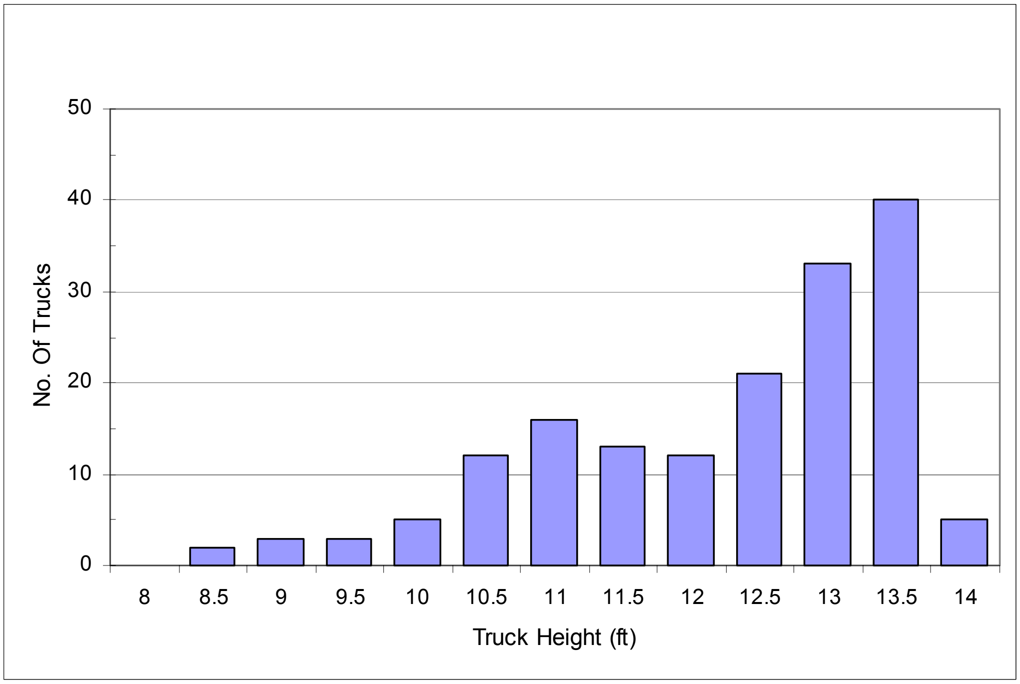
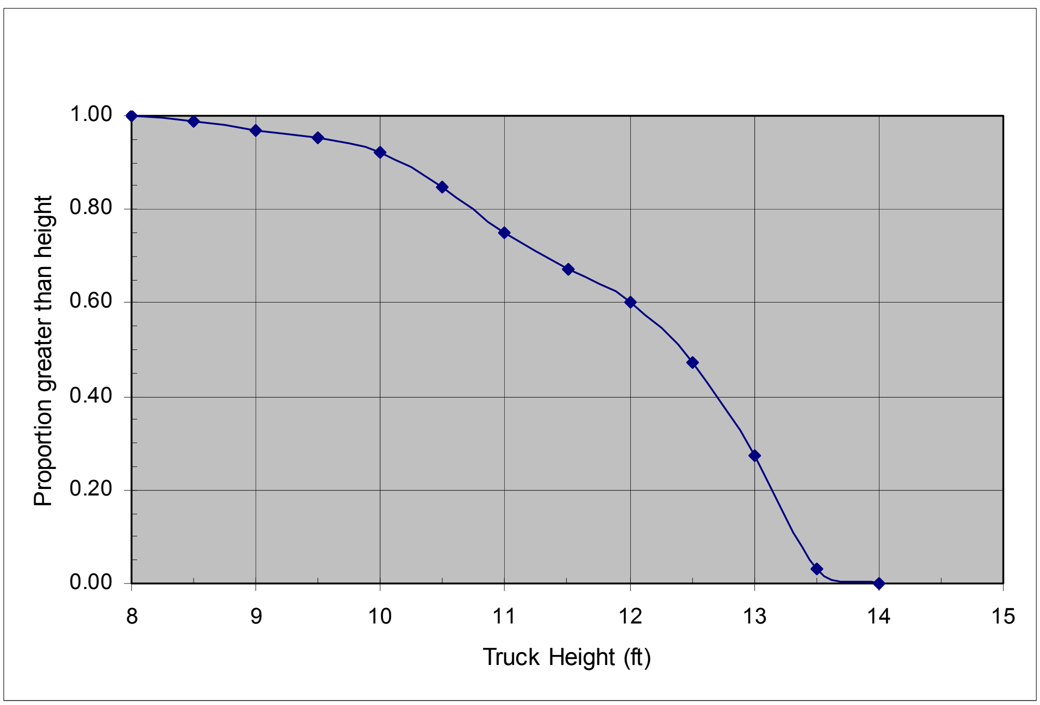



Figure C.5: Truck Height Histogram, OMCC Station #7, SR-121, MacClenny, FL. Functional Class 06. July 10-July 11, 2002

.

.




.

Figure C.6: Truck Height Reverse Cumulative Relative Frequency Curve, OMCC Station #7, SR-121, Macclenny, FL. Functional Class 06. July 10-July 11, 2002

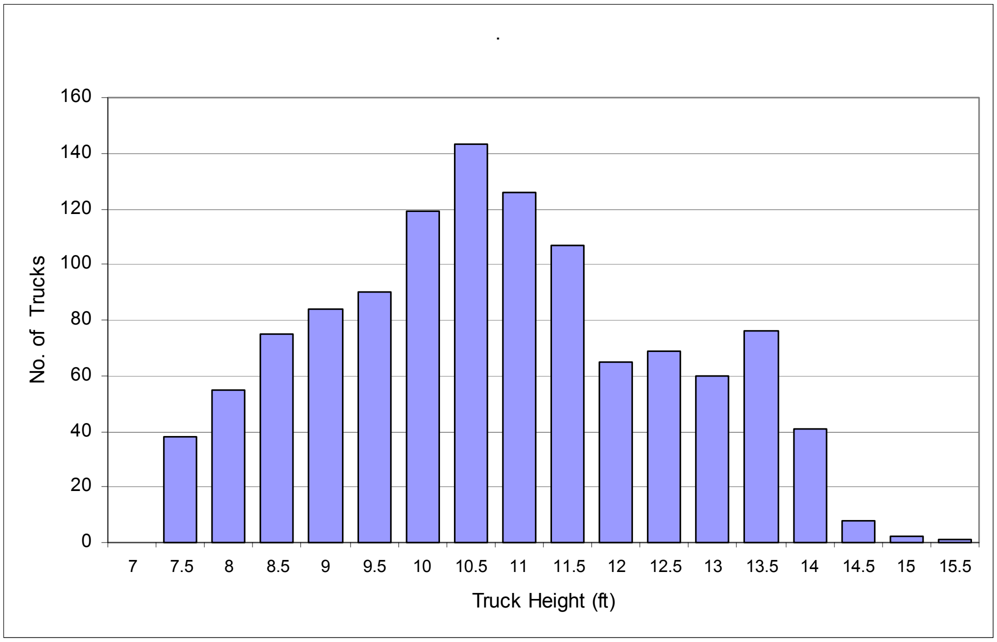



Figure C.7: Truck Height Histogram FDOT TTMS (WIM) 9908, US-319, Tallahassee, FL. Functional Class 14. June 27-June 28, 2002

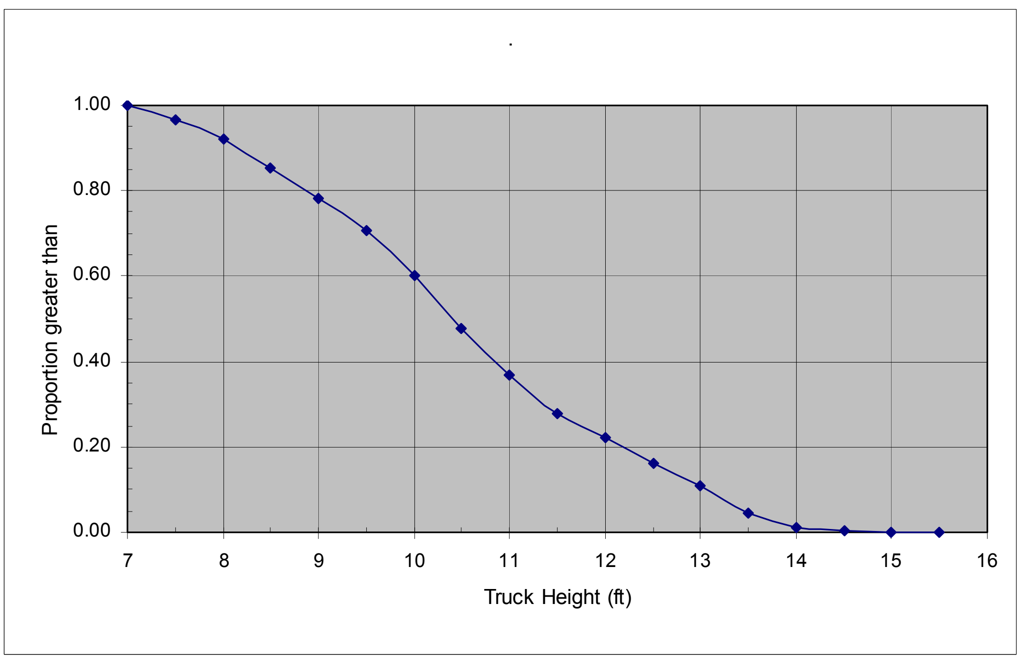



Figure C.8: Truck Height Reverse Cumulative Relative Frequency, FDOT TTMS (WIM) 9908, US-319, Tallahassee, FL. Functional Class 14. June 27-June 28, 2002

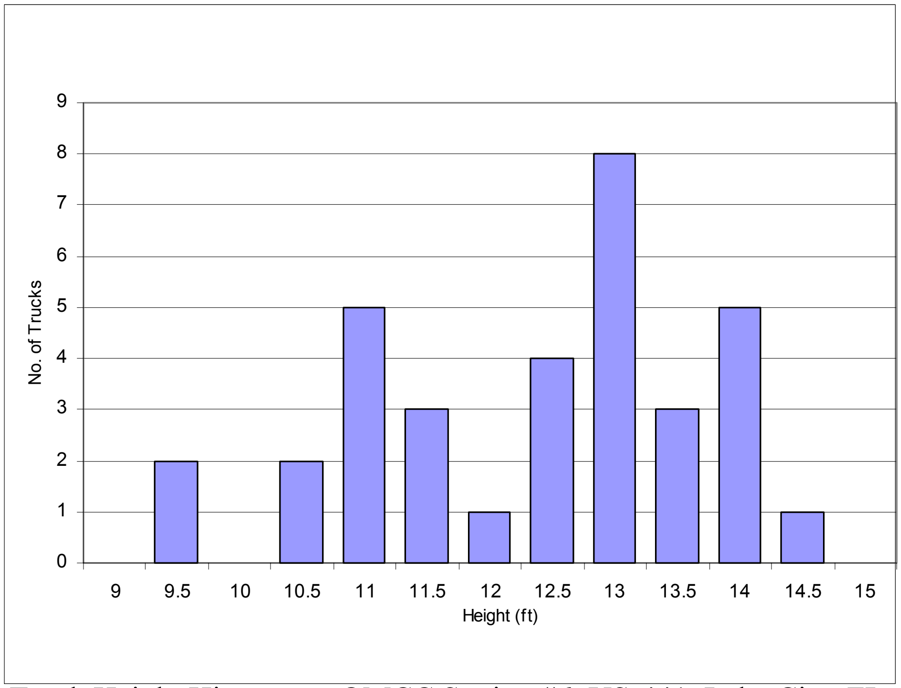



Figure C.9: Truck Height Histogram, OMCC Station #6, US-441, Lake City, FL. Functional Class 14. June 17, 2002

.

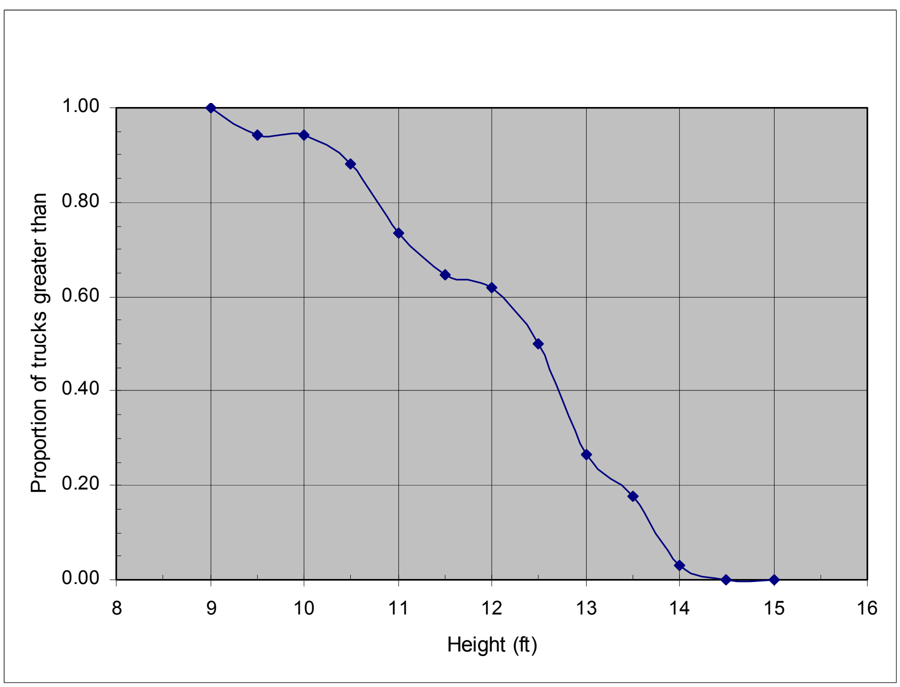



Figure C.10: Truck Height Reverse Cumulative Relative Frequency Curve, OMCC Station #6, US-441, Lake City, FL. Functional Class 14. June 17, 2002

.

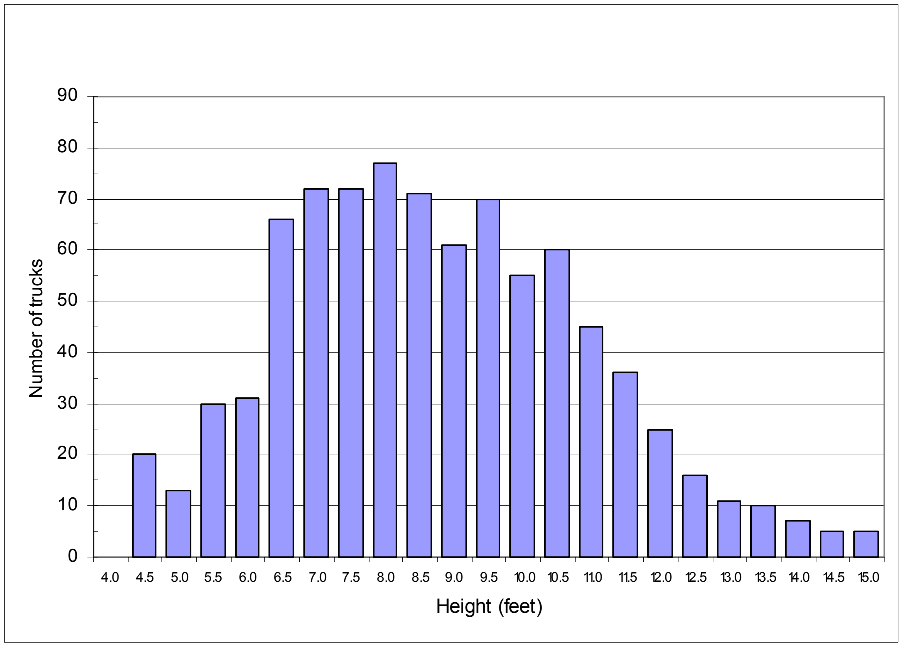



Figure C.11: Truck Height Histogram, FDOT TTMS 872515, SR-823, Miami, FL. Functional Class 19. October 7- October 9, 2002

.

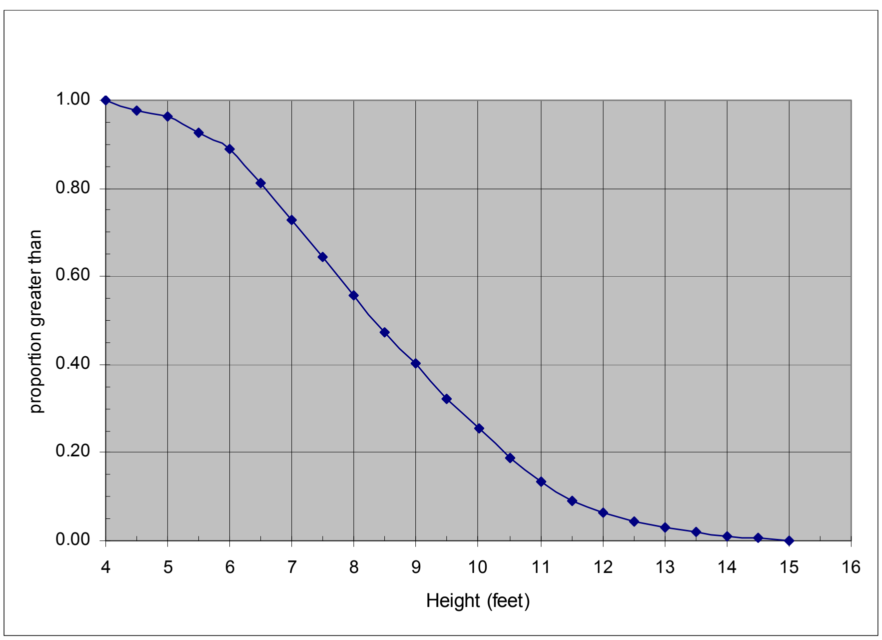



Figure C.12: Truck Height Reverse Cumulative Relative Frequency Curve FDOT TTMS 872515, SR-823, Miami, FL. Functional Class 19. October 7- October 9, 2002

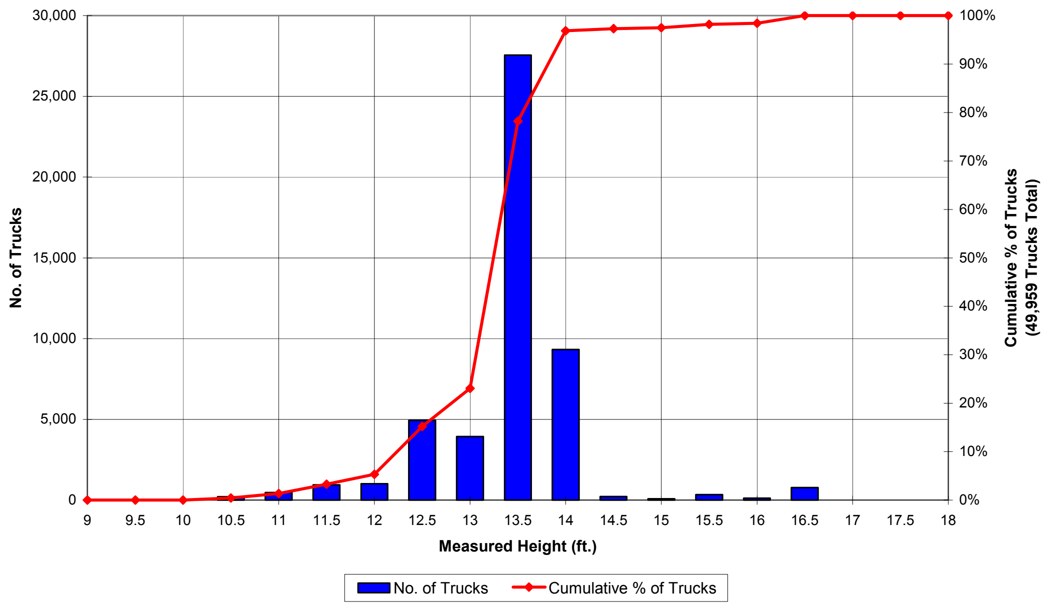



Figure C.13: Truck Height Histogram at Sneads Weigh Station, Interstate 10, Florida, Nov. 14 -- Dec. 14, 2002.

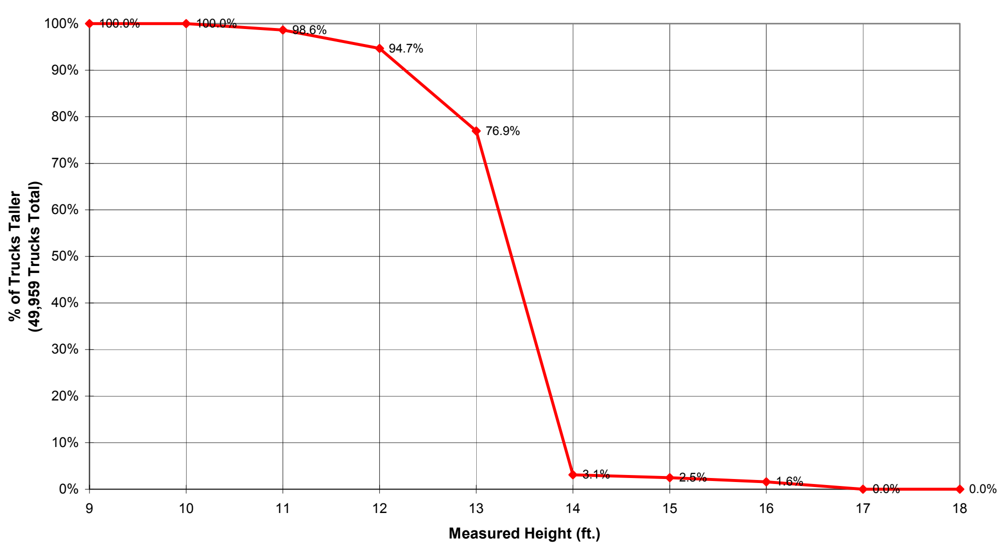



Figure C.14 Reverse Cumulative Relative Frequency Curve for Truck Heights at Sneads Weigh Station, Interstate 10, Florida, Nov. 14 -- Dec. 14, 2002.

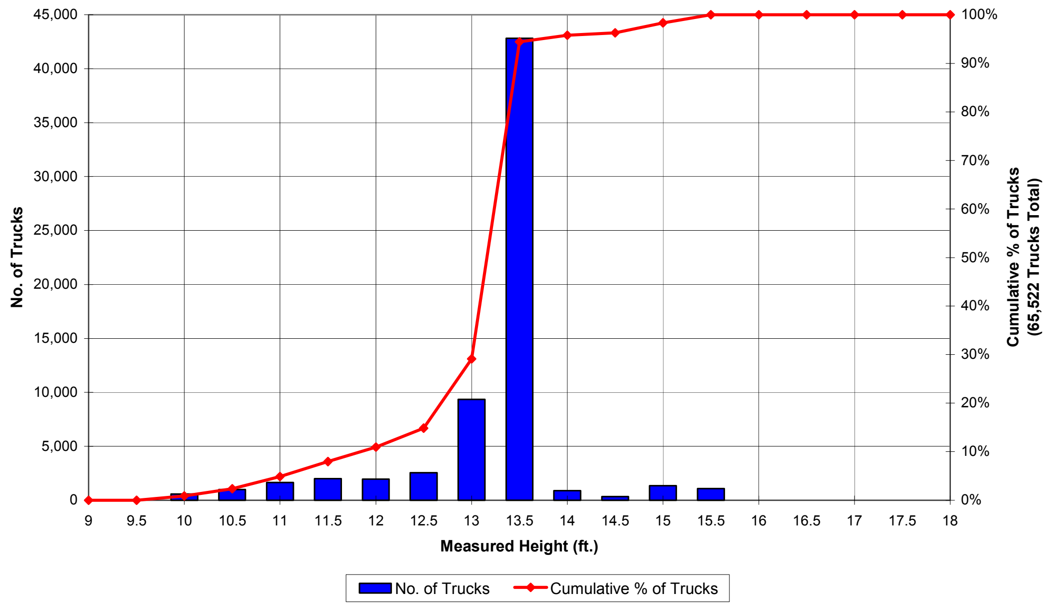



Figure C.15: Truck Height Histogram at White Springs Weigh Station, Interstate 75, Florida, Jan 22 -- Feb. 11, 2003.

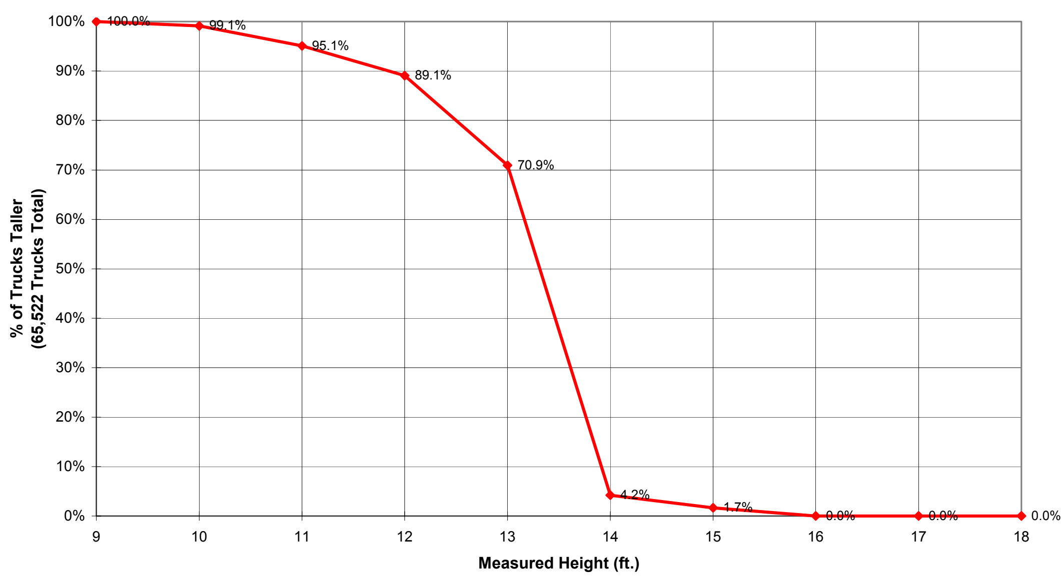



Figure C.16: Reverse Cumulative Relative Frequency Curve for Truck Heights at White Springs Weigh Station, Interstate 75, Florida, Jan 22 -- Feb. 11, 2003.

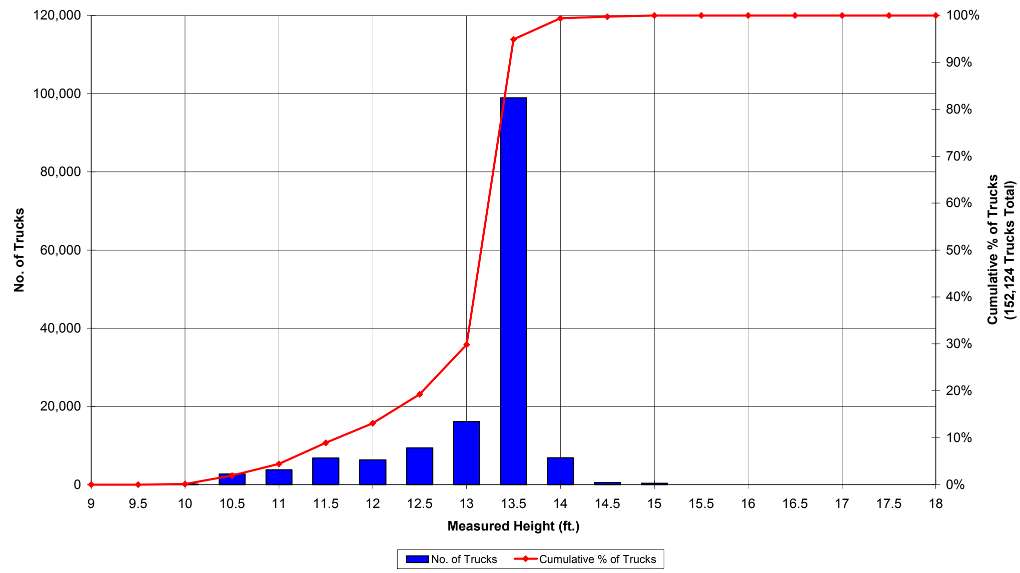



Figure C.17: Truck Height Histogram at Flagler Weigh Station, Interstate 95, Florida, April 1 – May 22, 2003.

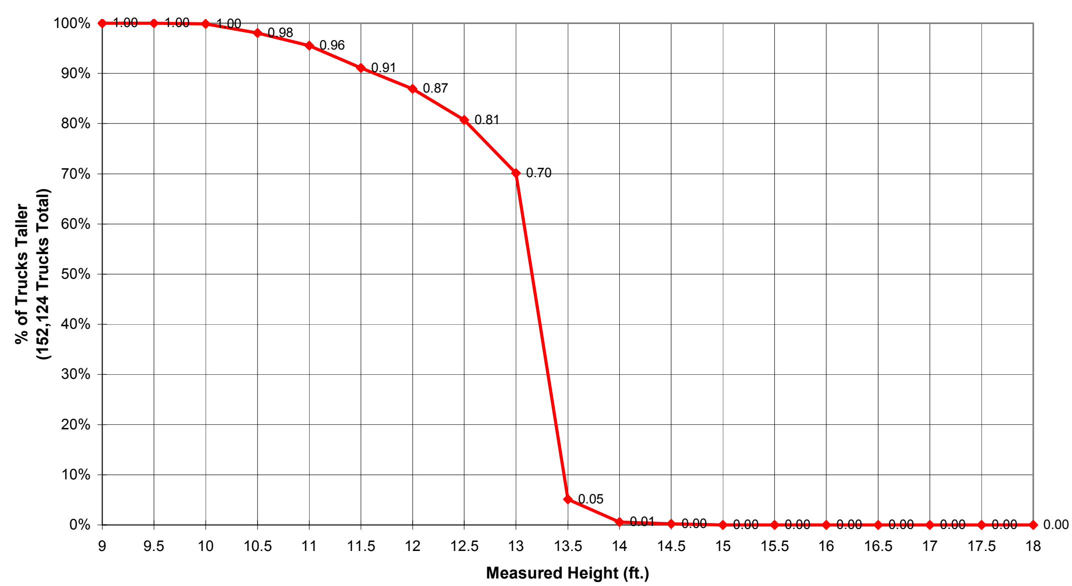



Figure C.18: Reverse Cumulative Relative Frequency Curve for Truck Heights at Flagler Weigh Station, Interstate 95, Florida, April 1 – May 22, 2003.

# Appendix D: Truck Weight Histograms

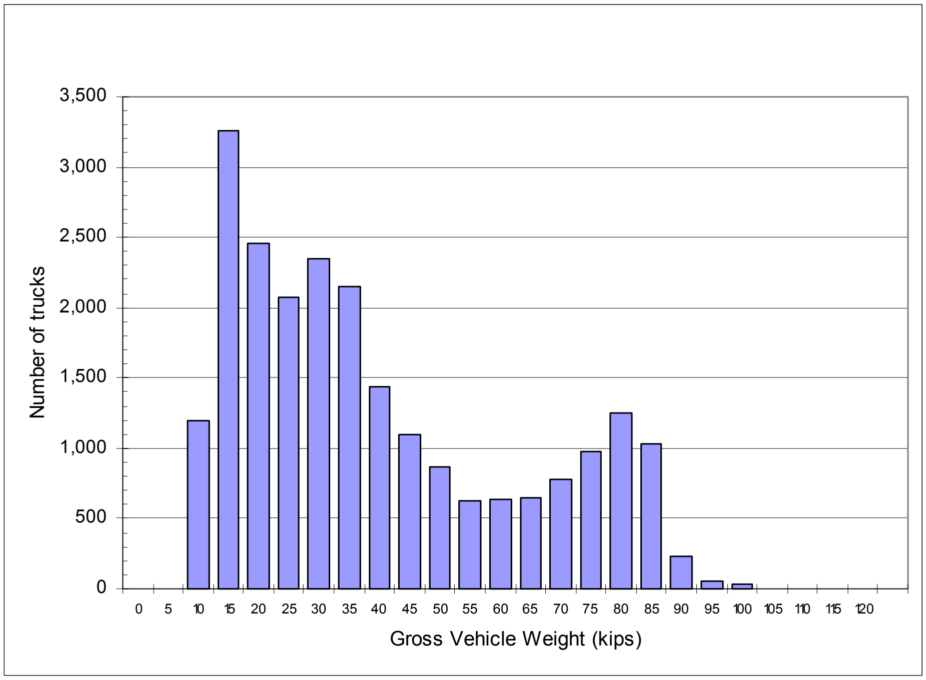



**Figure D.1: Truck Weight Histogram, FDOT TTMS 9924, I-10, Pensacola, FL. Functional Class 11. August 2002**

.

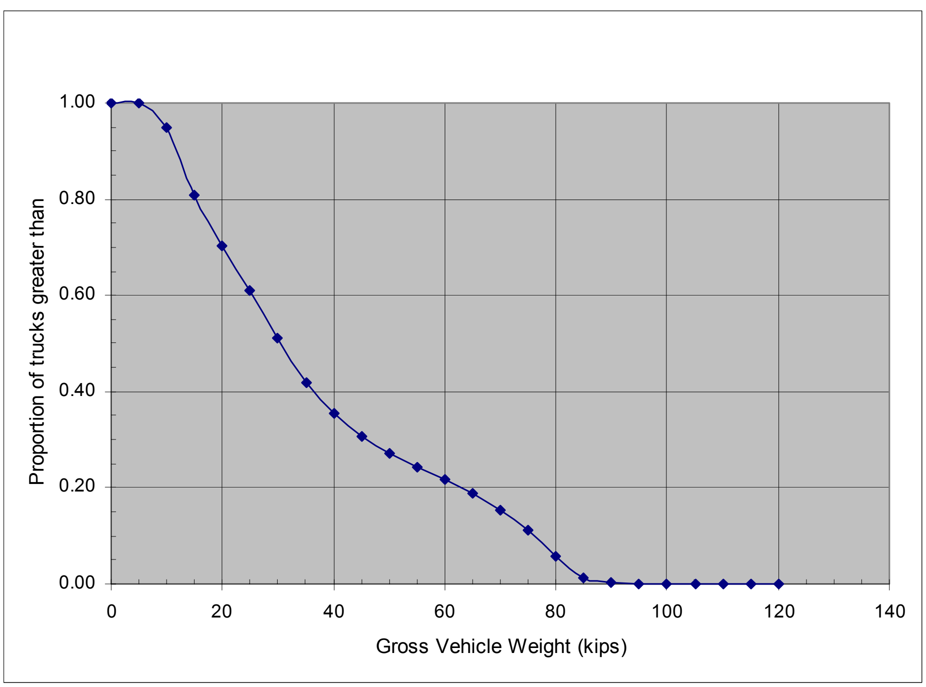



**Figure D.2: Truck Weight Reverse Cumulative Relative Frequency Curve, FDOT TTMS 9924, I-10, Pensacola, FL. Functional Class 11. August 2002**

.

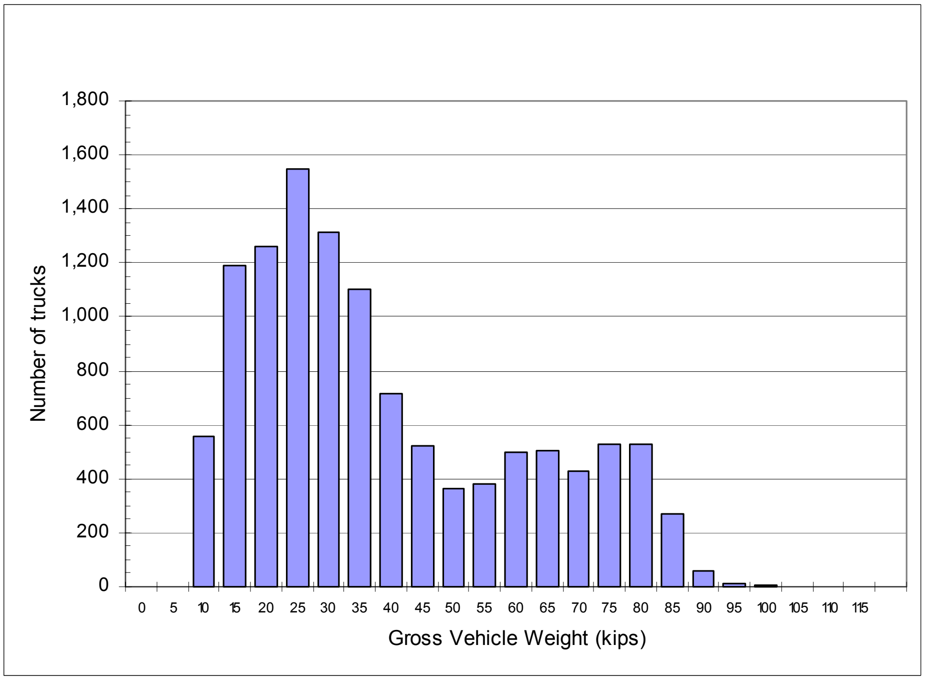



**Figure D.3: Truck Weight Histogram, FDOT TTMS 9940, SR-267, Quincy, FL. Functional Class 07. August 2002**

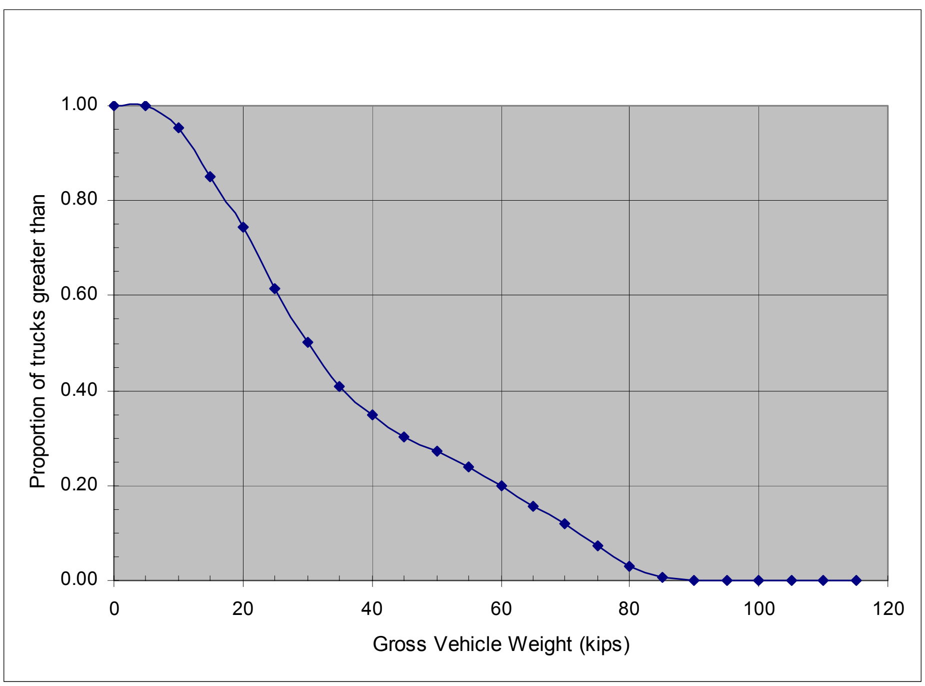



**Figure D.4: Truck Weight Reverse Cumulative Relative Frequency Curve, FDOT TTMS 9940, SR-267, Quincy, FL. Functional Class 07. August 2002**

.

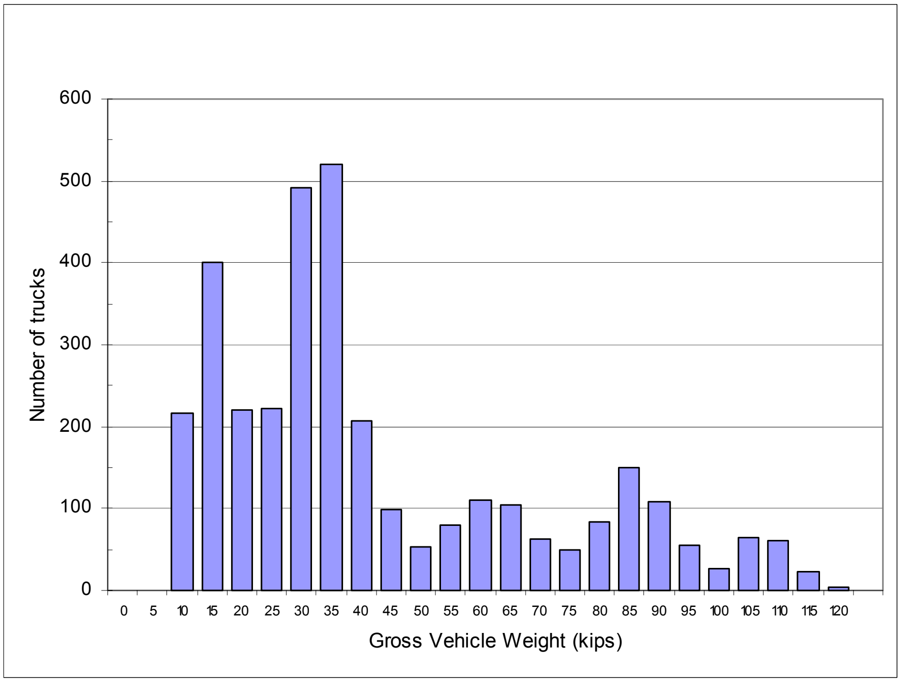



**Figure D.5: Truck Weight Histogram, FDOT TTMS 9946, US-98, St. Marks, FL. Functional Class 06. August 2002**

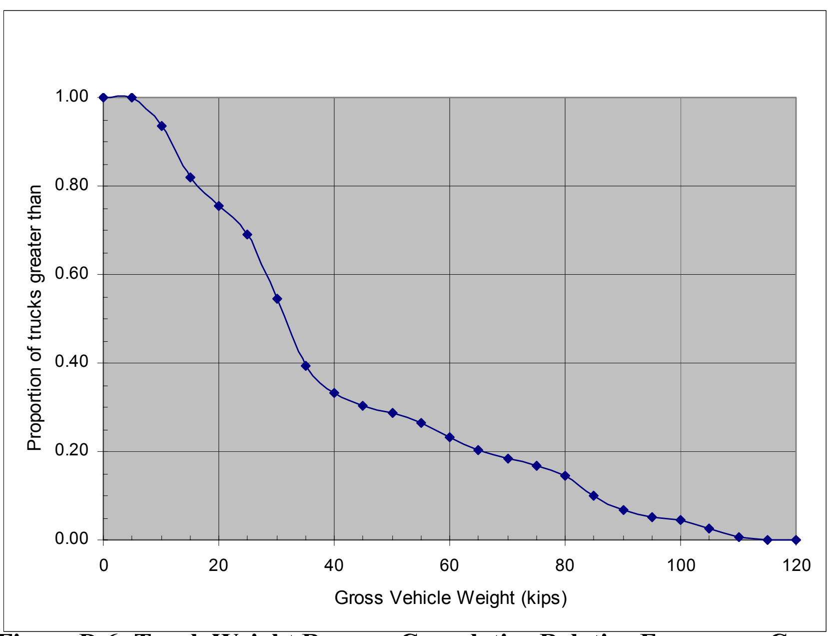



**Figure D.6: Truck Weight Reverse Cumulative Relative Frequency Curve, FDOT TTMS 9946, US-98, St. Marks, FL. Functional Class 06. August 2002**

.

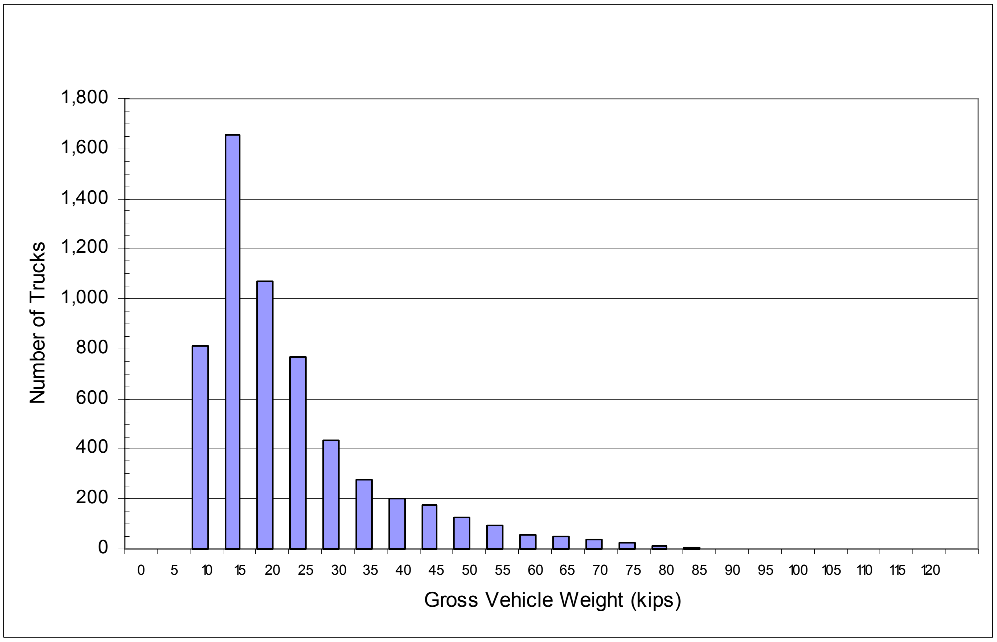



**Figure D.7: Truck Weight Histogram, FDOT TTMS 9921, US-1, Jupiter, FL. Functional Class 02. December 2000**

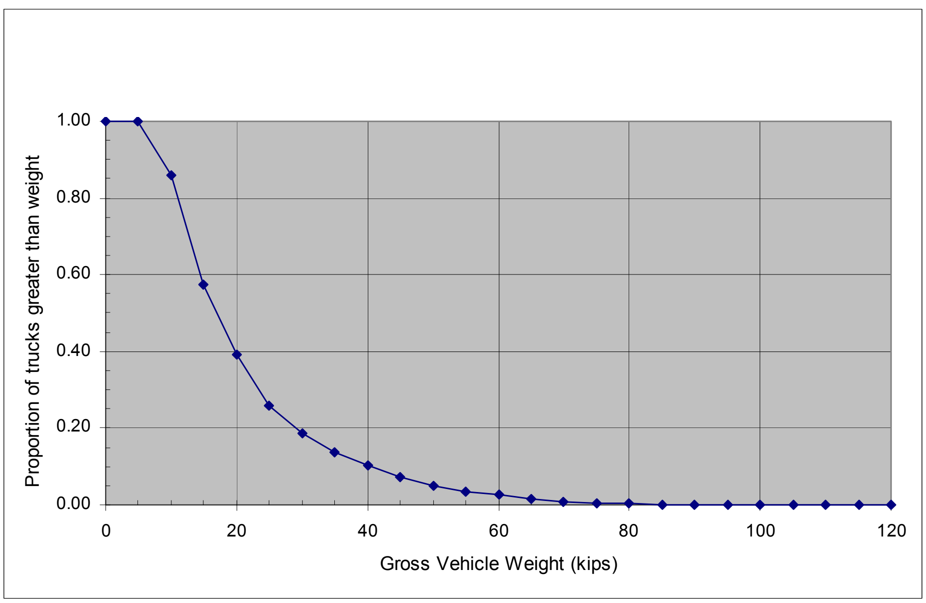



**Figure D.8: Truck Weight Reverse Cumulative Relative Frequency Curve, FDOT TTMS 9921, US-1, Jupiter, FL. Functional Class 02. December 2000**

.

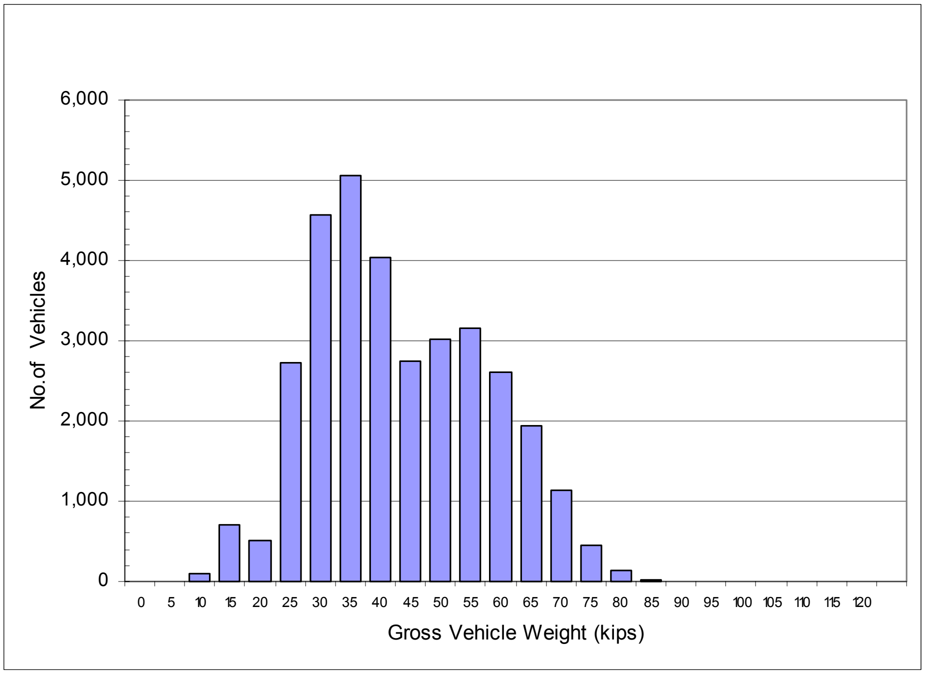



**Figure D.9: Truck Weight Histogram, FDOT TTMS 9935, US-27, Palm Beach, FL. Functional Class 02. December 2000**

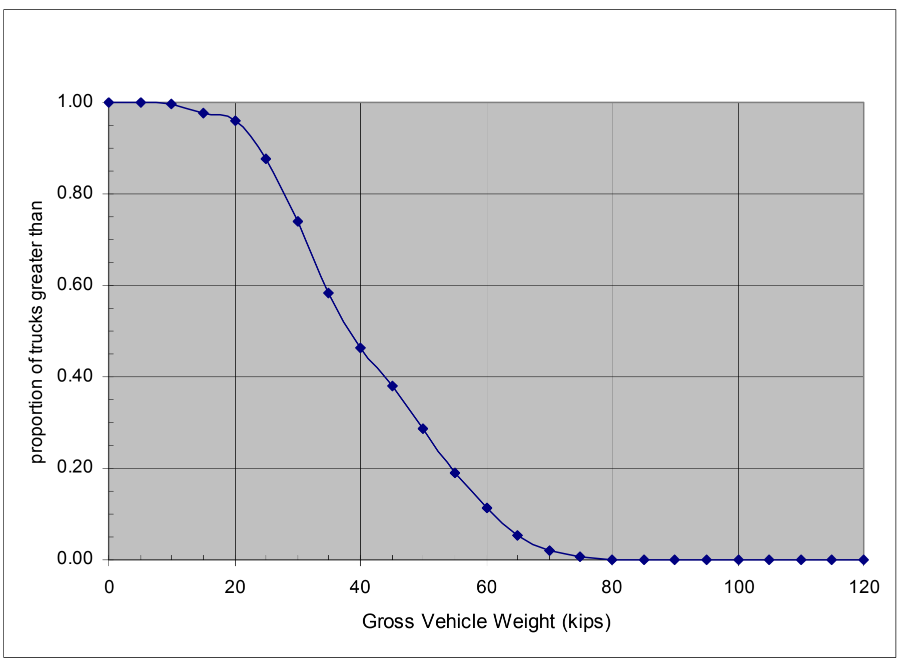



**Figure D.10: Truck Weight Reverse Cumulative Relative Frequency Curve, FDOT TTMS 9935, US-27, Palm Beach, FL. Functional Class 02. December 2000**

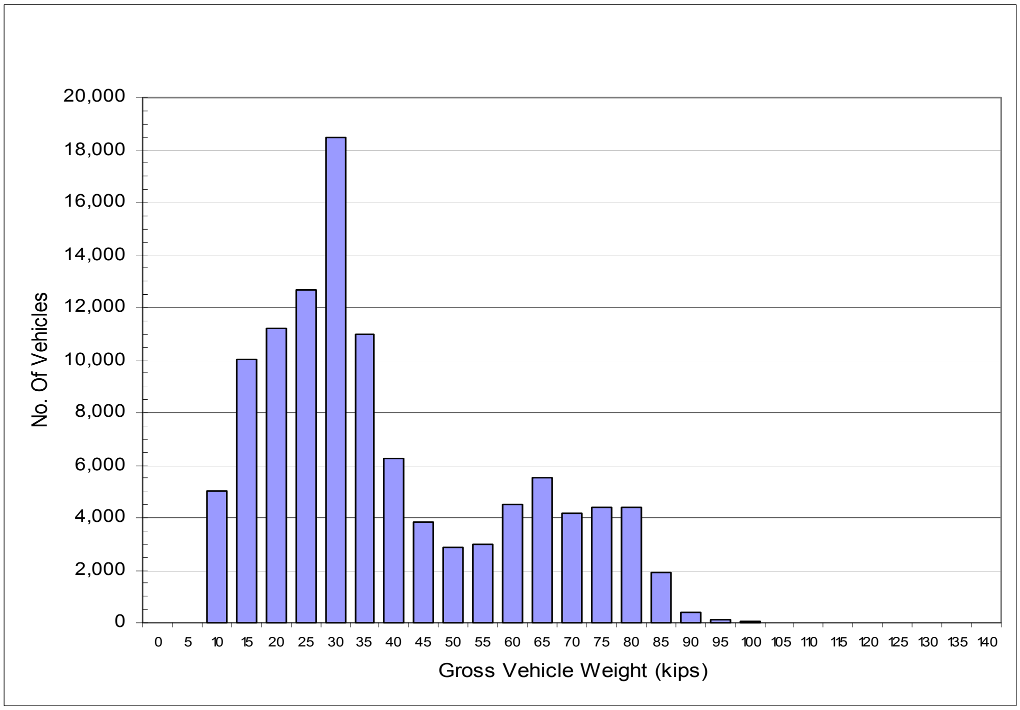



**Figure D.11: Truck Weight Histogram, FDOT TTMS 9908, US-319, Tallahassee, FL. Functional Class 14. 06-12/2000**

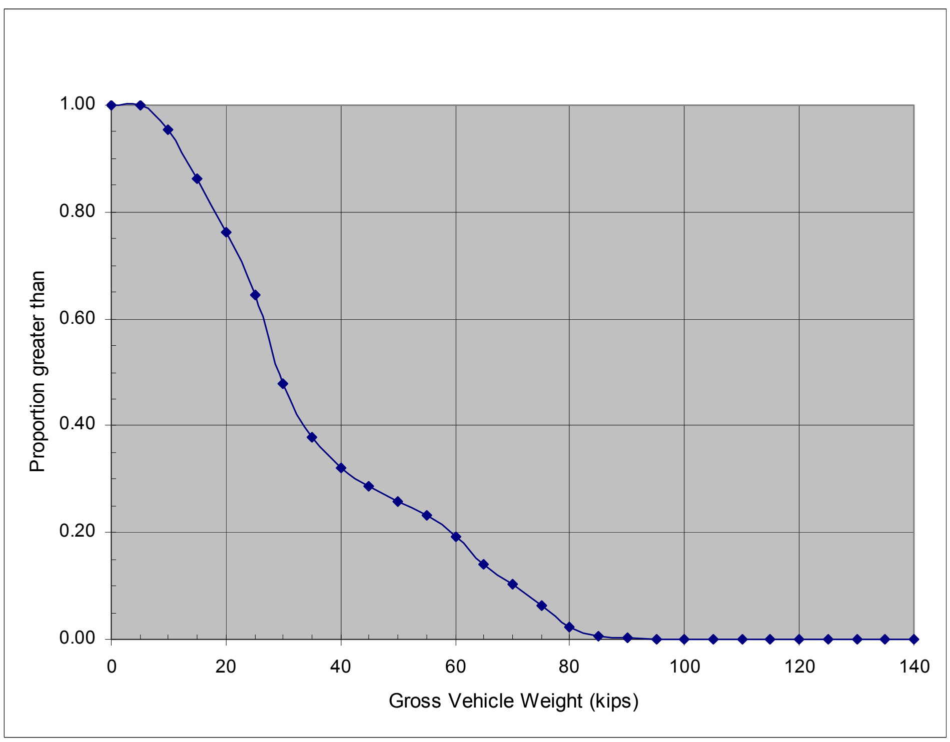



**Figure D.12: Truck Weight Reverse Cumulative Relative Frequency Curve, FDOT TTMS 9908, US-319, Tallahassee, FL. Functional Class 14. 06-12/2000**

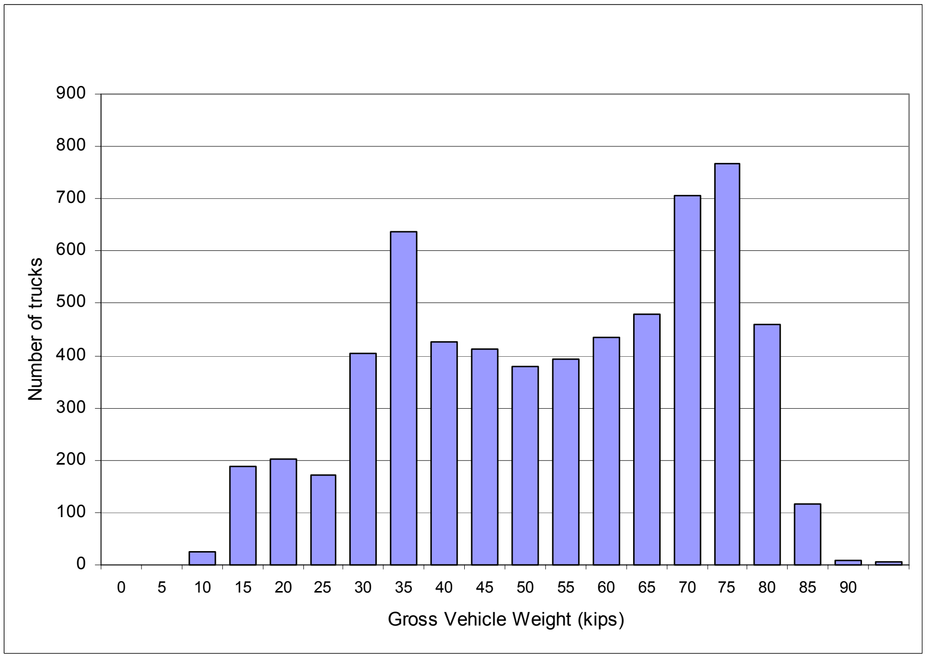



**Figure D.13: Truck Weight Histogram, FDOT TTMS 9946, I-10, Monticello, FL. Functional Class 01. June 1999**

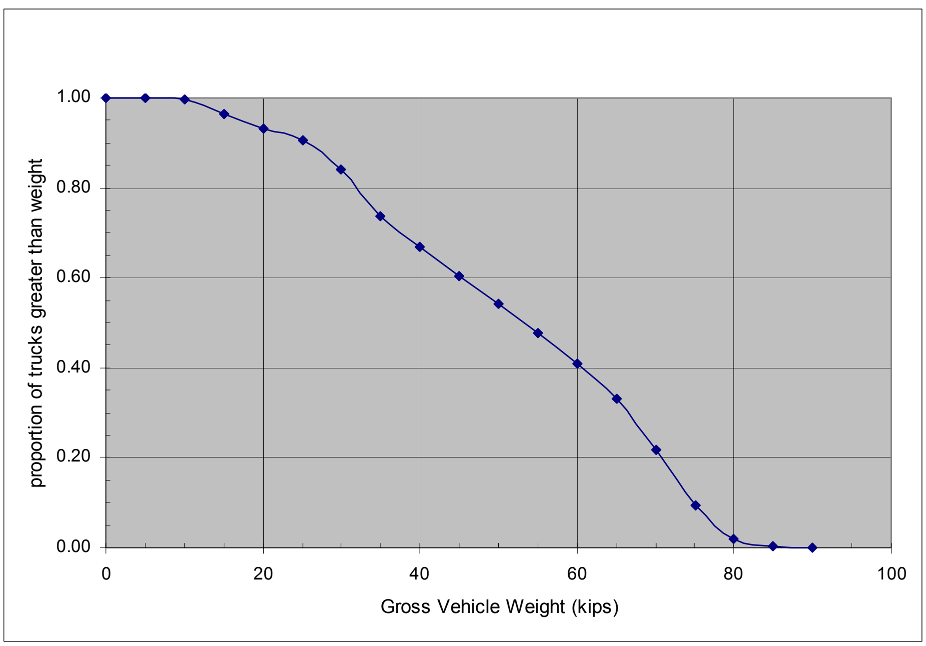



**Figure D.14: Truck Weight Reverse Cumulative Relative Frequency Curve, FDOT TTMS 9946, I-10, Monticello, FL. Functional Class 01. June 1999**

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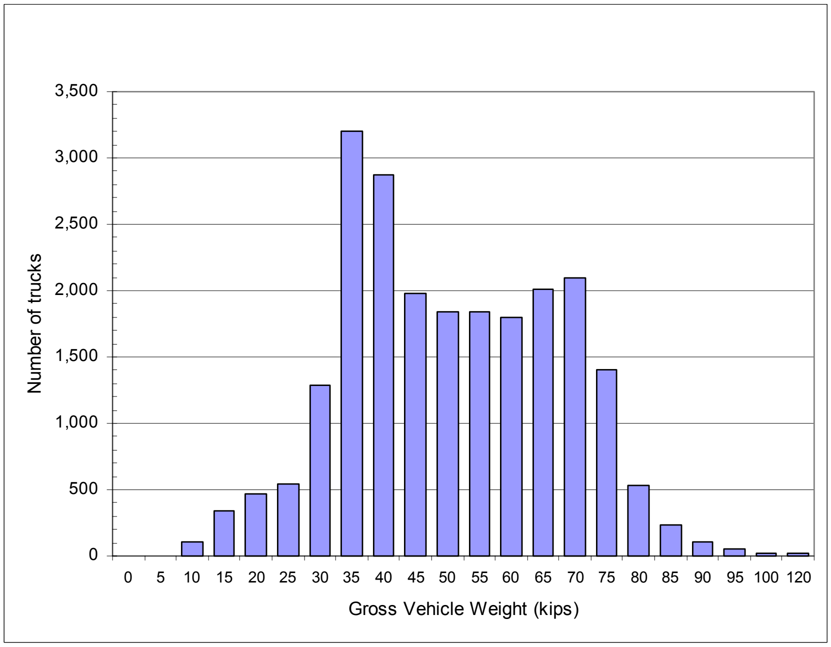


**Figure D.15: Truck Weight Histogram, FDOT TTMS 9936, I-10, Lake City, FL. Functional Class 01. 12/07/2000**

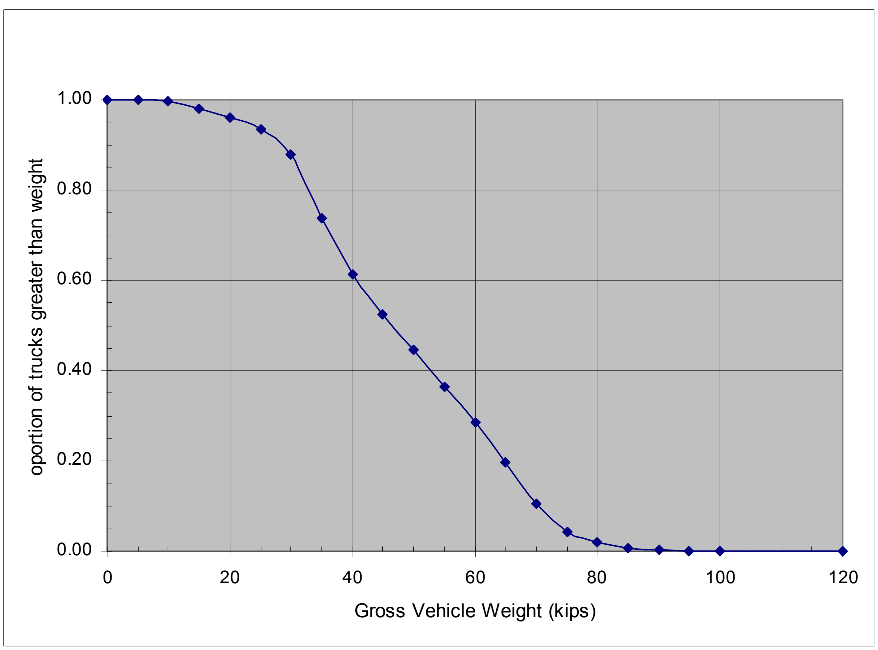


**Figure D.16: Truck Weight Reverse Cumulative Relative Frequency Curve, FDOT TTMS 9936, I-10, Lake City, FL. Functional Class 01. 12/07/2000**

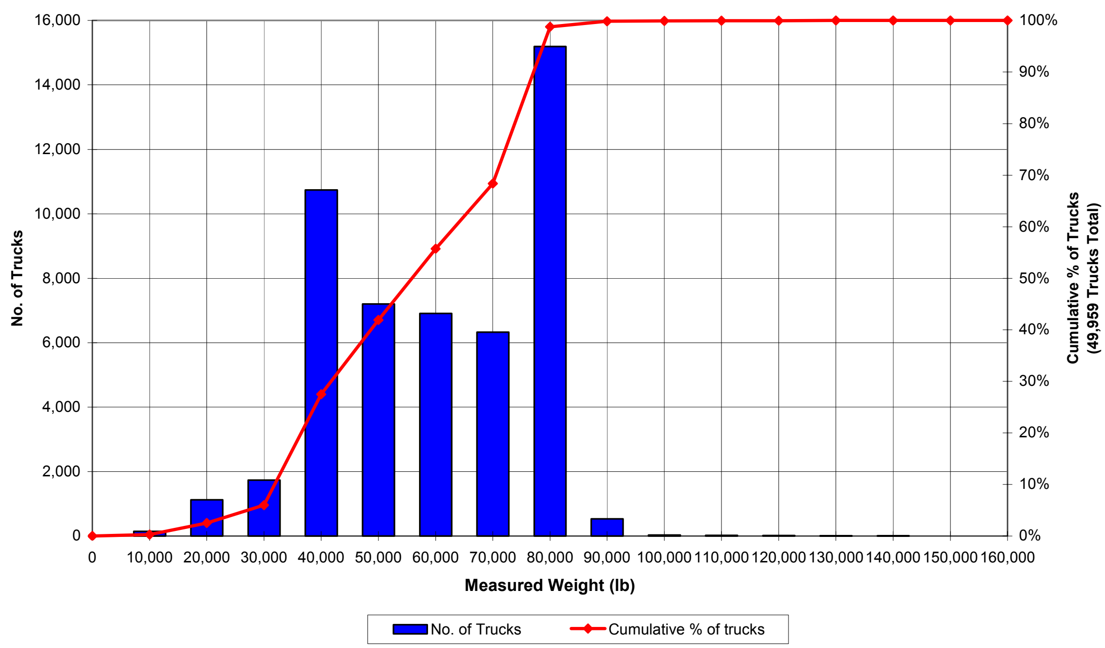


**Figure D.17: Truck Weight Histogram at Sneads Weigh Station, Interstate 10, Florida, Nov 14 – Dec. 14, 2003.**

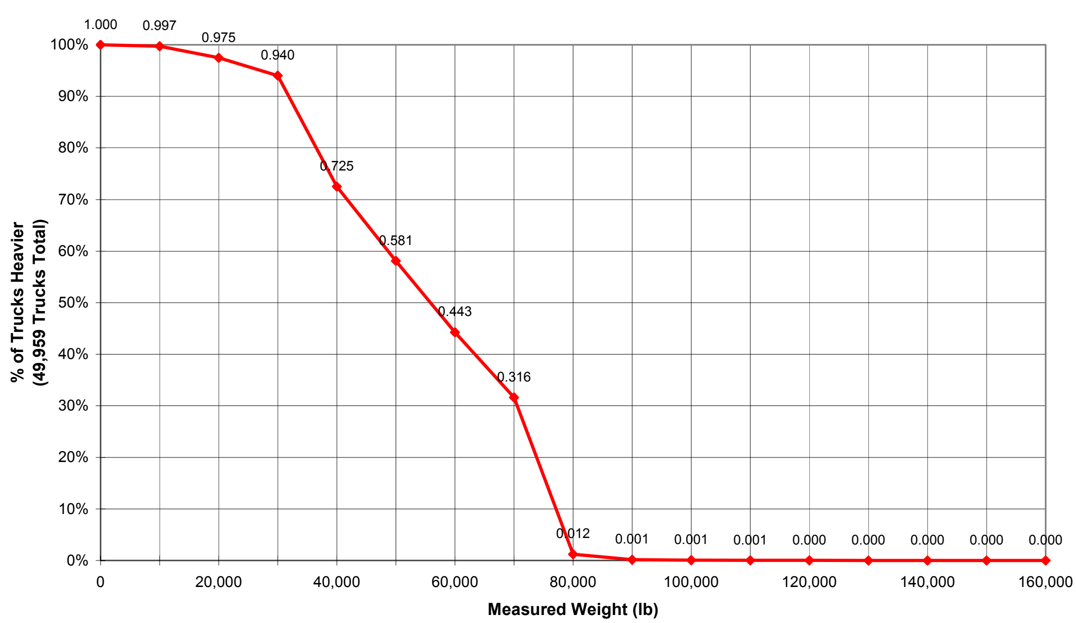


**Figure D.18: Truck Weight Reverse Cumulative Relative Frequency Curve, at Sneads Weigh Station, Interstate 10, Florida, Nov 14 – Dec. 14, 2003.**

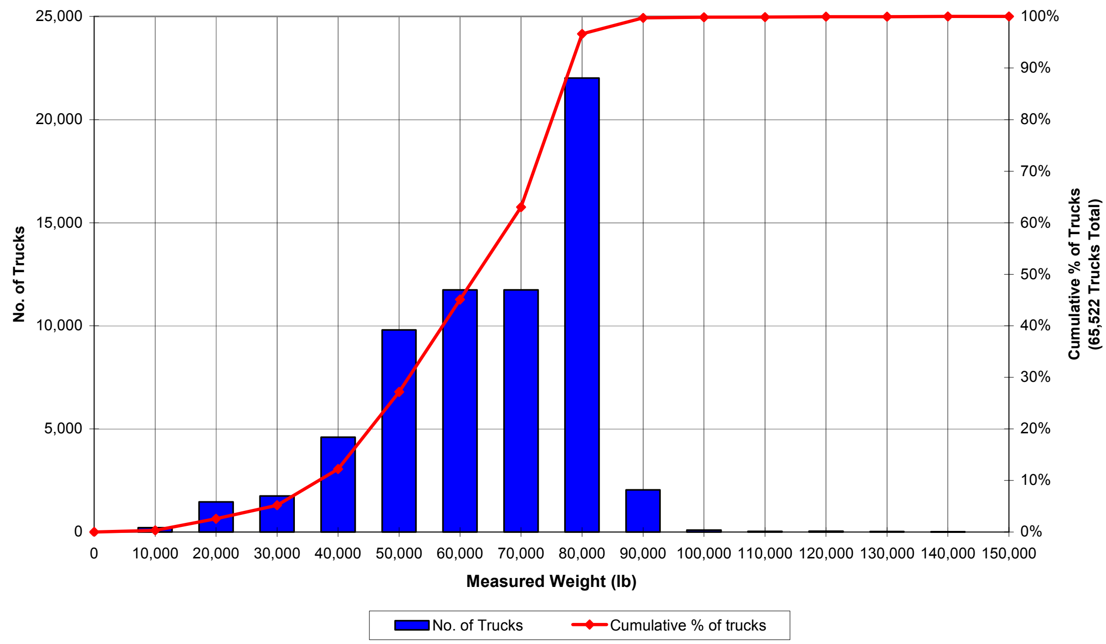


**Figure D.19: Truck Weight Histogram, at White Springs Weigh Station, Interstate 75, Florida, Jan. 22-- Feb. 11, 2003.**

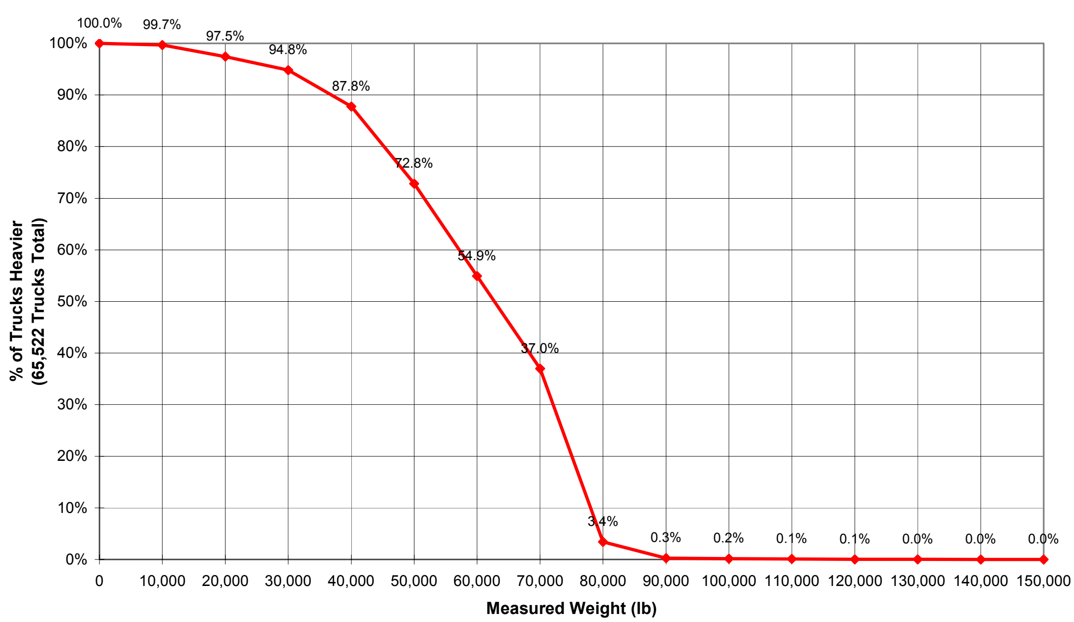


**Figure D.20: Truck Weight Reverse Cumulative Relative Frequency Curve, at White Springs Weigh Station, Interstate 75, Florida, Jan. 22-- Feb. 11, 2003.**

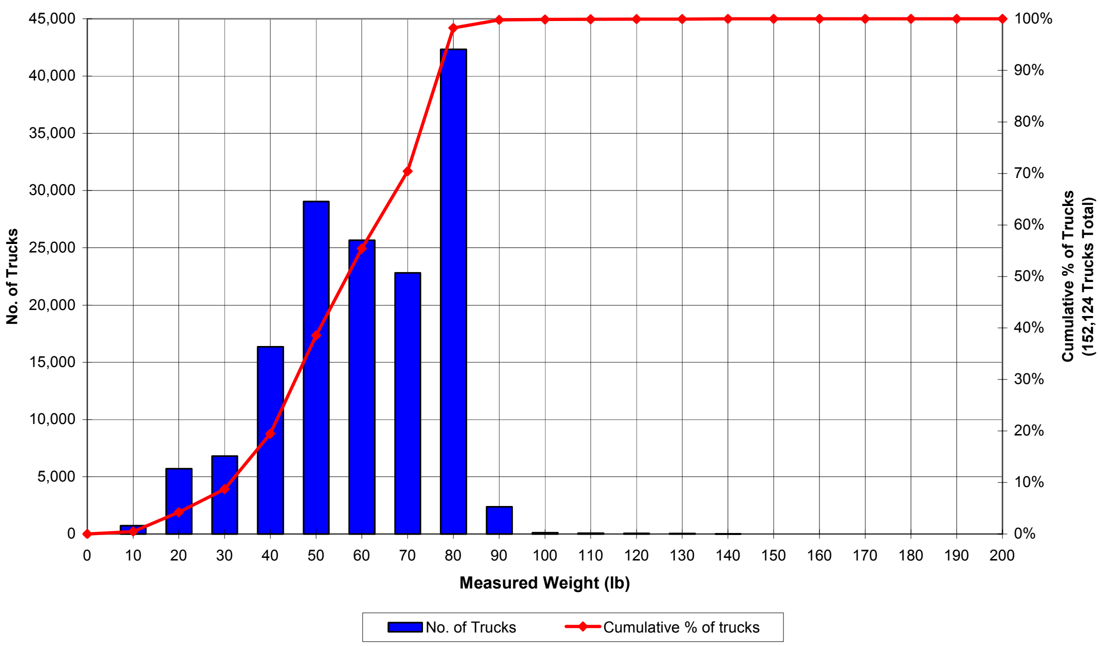


**Figure D.21: Truck Weight Histogram, Flagler Weigh Station, Interstate 95, Florida, April 1 -- May 22, 2003.**

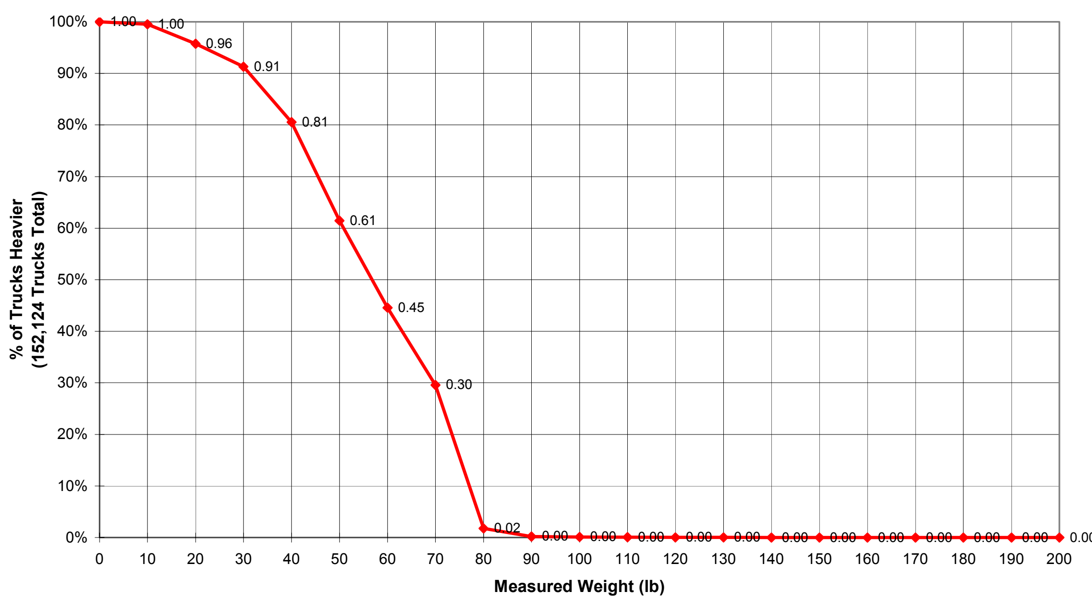


**Figure D.22: Truck Weight Reverse Cumulative Relative Frequency Curve, at Flagler Weigh Station, Interstate 95, Florida, April 1 -- May 22, 2003.**

# Appendix E: Regression Analyses and Curve Fitting

**Table E.1. Equations for Best-fitting Piecewise Functions (Linear and Non-linear) for Truck Heights on Individual Interstate Roadways**

| Data Site | Functional Class | Best-Fitting Piecewise Equations: *Prob* (TruckHeight > x) = | Limits (ft.) | $R^2$ |
|---|---|---|---|---|
| Sneads Weigh Station | 01 | 1.0 | x # 11.0 | |
| | | $-4.58300 + 1.03380x - 0.04786x^2$ | 11.0 < x # 13.0 | 0.98518 |
| | | $141.4300 - 20.18000x + 0.72000x^2$ | 13.0 < x # 14.0 | 1.0000 |
| | | $0.19010 - 0.01119x$ | 14.0 < x # 16.5 | 0.89277 |
| | | 0.0 | x > 16.5 | |
| Whitesprings Weigh Station | 01 | 1.0 | x # 9.0 | |
| | | $0.001431 + 0.22246x - 0.01237x^2$ | 9.0 < x # 12.5 | 0.99845 |
| | | $10.88800 - 0.79607x$ | 12.5 < x # 13.5 | 0.87926 |
| | | $0.42724 - 0.02738x$ | 13.5 < x # 15.5 | 0.96820 |
| | | 0.0 | x > 13.5 | |
| Flagler Weigh Station | 01 | 1.0 | x # 10.0 | |
| | | $-1.37580 + 0.49407x - 0.02564x^2$ | 10.0 < x # 13.0 | 0.99421 |
| | | $17.61400 - 1.30100x$ | 13.0 < x # 13.5 | 1.0000 |
| | | 0.0 | x > 13.5 | |

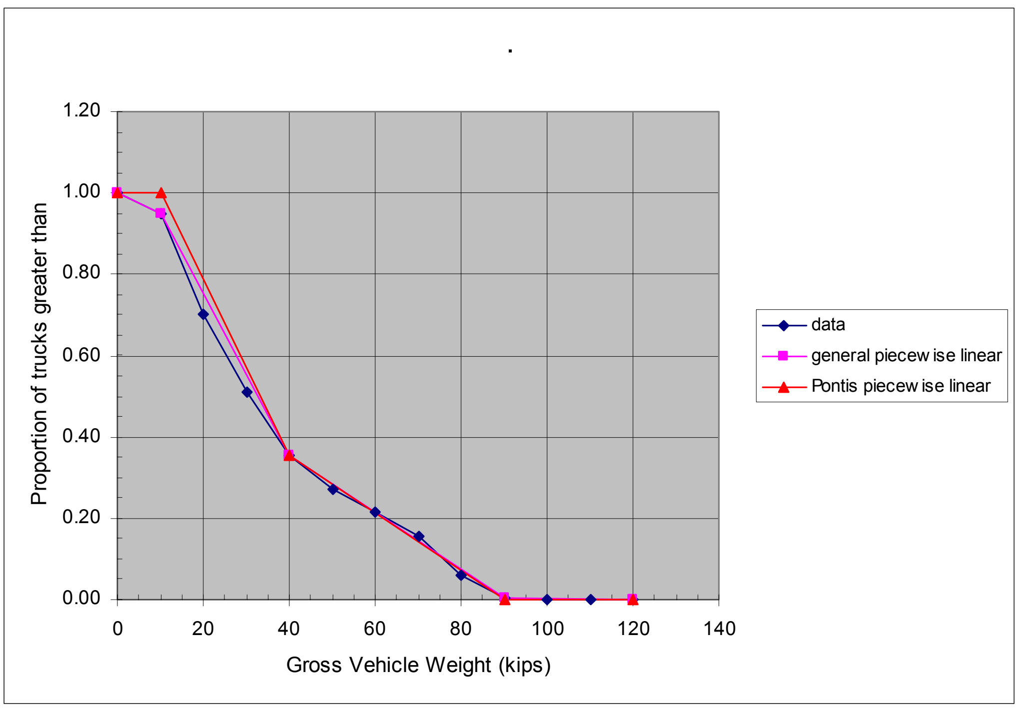


**Figure E.1. Truck Weight Piecewise Linear Regression for FDOT TTMS (WIM) 9924, I-110, Pensacola, FL. Functional Class 11.**

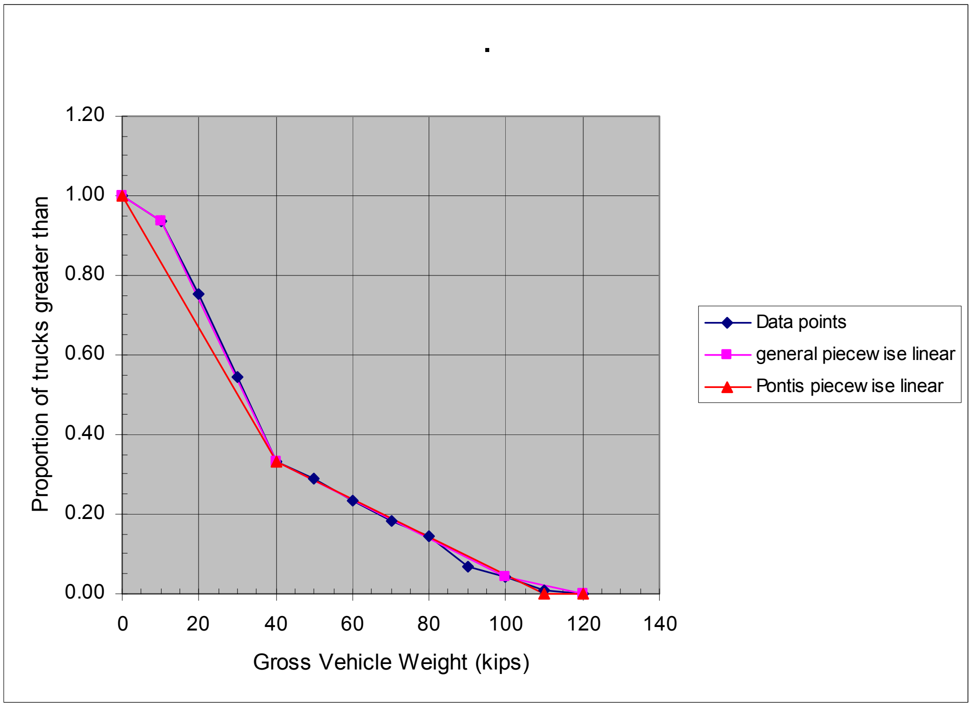


**Figure E.2: Truck Weight Piecewise Linear Regression for FDOT TTMS (WIM) 9946, US-98, St. Marks, FL. Functional Class 06.**

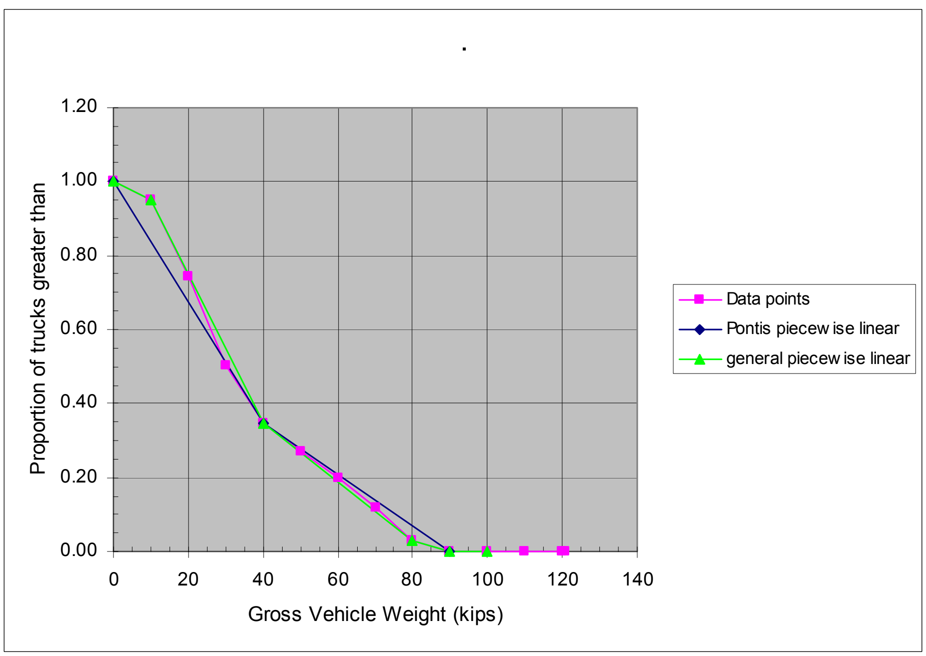


**Figure E.3: Truck Weight Piecewise Linear Regression for FDOT TTMS (WIM) 9940, SR-267, Quincy, FL. Functional Class 07.**

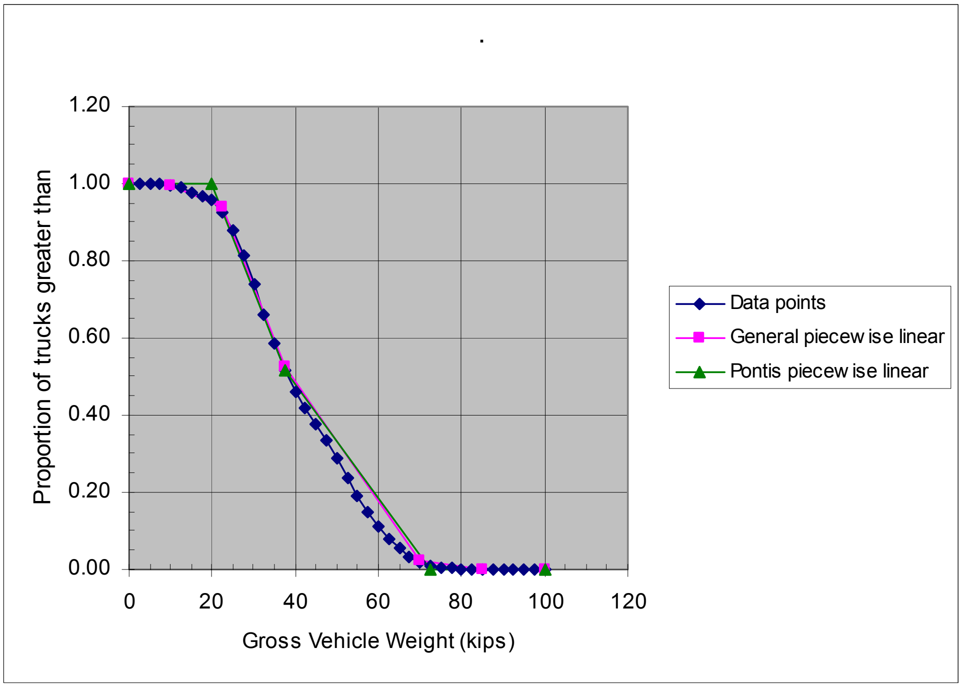


**Figure E.4: Truck Weight Piecewise Linear Regression for FDOT TTMS (WIM) 9935, US-27, Palm Beach, FL. Functional Class 02.**

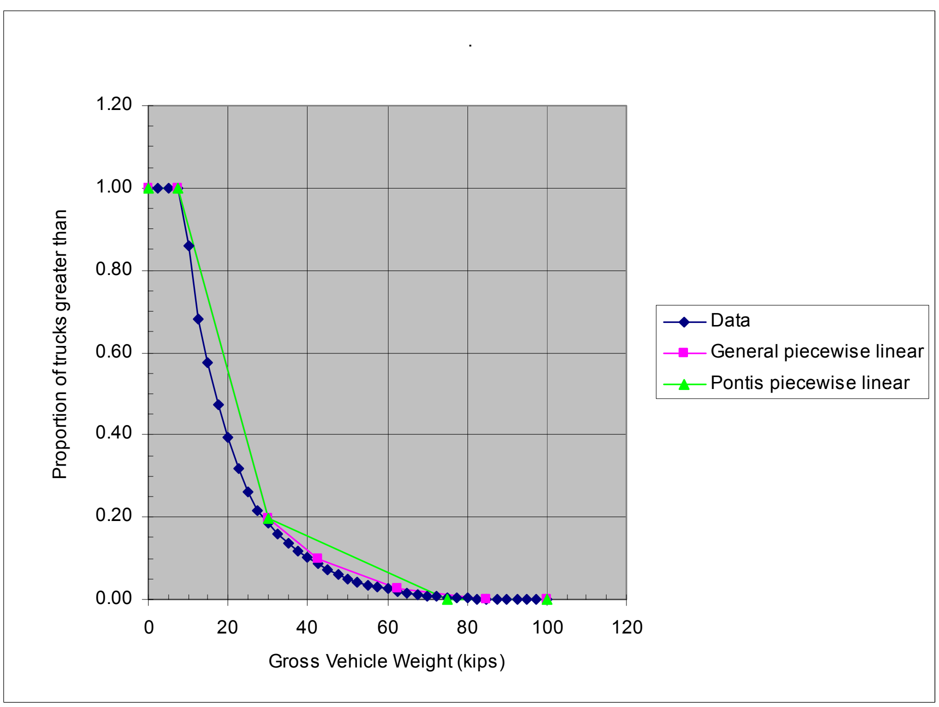


**Figure E.5: Truck Weight Piecewise Linear Regression for FDOT TTMS (WIM) 9921, US-1, Jupiter, FL. Functional Class 02.**

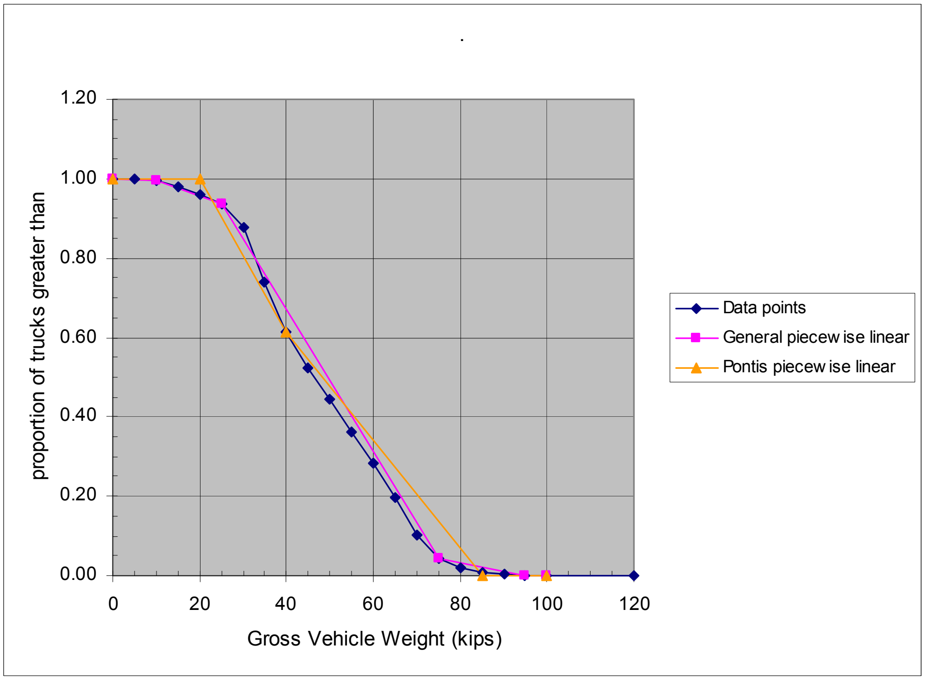


**Figure E.6: Truck Weight Piecewise Linear Regression for FDOT TTMS (WIM) 9936, I-10, Lake City, FL. Functional Class 01.**

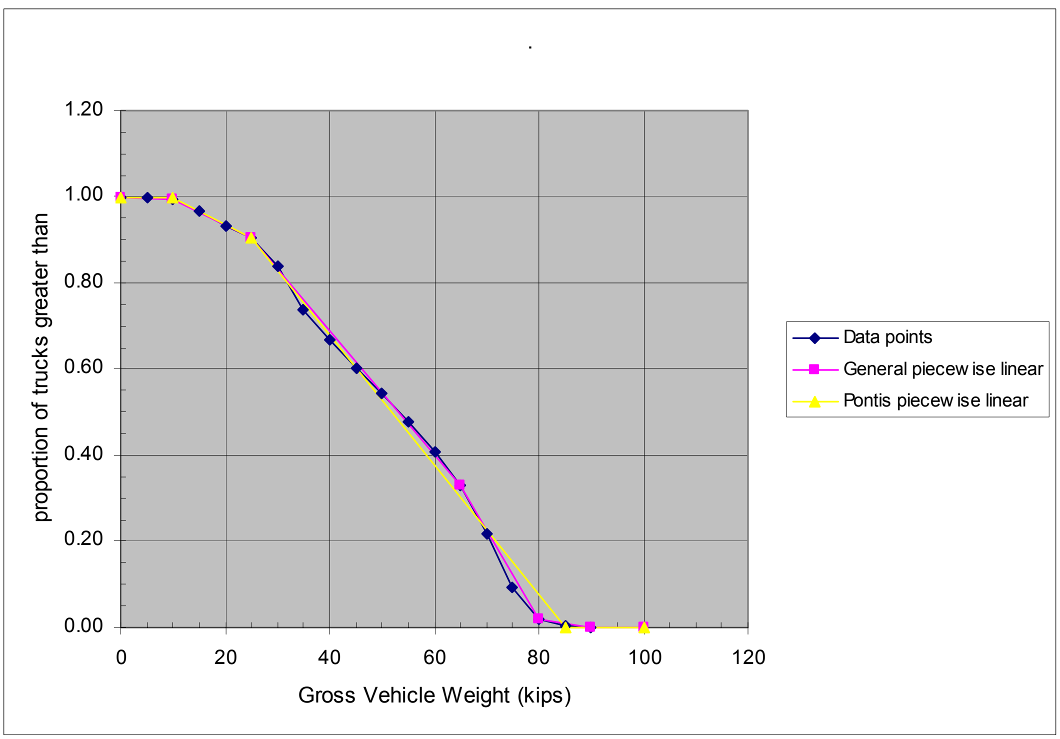


**Figure E.7: Truck Weight Piecewise Linear Regression for FDOT TTMS (WIM) 9901, I-10, Monticello, FL. Functional Class 01.**

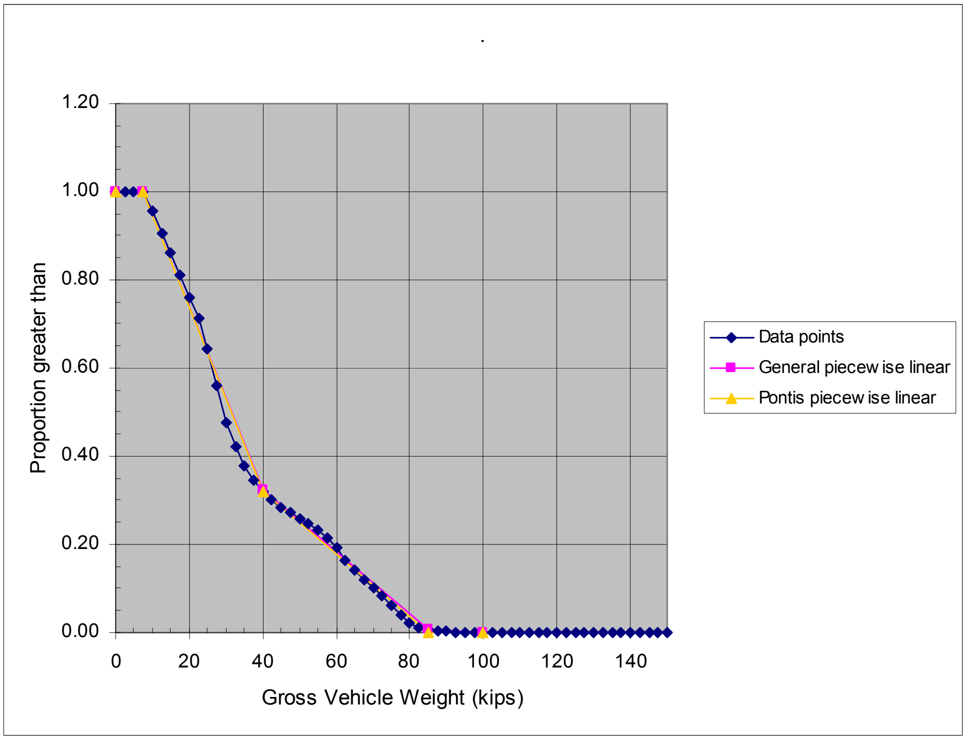


**Figure E.8: Truck Weight Piecewise Linear Regression for FDOT TTMS (WIM) 9908, US-319, Tallahassee, FL. Functional Class 14.**

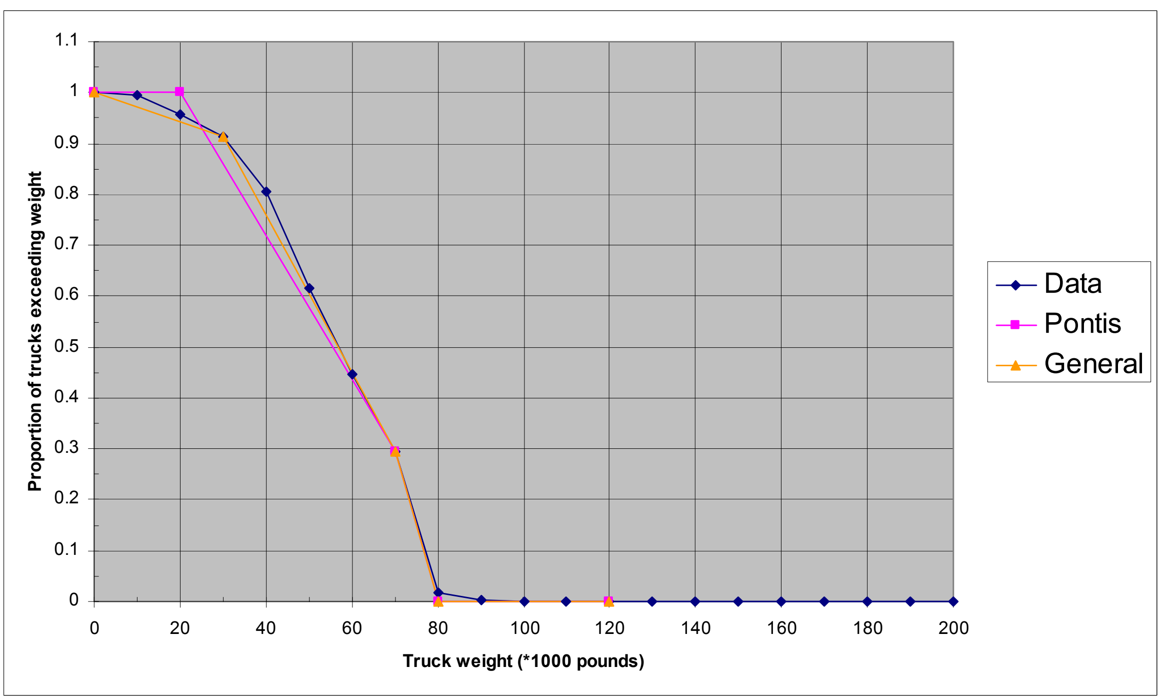


**Figure E. 9: Truck Weight Piecewise Linear Regression for Flagler Weigh Station, Interstate 95, Florida, April 1 -- May 22, 2003.**

**Table E.2. Equations for Best-fitting Piecewise Functions (Linear and Non-linear) for Truck Weights on Individual Interstate Roadways**

| Data Site | Functional Class | Best-Fitting Piecewise Equations: *Prob* (TruckWeight > x) = | Limits (ft.) | $R^2$ |
|---|---|---|---|---|
| Sneads Weigh Station | 01 | 1.0 | x # 10.0 | |
| | | $1.01680 + 0.00007x - 0.00015x^2$ | 10.0 < x # 80.0 | 0.98717 |
| | | $0.09000 - 0.00100x$ | 80.0 < x # 90.0 | 1.0000 |
| | | 0.0 | x > 90.0 | |
| White Springs Weigh Station | 01 | 1.0 | x # 20.0 | |
| | | $0.92632 + 0.00799x - 0.00024x^2$ | 20.0 < x # 80.0 | 0.99660 |
| | | $0.28311 - 0.00311x$ | 80.0 < x # 90.0 | 1.0000 |
| | | 0.0 | x > 90.0 | |
| Flagler Weigh Station | 01 | 1.0 | x # 10.0 | |
| | | $1.00450 + 0.00133x - 0.00017x^2$ | 10.0 < x # 80.0 | 0.99596 |
| | | $0.14626 - 0.00160x$ | 80.0 < x # 90.0 | 1.0000 |
| | | 0.0 | x > 90.0 | |

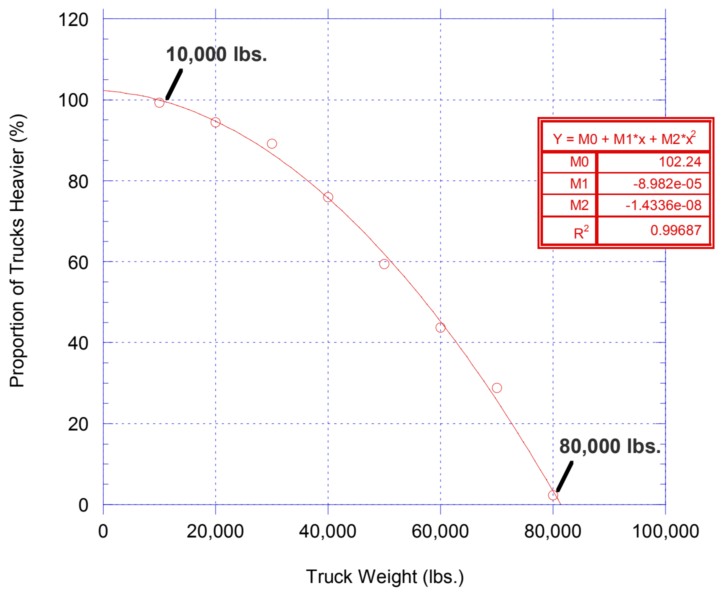


Figure E.10. 10,000 lb. – 80,000 lb Segment Weight Curve Function for Interstate Roadways

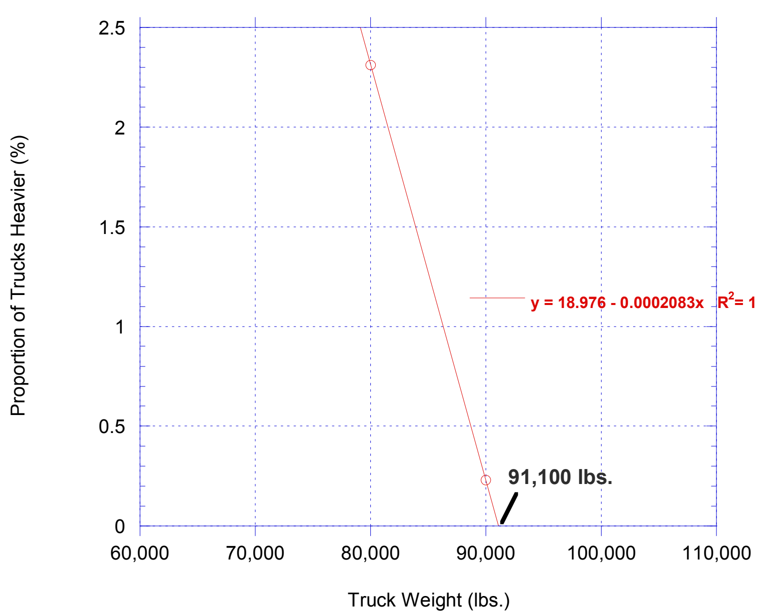


Figure E.11. 80,000 lb. – 91,000 lb Segment Weight Curve Function for Interstate Roadways

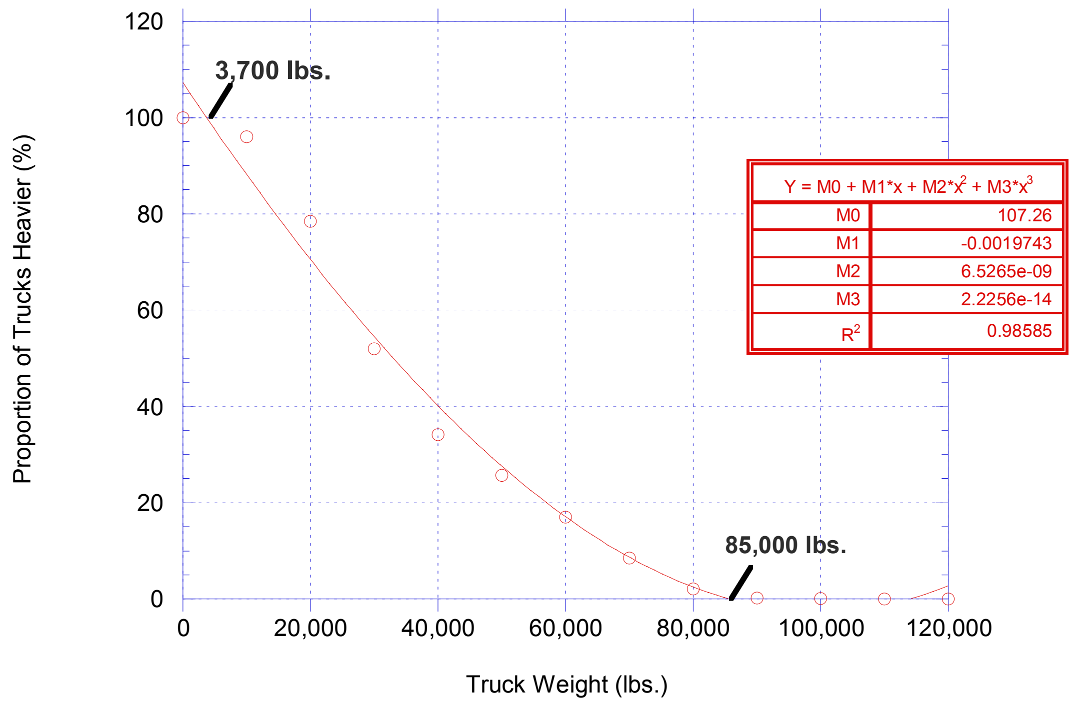


Figure E.12.. 3,700 lb. – 85,000 lb Segment Weight Curve Function for Non-Interstate Roadways

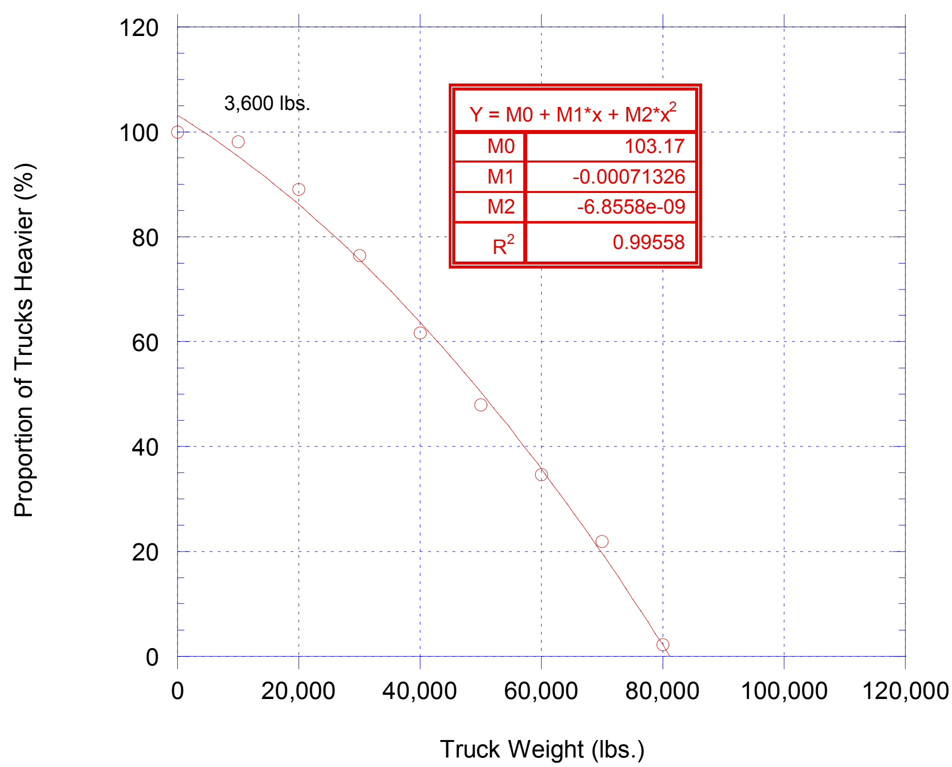


Figure E.13. 3,600 lb. – 80,000 lb Segment Weight Curve Function for All Florida Roadways

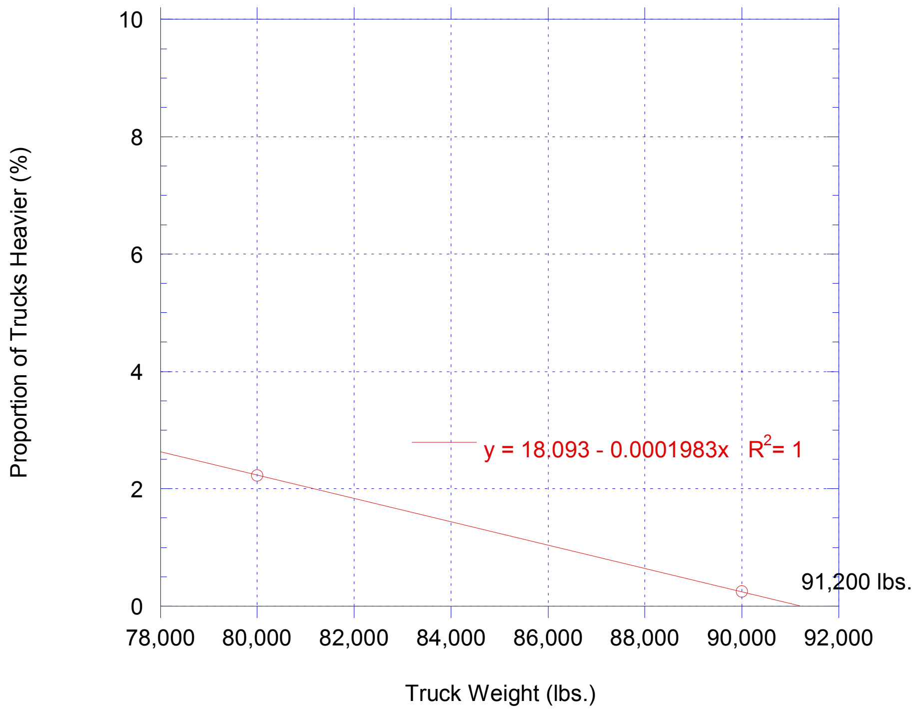


Figure E.14. 80,000 lb. – 91,200 lb Segment Weight Curve Function for All Florida Roadways

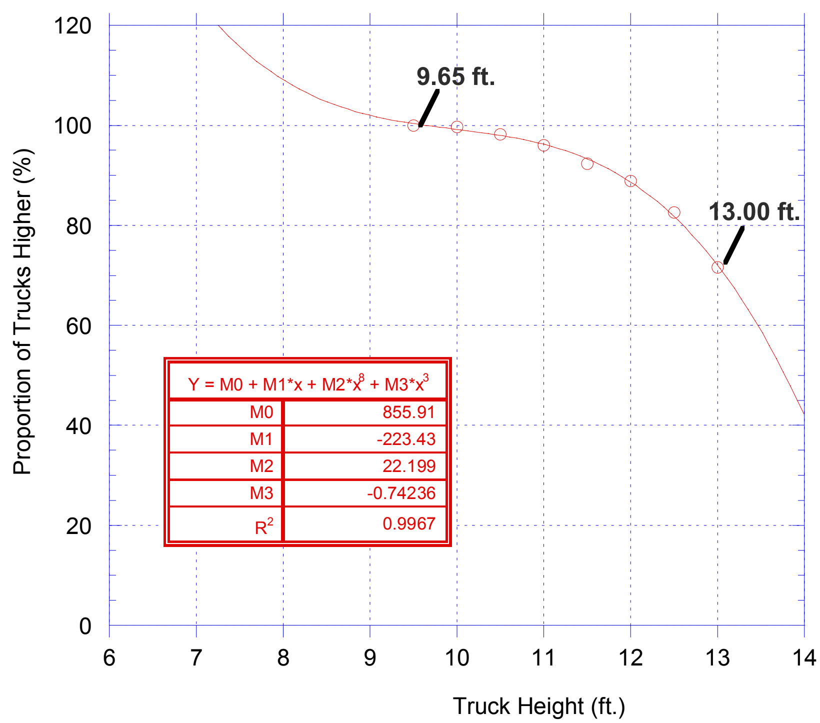


Figure E.15. 9.65 ft. – 13.0 ft. Segment Height Curve Function for Interstate Roadways

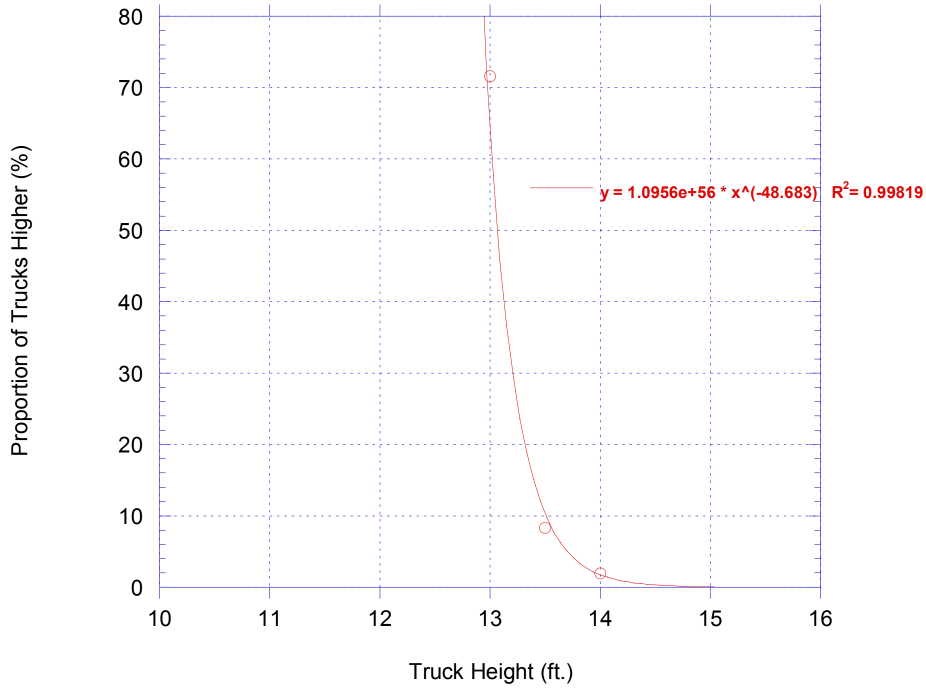


Figure E.16. 13.0 ft. – 14.0 ft. Segment Height Curve Function for Interstate Roadways

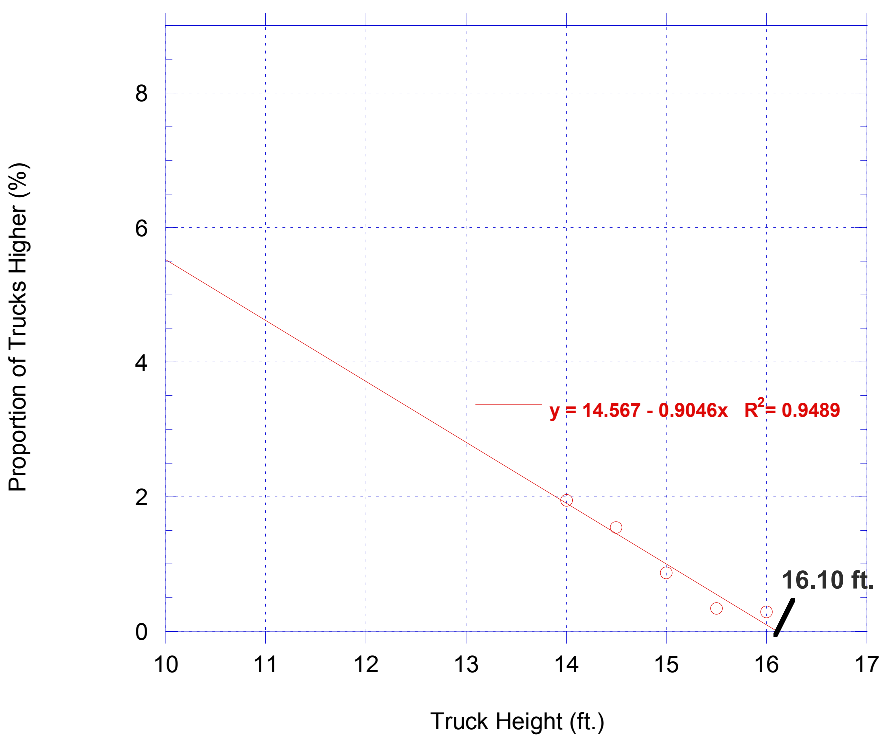


Figure E.17. 14.0 ft. – 16.1 ft. Segment Height Curve Function for Interstate Roadways

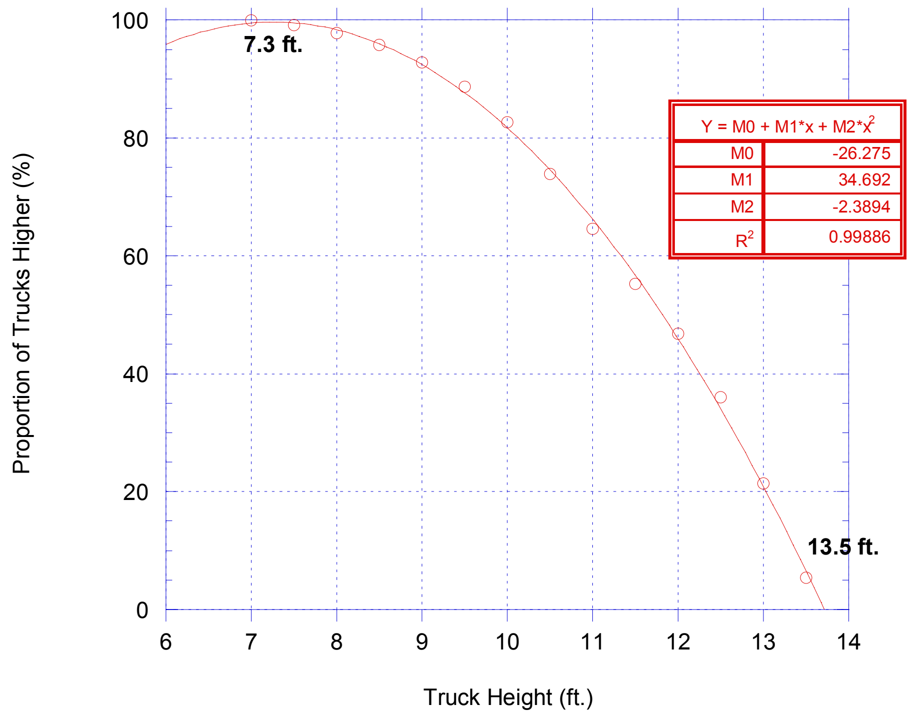


Figure E.18. 7.3 ft. – 13.5 ft. Segment Height Curve Function for Non-Interstate Roadways

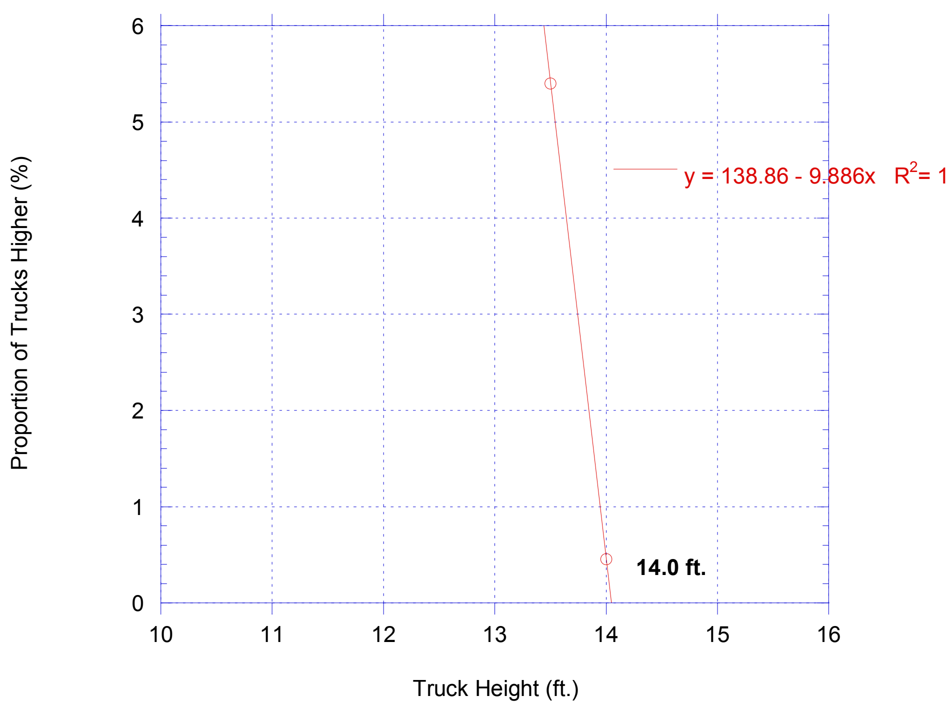


Figure E.19. 13.5 ft. – 14.0 ft. Segment Height Curve Function for Non-Interstate Roadways